C6. Concrete (Systematic Rehabilitation)

C6.1 Scope

The scope of Chapter 6 is broad, in that it is intended to include all concrete structural systems and embedded connection components. Concrete masonry systems are covered in Chapter 7. Exterior concrete cladding is covered in Chapter 11.

Material presented in Chapter 6 is intended to be used directly with the Analysis Procedures presented in Chapter 3.

C6.2 Historical Perspective

This section covers a broad range of older existing reinforced concrete construction. A historical background is provided in the following paragraphs to aid in defining the scope, as well as to provide guidance on likely characteristics of existing construction. Tables 6-1 through 6-3 of the *Guidelines* also contain historical material properties, as illustrated in the following text.

History of Reinforced Concrete Materials. Concrete as material has engineering properties that are highly complex. Despite the complex nature of the material, the characteristics of concrete are usually summarized in terms of the compressive strength. It is assumed that other properties—such as the concrete contribution to shear strength, the elastic modulus, the shear modulus, and the tensile strength—are related to the compressive strength by standard relationships that are expressed in the provisions for design of new buildings. It has been found that this approach is suitable both for design of new buildings and for evaluation of existing buildings. No change in this approach is suggested.

Concrete compressive strengths have increased steadily over the years. Results of tests of cores from early buildings may be found to be highly variable, but typical maxima strengths are in the range of 2500–3000 psi. These values are consistent with those found in building codes of the time of construction, and in textbooks of the same era. Currently, these same values are the minimum that will be found in practice, and concrete strengths for routine cast-in-place construction generally are in the range of 4000–5000 psi, with considerable variation in different areas of the United States. Strengths of concrete in prestressed construction are generally specified in the range of 6,000–10,000 psi. Some specialized concretes, such as for columns in tall buildings, may be found with compressive strengths as high as 18,000 psi.

To the greatest extent possible, concrete structures should be inspected throughout for evidence of concrete that has properties different from the average or from test results that may have been obtained. This is particularly important for very early structures, or structures for which the test results have been very erratic. Visual evidence may include changes in color or consistency of the concrete, poor compaction, distress, or obvious deterioration.

Reinforcing bars also have shown a consistent increase in strength over the years. Early bars may be structural grade with a yield strength of 33,000 psi, while 60,000 psi yield is the current design standard. However, high-strength bars have been available for many years, from early hard grade bars with 50,000 psi yield to the current 75,000 psi yield.

Proprietary bar shapes used in early construction can be expected to have strengths similar to those of standard bars. These include shapes such as square bars, twisted bars, and plain round bars. Plain bars, without deformations, will often be found in early structures. Bond capacity values should be reduced accordingly (see Section C6.3).

Chronology of the Use of Reinforced Concrete in Buildings. The date of construction correlates with the architectural treatment, type of construction, construction methods, materials, and building codes. These factors in turn influence seismic performance, and must be considered in evaluation and design of retrofit measures. Types of construction and, to a certain extent, construction methods, are discussed in the following sections.

1900–1910. Construction of buildings using reinforced concrete began at about the start of the 20th century, as portland cement became commercially available and more individuals became familiar with its characteristics. As would be expected, the first buildings mimicked the structural systems common with other materials, so we find frame buildings with concrete columns, girders, beams, and slabs. Concrete bearing wall buildings are found as well, but these seem to be less common in early construction than the frame configuration.

Concrete in some early buildings may have been mixed by hand, batch by batch, in wheelbarrows immediately adjacent to where it would be placed in the structure. The resulting concrete would be highly variable in quality within very short distances in a structure—a possibility to be kept in mind in analyzing the strength of very early structures.

Exterior walls in frame buildings of this era commonly were either masonry infills in the plane of the frame, or curtain walls partially within the frame and partially outside it. Infill materials might be brick or concrete masonry, which are relatively strong but brittle, or clay tile stucco or terra cotta, which are weak and brittle. Exterior facing materials commonly were brick or stone masonry.

Most frame buildings constructed in this period had multiple interior partitions, which contributed to stiffness, strength (to a certain degree), and internal damping. Original construction materials included clay tile, lath (wood or metal) and plaster, or masonry. In the intervening years, these partitions may have been moved repeatedly, or removed without replacement. The replacements in recent years are likely to be gypsum board on wood or metal studs—a weaker, more flexible system, but much lighter. In many cases, the original partitions may not have been replaced at all, leaving an open floor plan. The resulting current configuration in many of these older buildings may be mixture of interior partitions of many types, with the accompanying variations in weight, stiffness, and strength, and with some partitions missing entirely. These variations may be within a floor, and between floors. The resulting eccentricities in mass and stiffness, and vertical variations, should be taken into account in the analysis process.

1910–1920. Dates for introduction of specific structural systems are always approximate, but it is fair to say that the development of specialized systems in cast-in-place concrete began about this time. A notable example is the flat slab floor system, which utilizes the heterogeneous nature of concrete to create a floor system more free of directional characteristics. The flat slab floor system consists of an array of columns, not necessarily on a rectangular grid, supporting a constant thickness floor that does not have beams. Most early examples were designed for heavy loads, so that it was necessary to thicken the floor in the vicinity of the columns. These thickened portions, called drop panels, provided increased moment and shear capacity. In many cases, enlargements of the tops of the columns, called capitals, were also provided.

These early flat slabs often were reinforced with proprietary systems using reinforcement arrangements that seem very strange when compared with current practice. Elaborate combinations of multiple directions of bars, interlocking circles, and other complex forms are found. The possibility of the presence of one of these systems should be considered if location of bars by electromagnetic means is being attempted in one of these early buildings. Similarly, reinforcing steel optimization became more attractive; continuity of bars at member connections must be carefully considered.

About this same time period, techniques for reduction of structural weight became of interest, particularly for buildings with lighter live loads. Concrete joist construction was developed, where in one direction the beam and slab construction became a constant depth arrangement of narrow, closely spaced (about 30 inches, typically) beams called joists, with very thin concrete slabs between them to complete the floor surface. The construction of the floor system is started by building a form work platform on which void formers are placed in the desired pattern. Reinforcement for the joists and slab are placed. Concrete is then cast around and above the void formers to create a ribbed slab with a smooth upper surface.

The void formers may be steel pans open on the bottom, or they may be hollow clay tiles, which would result in a smooth ceiling line. The smooth appearance may have been enhanced by a coat of hard plaster. As far as evaluation is concerned, the significance is that what appears to be solid concrete—and may sound like solid concrete when tapped lightly with a hammer—may actually be weak, brittle clay tile in some locations. Care should be taken to ensure that a proposed retrofit element bears on concrete, not on an area of concealed voids such as may be represented by the clay tile. Also, the additional weight of the masonry forming materials must be accounted for.

A variation on the concrete joist system is the waffle slab system. As the name implies, the joists run in perpendicular directions so that the crossing patterns leave square voids that appear on the underside not unlike a waffle pattern. Some early versions used clay tile left in place and plastered over on the bottom, so the above cautions about the same construction in concrete joists also apply for such waffle slabs. More recent examples—using metal pans or cardboard forms—leave the system exposed for architectural effect, which makes identification very easy.

All these structural systems are still in use for new construction, although clay tile void formers are no longer in use in the United States. It should be noted that the heavy, and relatively deep, floor systems are likely to create a strong beam-weak column situation that will be discussed further in conjunction with concrete construction.

About this same time period, use of concrete bearing walls became more common, particularly for industrial structures and for commercial structures built against lot lines. For the most part these would be low-rise structures. The walls may have very little reinforcement, and may not be adequately connected to the floors and roof diaphragm.

1920–1930. This period represented an era of improvement more than one of innovation. Construction became more mechanized, so the likelihood of encountering localized variations in concrete quality was reduced, although voids due to poor consolidation are a possibility.

By this period, sufficient time had elapsed since concrete construction had become common that weak points in performance could be identified and corrected, at least for response to gravity loads. Seismic design was in its infancy, so it is likely that any intentional lateral-force-resisting systems found during evaluations will be proportioned for wind forces only.

1930–1950. This period was dominated by external events, namely the Depression and World War II, so progress in concrete construction was slight. Research went on, to a degree, and some refinement continued in design and construction, but for the most part building types and construction methods changed little in this period. The level of construction, particularly in the Depression, was only a fraction of what it had been earlier. Construction activity increased during and after the war, but most research efforts and refinements in materials and construction techniques were directed elsewhere.

1950–1960. This period saw a very rapid change in building systems, design methods, and construction practice. As a result of problems associated with the increased rate of change, buildings built in this period may well require closer scrutiny than their counterparts built earlier. The use of deformed reinforcing steel became prominent during this period, displacing smooth and proprietary systems.

More open interiors, and the use of lightweight metal or glass curtain wall exterior cladding, meant that frame buildings had less stiffness, and possibly less initial strength as well. Coupled with the fact that design for lateral loads in general, and seismic loads in particular, had still not reached relative maturity, these buildings may be found to have significant structural weaknesses. Specific concerns include the likely lack of confinement reinforcement in columns, joints, and potential beam hinge regions, which because of the increased flexibility may have increased demands compared to earlier construction.

The trend toward lighter and more flexible construction was particularly apparent in the case of flat slab/flat plate buildings, where the use of the flat plate configuration became more common for office and residential construction up to substantial heights. Many of these buildings had neither drop panels nor column capitals, relying solely on the frame action of the floor slab and columns for resistance to lateral loads. The small shear perimeters around the columns, which are forced to transfer the gravity load shears as well as the unbalanced moment due to lateral load, can be the weak points of these structures. Post-tensioning of these slabs became common by 1960.

On the positive side, seismic code provisions were beginning to be developed, and many of the issues still being addressed today had been identified. The appearance in the codes of lateral load provisions, for both wind and earthquake, was leading to the inclusion of identified portions of the building assigned to the lateral-force-resisting system.

A number of new concepts and construction methods were coming into use. Prestressing—both pretensioned and post-tensioned—was becoming a factor in building construction. Accompanying pretensioned concrete was a greater degree of precasting, but not all precast concrete was prestressed. Precasting was done both in off-site fabricating plants and on-site. On-site precasting was most commonly associated with tilt-up construction—used mainly for low-rise commercial light industrial, and warehouse buildings—or with lift-slab construction.

Bonded post-tensioning, in both cast-in-place and precast construction, was used mainly for heavy construction such as parking garages. Adequate grouting of the tendon ducts is an issue from both construction and current condition standpoints. Adequate ductility is an issue from the seismic analysis standpoint, as it is for all prestressed construction. Prestressing cable is not ductile.

Because of the lack of service experience (with the corollary of lack of building code guidance), and novel features, many of the early structures employing the new systems had problems. Notable examples were lack of proper accommodation of length changes in prestressed systems due to continuing creep, and consequent difficulty with connections between precast elements. Even after decades of experience, these problems are not entirely solved. For early structures, these items should always be checked for possible reduction of both vertical and lateral load capacity, and for cracked or broken connections.

Connections between precast units, and between precast units and adjacent members, are vital to the integrity of the gravity- and lateral-force-resisting systems in many applications. Examples are the connections between precast roof units, between wall panels, and between walls and roofs. One of the most notable examples of the latter is the connection between wood roofs and tilt-up walls, which have failed during earthquakes in several instances. Current code provisions prohibit the use of wood ledgers in cross-grain tension or bending, in an effort to minimize the likelihood of this type of failure.

Some unbonded post-tensioned structures were also appearing about this time. Early versions frequently lacked supplementary deformed bar reinforcement for crack control and strength enhancement at overload states, a deficiency that was reduced by improved code provisions. Early versions of these systems should be checked for this problem, and for tendon corrosion as well. Another problem deserving attention is the “lock-up” of forces from unbonded tendons with vertical concrete wall systems; this has been witnessed in numerous post-tensioned structures.

In lower seismic zones in particular, support bearing length and connections between roof and floor elements and their supports should be reviewed. The need for adequate support and ductile connections may not have been appreciated in the original designs.

Precast frame buildings began to become more common about this period as well. If the frame is proportioned and connected in such a way that hinging takes place other than at the joints, then the structure should behave much like its cast-in-place counterpart. However, if hinging takes place at connections between elements, the earthquake resistance of the structure should be reviewed very carefully with respect to brittle behavior.

The use of shear walls to resist lateral forces, as part of the basic design procedure, was formalized in this period. Shear walls had often been present in one way or another, but conscious use of rigid walls at selected location, size, and strength appears to date from this period. Earlier walls that serve a comparable function can be found as bearing walls, elevator shaft walls, and infill walls in frames.

Shear wall buildings tend to be much stiffer than frame buildings—this produces the advantage of reduction of drift and deformations, and the disadvantage of attracting higher internal loads than frame buildings. One of the most serious deficiencies occurs where shear walls do not extend all the way to the foundation. Supports for discontinuous shear walls have frequently been damaged in earthquakes.

Increased use of automobiles in this period led to a substantial increase in the number of parking garages, many of which often are of concrete construction. Several features of these structures present challenges, including the size, which invites significant dimensional changes when prestressed; unfavorable environment, which promotes deterioration; irregular framing, which invites unsymmetrical response to earthquake excitation; small story heights, which may encourage weak column and short column behavior; and problems with connections in precast systems.

1960–1970. This period represents improvement and consolidation in design, code provisions, and construction. Concerns for seismic design, and hence code requirements of seismic resistance, remained concentrated mainly in California and Washington. The *Uniform Building Code*, in use mainly in the western portions of the US, was being improved continually to deal with the seismic concerns summarized earlier in this section, as technology and research provided improved resistance in design. However, the seismic sections of the UBC were not adopted or enforced in many locations, and many important deficiencies remained to be resolved. In the remainder of the United States, building codes tended to ignore seismic issues, since it was not universally recognized at the time that many other areas were at substantial seismic risk.

A major development in concrete design in this era was the conversion of the code from allowable stress methods to strength methods. Concurrently, the concepts of assigning characteristics to a designated lateral-force-resisting system were being developed. Confinement and ductility in concrete detailing were described explicitly, though still not mandated by the codes. Improvements such as continuity in positive moment reinforcement, and joint shear provisions, made their appearance.

1970–1980. This was a period of continued development of seismic design in the western United States, but attention to seismic concerns in the eastern United States was still not extensive. The major San Fernando earthquake in 1971 resulted in additional understanding of earthquake demands and detailing requirements, and may be considered a turning point in development of ductile detailing and proportioning requirements for reinforced concrete construction in the western United States. Whereas earlier codes focused on providing strengths in structural members to resist code-specified forces, the western US codes developed during this period began to focus on aspects of proportioning and detailing to achieve overall system ductility or deformability.

In beam-column moment frame constructions, requirements emerged for transverse reinforcement in beams, columns, and joints, intended to reduce the likelihood of nonductile shear failures. Requirements that columns be stronger than beams—thereby promoting strong column-weak beam inelastic deformation modes—also appeared.

For shear wall buildings, requirements for ductile boundary elements of shear walls were incorporated in codes. These provisions include transverse reinforcement to confine concrete and restrain rebar buckling, and tension lap splices designed to sustain inelastic strain levels. Provisions to reduce the likelihood of shear failure also appeared in western US codes. For tilt-up wall buildings, improvements were made in tying together the various components.

1980–Present. This period represents a continuation of improvement and consolidation in design, code provisions, and construction, as an extension of the previous period. A significant change, however, has been the broadening of attention to seismic effects, from a regional outlook to a national outlook. The *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings* (BSSC, 1995) have become influential in FEMA efforts to focus attention on earthquakes as a national, not a regional, issue. The *Provisions* have been incorporated, with minor modifications, into the building codes in those portions of the United States not using the UBC. Since the *Provisions* differ little in their effect from the UBC, for the first time in the early 1990s there were well-established seismic code provisions in effect throughout the United States. The level of earthquake resistance of new construction should continue to improve, and there are reference standards to evaluate the capabilities of existing structures. A number of smaller magnitude earthquakes in the eastern United States and Canada demonstrated the vulnerability of the entire United States to seismic behavior, and prompted many municipalities to add appropriate design requirements.

Causes for Collapses in Reinforced Concrete Buildings. This section presents a brief discussion on causes of collapse in reinforced concrete (RC) buildings. The emphasis is on collapse as opposed to local failures. For example, the failure of a coupling beam may be dramatic, but it would not normally lead to an overall building collapse. Most collapses are ultimately caused by the deterioration and eventual failure of the gravity-load-carrying system for the structure.

- Poor Conceptual Design

  Certain structural design concepts that work well in nonseismic areas perform poorly when subjected to earthquake motions. Examples are frame structures with strong beams and weak columns, or frame structures employing soft (and weak) first stories. For either case, a single story sway mechanism can develop under lateral loading. Inelastic deformations will concentrate in this story, with the remainder of the structure staying in the elastic range of response. Even well-detailed columns will lose strength, stiffness, and energy absorbtion capacity due to the concentrated inelastic demands placed on this single story. Thus, complete structural collapse is a likely result.

Poor layout of structural walls during the initial design of a building leads to significant plan eccentricities between the center of mass and the center of lateral load resistance. Under lateral loading, torsional response modes will dominate, and large displacement demands will be placed on vertical elements farthest away from the center of stiffness. The vertical elements farthest from the center of resistance are usually perimeter columns. The large cyclic motions would typically put biaxial displacement demands on the columns; even well-detailed columns will typically fail under such extreme loading conditions.

Another poor design concept is to not provide adequate spacing between adjacent structures. When there is not adequate spacing, the buildings will “pound” against each other as they respond to the earthquake excitation. Clearly, structures are not normally designed to absorb pounding loads from adjacent structures. Also, these impulsive pounding forces can significantly alter the dynamic response of the structure in question. The 1985 Mexico City earthquake offered several examples of significant pounding damage and partial collapses of buildings due to pounding from an adjacent structure (Bertero, 1987).

- Column Failures

  Columns are the primary gravity-load-carrying members for most concrete structures. Therefore, most dramatic collapses of reinforced concrete structures during past earthquakes have been due to column failures. Common causes of column failure are discussed below.

  – Inadequate Shear Capacity

    Typical gravity and wind load designs will normally result in a design shear force significantly lower than the shear force that could be developed in a column during seismic loading. Early seismic designs that used factored loads—as opposed to a mechanism analysis—may also lead to column design shear forces well below potential shears that could act in the column during an earthquake. Another common problem is to artificially “shorten” a column by adding partial-height nonstructural partition walls that restrict the movement of the columns. The resulting short columns are stiff and attract much higher shear forces than they were designed to carry. There are numerous examples of column shear failures during past earthquakes.

  – Inadequate Confinement of Column Core

    Although most frame structures are designed using the strong column-weak beam philosophy, first-story columns often form plastic hinges during strong seismic loading. As in beam plastic hinging regions, the concrete core in a column plastic hinging region must be adequately confined to prevent deterioration of the shear and flexural strength of the column. This confinement requirement in a column is more severe because of the high axial load and shear that typically needs to be carried through the plastic hinging region. Again, there are numerous examples of failure of poorly confined columns during past earthquakes.

  – Combined Load Effects

    Poor design concepts, such as terminating shear walls above the foundation level, may result in columns that are required to carry very high axial compression and shear forces. If such columns do not have adequate confinement, there can be an explosive shear failure that is similar to the failure of the compression zone of an overreinforced beam subjected to bending and shear. A typical example would be a shear wall boundary column that extends down to the foundation while the wall terminates at the first-story level.

  – Biaxial Loading

    The problems of shear strength and confinement are commonly more severe in corner columns, especially if the building has significant eccentricity between the center of mass and the center of resistance. Corner columns need to have a higher degree of confinement (toughness) if they are to survive the biaxial displacement demands that will likely be placed on them. Examples of failure of corner columns are common in past earthquakes.

- Failures of Beams and Beam-Column Connections

  Failures in beams and beam-to-column connections are most commonly related to inadequate use of transverse reinforcement for shear strength and confinement. These are typically local failures and will not necessarily lead to collapse of the building.

  During severe seismic loading of a frame structure, plastic flexural hinging regions will develop at the beam ends. The shear in the beam at the formation of these hinging zones could be significantly higher than the shear forces the beam was designed for, leading to a shear failure. However, a more common problem is inadequate transverse confinement reinforcement in the beam plastic hinging zones. As the plastic hinge "works" during the earthquake, the lack of adequate confinement reinforcement will result in a steady deterioration of the shear strength and stiffness in the hinging zone.

  Both beam-to-column and slab-to-column connections can suffer a significant loss of stiffness due to inadequate shear strength and anchorage capacity in the connection. Both of these "failures" are related to inadequate use of confinement reinforcement in the connection, and improper detailing of the main reinforcement anchored in or passing through the connection. For buildings on firm soil, the loss of stiffness may lead to a reduction in the displacement response—or at least very little increase—because the period of the structure tends to lengthen. However, for structures on soft soils this loss of stiffness and lengthening of the building natural period may lead to an increase in the displacement response of the structure. The increased displacements mean higher eccentric (P-Δ) loads on the structure and can cause a total collapse. The 1985 Mexico City earthquake gives some examples of this type of failure (Meli, 1987).

- Failures of Slabs at Slab-Column Connections

  Slab-to-column connections that are adequate for gravity loading may suffer a punching shear failure when required to transfer gravity loads plus moments due to seismic lateral loads. Laboratory experiments as well as post-earthquake investigations have indicated that when the gravity load shear stresses are high on the critical slab section surrounding the connection, the connection has little ability to transfer moments due to lateral loads, and will fail in a brittle manner if the lateral load moments cause yielding of the slab reinforcement. This potential punching problem is a primary reason for not allowing slab-column frame structures in high seismic zones. Although punching may be considered as a "local" collapse, a potential exists for a progressive collapse of the entire structure. Some failures during the 1985 Mexico City earthquake are examples of this type of building collapse (Meli, 1987).

- Failures of Structural Walls

  Structural walls with inadequately sized or poorly confined boundary elements have suffered shear-compression failures at their bases when subjected to lateral forces large enough to force the formation of a plastic hinge at the base of the wall. Again, this is typically a local failure and will not normally result in the collapse of a building, because in most structures there are either other wall elements or frame members capable of carry the gravity loads. However, such wall failures can seriously compromise the safety of the structure and make required repairs difficult to accomplish after an earthquake.

  In long structural walls with a low percentage of vertical reinforcement, the tensile strains may become very large if the wall is forced to respond inelastically during an earthquake. The high tensile strains and high range of cyclic strain can lead to low-cycle fatigue fracture of the reinforcing bars. One example of this type of failure was observed following the 1985 earthquake in Chile (Wood et al., 1987). The building was a total loss and was demolished shortly after the earthquake.

- Special Problems with Precast Concrete Construction

  The major issue for precast concrete construction is proper connections between the various components of the structure in order to establish a load path from the floor masses to the foundation. There are numerous examples of failures of precast buildings and tilt-up construction during earthquakes, due to inadequate connections between the different components of the structure. In many cases the components were simply not adequately connected. The true seismic demand required to be transmitted through a connection was not properly investigated, resulting in an inadequate connection.

Diaphragm flexibility and the transfer of diaphragm forces to lateral-load-resisting elements were two major problems with precast parking structures that suffered partial or total collapse during the January 1994 Northridge earthquake. Large diaphragms composed of precast elements and a thin concrete topping will deform inelastically during earthquake excitation, and the effect of these deformations on connections to the supporting elements, as well as the response of the supporting element, must be considered. Also, reinforcement in shear transfer zones between diaphragms and lateral-load-resisting elements must be carefully designed to transfer forces between these elements, considering all possible failure modes.

C6.3 Material Properties and Condition Assessment

C6.3.1 General

Each structural element in an existing building is composed of a material capable of resisting and transferring applied loads to foundation systems. One material group historically used in building construction is concrete, which includes both unreinforced and conventionally reinforced, and prestressed forms of construction. Of these, conventionally reinforced concrete has received the greatest use in buildings, from single elements such as the foundation system through primary use in frames and the superstructure. Concrete structural elements in the US building inventory have a wide diversity in size, shape, age, function, material properties, and condition, as cited in Chapter 4 of the *Guidelines*. Each of these factors has a potentially significant influence on the seismic performance of a particular building. This section is concerned with the influence of material properties and physical condition on the structural performance.

It is essential that the seismic rehabilitation effort include provisions to quantify material properties and condition during the early stages of work. Many references exist to support the determination of properties and assessment of physical condition. These references, and their recommended implementation, are addressed in this section. The focus of the materials testing and condition assessment program shall be primary gravity- and lateral-force-resisting elements.

C6.3.2 Properties of In-Place Materials and Components

C6.3.2.1 Material Properties

The primary properties of interest in an existing concrete structure are those that influence the structural analysis and rehabilitation effort. Both classical structural design and analysis of concrete, as well as typical code-prescribed requirements, are commonly based on the following strengths, which also dictate virtually all concrete component elastic and inelastic limit states:

- Compressive strength, modulus of elasticity, and unit weight of concrete; splitting tensile strength of lightweight aggregate concrete
- Yield strength and modulus of elasticity of reinforcing and connector steel
- Tensile (ultimate) and yield strength of prestressing steel reinforcement

Other material properties—such as concrete tensile and flexural strength, dynamic modulus of elasticity, and modulus of rupture; reinforcing steel bond strength and ductility; and relaxation properties of prestressing steels—may also be desirable. There are standard tests to measure these properties; most of these tests have been standardized by the ASTM. In general, accurate determination of these properties requires removal of samples of specific dimensions for laboratory testing. As indicated in Section C6.3.2.3, approximation of concrete compressive strength may also be obtained nondestructively. Samples removed shall also be examined for condition prior to mechanical testing (see Section C6.3.3).

Many factors affect the in-place compressive strength of concrete, including original constituents and mix design, age, thermal and environmental exposure history, load history, creep effects, and many others. These factors commonly introduce a certain amount of strength variability, even within specific components of a building. Additional variability may be introduced during the sampling and testing of the concrete. Thus, the derivation of existing concrete strength must be carefully approached by the design professional.

The yield strength of conventional reinforcing steel and connector materials used in concrete construction generally remains constant for the life of the building. Certain environmental conditions may weaken the steel, but these are generally confined to exposures in specific industrial and chemical plants, or buildings exposed to ocean spray or road salts. In addition, it is common for the same grade of steel (e.g., yield strength of 60,000 psi) to be used throughout a building.

The ultimate strength of prestressing steels is also generally a constant throughout the lifespan of a building. However, certain corrosive environments may alter the metallurgical structure of the steel, resulting in a weakening effect or embrittlement. In addition, relaxation of the steel, concrete volume changes, creep, and other factors may contribute to a loss of the originally introduced prestress.

Determination of other material properties may be warranted under special conditions (e.g., presence of archaic reinforcing, significant environmental exposure, special prestressing system). The design professional should consult with a concrete consultant to identify these properties if such special conditions exist.

C6.3.2.2 Component Properties

Concrete component properties include those that affect structural performance, such as physical size and thickness, geometric properties, condition and presence of degradation, and location and detailing of the reinforcing steel system. The need for tolerances in concrete construction, and factors such as concrete volume change and permeability, also affect as-built component properties. Design professionals responsible for the reanalysis of an existing building require an understanding of actual properties in order to model behavior properly.

The following component properties are cited in the *Guidelines* as important to evaluating component behavior; explanations are provided in parentheses:

- Original and current cross-sectional area, section moduli, moments of inertia, and torsional properties at critical sections (needed to establish appropriate section properties for capacity and allowable deformation checks)
- As-built configuration and physical condition of primary component end connections, and intermediate connections such as those between diaphragms and supporting beams/girders (needed to assess load transfer in the building)
- Size, anchorage, and thickness of other connector materials, including metallic anchor bolts, embedments, bracing components, and stiffening materials, commonly used in precast and tilt-up construction (materials commonly identified as “weak links” in building performance)
- Characteristics that may influence the continuity, moment-rotation, or energy dissipation and load transfer behavior of connections (needed to assess load transfer, and to understand connection behavior and implications on building deformation)
- Confirmation of load transfer capability at component-to-element connections, and overall element/structure behavior (needed to ensure element integrity and stability)

An important starting point for developing component properties is the retrieval of original design/construction records, including drawings. Such records may then be used at the building site for as-built comparison and conformance checks. The process of developing component properties and inspecting of the physical condition of a concrete structure is commonly referred to as “condition assessment” or “condition survey.”

C6.3.2.3 Test Methods to Quantify Properties

Concrete. The sampling of concrete from existing structures to determine mechanical and physical properties has traditionally employed the use of *ASTM C 823, Standard Practice for Examination and Sampling of Hardened Concrete in Constructions* (ASTM, 1995). All sampling shall be preceded by nondestructive location of underlying reinforcing steel to minimize sampling effects on the existing structure. In general, the property of greatest interest is the expected compressive strength, f_c' .

The accurate determination of mechanical properties of existing concrete in a building requires the removal of core samples (sawed beams for flexural tests) and performance of laboratory testing. The sampling effort shall follow the requirements of *ASTM C 42, Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete* (ASTM, 1990) (sawed beams should not be used unless core extraction is prohibitive). The testing of core concrete to determine mechanical properties shall follow specific ASTM procedures relative to the property of interest:

C 39, Standard Test Method for the Compressive Strength of Cylindrical Concrete Specimens

C 496, Test of Splitting Tensile Strength of Concrete

C 78, Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)

C 293, Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Center-Point Loading)

Derivation of in-place concrete strength from core samples taken requires statistical analysis and correlation of core strength to actual strength. A recently developed procedure (Bartlett and MacGregor, 1995) for this correlation involves the following equation:

$$f^{i}_{c,ip} = F_{l/d} F_{dia} F_r F_{mc} F_d f_c \qquad \text{(C6-1)}$$

where: $f^{i}_{c,ip}$ is the equivalent in-place strength for the *ith* core sample taken from a particular concrete class, and f_c is the measured core strength. The other expressions are strength correction factors for the effect of length to diameter ratio ($F_{l/d}$), diameter of the core (F_{dia}), presence of reinforcing steel (F_r), moisture condition of the core (F_{mc}), and strength loss due to damage during drilling (F_d). Mean values for these coefficients may be used, as derived from the following table:

Factor	Mean Value	Variability (%)
$F_{l/d}$: l/d ratio[a]		
Soaked[b]	$1 - \{0.117 - 4.3 \times (10^{-4}) f_c\} \times (2 - l/d)^2$	$2.5(2 - l/d)^2$
Air dried[b]	$1 - \{0.144 - 4.3 \times (10^{-4}) f_c\} \times (2 - l/d)^2$	$2.5(2 - l/d)^2$
F_{dia}: Core diameter		
50 mm	1.06	11.8
100 mm	1.00	0.0
150 mm	0.98	1.8
F_r: bars present		
None	1.00	0.0
One bar	1.08	2.8
Two bars	1.13	2.8
F_{mc}: Core moisture		
Soaked[b]	1.09	2.5
Air dried[b]	0.96	2.5
F_d: Damage due to drilling	1.06	2.5

[a] f_c is in MPa; for f_c in psi, the constant is $-3(10^{-6})$.

[b] Standard treatment specified in ASTM C 42.

This procedure should be utilized for determining the compressive strength for use in structural calculations, using the following approach. The equivalent in-place concrete strength for structural analysis shall consist of the mean of the converted core strengths from Equation C6-1 as:

$$f_{c,ip} = \frac{(f^{1}_{c,ip} + f^{2}_{c,ip} + \ldots + f^{n}_{c,ip})}{n} \qquad \text{(C6-2)}$$

where $f^{1}_{c,ip}, f^{2}_{c,ip}, \ldots f^{n}_{c,ip}$ are the equivalent compressive strengths computed from individual cores sampled (as computed via Equation C6-1) and n is the total number of cores taken from the particular concrete class.

The variability in measured core strengths should also be checked to: (1) determine the overall quality of the concrete, (2) determine if enough core samples were removed, (3) eliminate error, (4) properly identify outliers, and (5) make any needed adjustments to $f_{c,ip}$. The standard deviation, variance, and coefficient of variation should be checked via the following equations:

$$Q_c = [(f^{1}_{c,ip} - f_{c,ip})^2 + (f^{2}_{c,ip} - f_{c,ip})^2 + \ldots + (f^{n}_{c,ip} - f_{c,ip})^2] \qquad \text{(C6-3)}$$

$$S_c = (Q_c)^{0.5} \qquad \text{(C6-4)}$$

$$C.O.V. = \left[\frac{S_c}{f_{c,ip}}\right] \tag{C6-5}$$

where:

Q_c = Variance

S_c = Standard deviation

C.O.V. = Coefficient of variation

Further reduction of the equivalent strength values is suggested by the literature (Bartlett and MacGregor, 1995) to improve upon the confidence in results; it is reported that the probability that the in-place compressive strength is less than f'_c is 13.5% (rounded to 14%). As opposed to further reduction of correlated values, if the *C.O.V* is less than 14%, then the mean strength from testing may be used as the expected strength in structural analyses ($f'_c = f_{c,ip}$). The *C.O.V.* cut-off value was established to account for testing errors, damage from improper coring, and other factors that may alter individual test results as noted in the literature. However, if the coefficient of variation from this testing exceeds 14% or the results are greater than 500 psi below specified design, f'_c , further assessment of the cause through additional sampling/ testing is needed. Such causes might be, among others, poor concrete quality, an insufficient number of samples/tests, or sampling or testing problems. In general, the expected strength taken from results with higher variation should be a maximum of the mean less one standard deviation ($f'_c \le f_{c,ip} - S_c$). The design professional may further reduce the expected strength (and gain confidence in actual strength levels) if concrete quality or degradation are observed. The results should also be examined to ensure that one or more outliers (e.g., individual test results with large differences from other tests) are not influencing results. Outliers should be dispositioned per *ASTM E 178, Standard Practice for Dealing with Outlying Observations*.

Appropriate values for other strengths (e.g., tensile, flexural) shall be derived from the referenced ASTM tests and accepted statistical methods.

Other nondestructive and semi-destructive methods have been established to estimate the in-place compressive strength of concrete (ACI, 1995a). Methods applicable to hardened concrete, with referenced ASTM procedures, include the ultrasonic pulse velocity method (*ASTM C 597*), penetration resistance methods (*ASTM C 803*), and surface hardness or rebound methods (*ASTM C 805*). However, to date, these methods have demonstrated limited correlation to strength, with high internal coefficients of variation. Because of these constraints, and the need for calibration standards for each method, substitution of these methods for core sampling and laboratory testing is prohibited. These methods may be economically used, however, to qualitatively check concrete strength uniformity throughout the structural system as opposed to core drilling samples. The guidance of ACI Report 228.1R-95 (ACI, 1995) should be used if nondestructive methods are to be employed in this manner.

Conventional Reinforcing Steel. The sampling of reinforcing and connector steels shall be done with care and in locations of reduced stress; sampled areas should be repaired unless an analysis indicates that the local damage produced is acceptable. Sample sizes should be per *ASTM A 470, Standard Test Methods and Definitions for Mechanical Testing of Steel Products*, with longitudinal, planar, or stirrup bars used as opposed to ties. There shall be a maximum of one sample taken at any one cross-section location, and samples should be separated by at least one development length (*ACI 364.1R*).

Determination of tensile and bend strength and modulus of elasticity of conventional reinforcing and connector steels shall be as defined in *ASTM A 370*. Included in the determination of reinforcing steel strength properties is the characterization of material type; bond strength with the existing concrete may also be of interest, but this is extremely difficult to accurately measure in field conditions. Reinforcing steels used before 1950 had various cross-sectional shapes (e.g. square, rectangular, round), surface conditions (e.g., ribbed, deformed, smooth, corrugated), and proprietary additions (e.g., herringbone shape, special deformations). Each of these characteristics may contribute to overall performance of the particular structure. The history of reinforcing steel and mechanical properties is summarized in *Evaluation of Reinforcing Steel Systems in Old Reinforced Concrete Structures* (CRSI, 1981). This document also recommends that older reinforcing steel systems be treated as 50% effective, the primary problems being with tensile lap splice deficiencies.

Connector steel properties shall be determined either via sampling and laboratory testing using *ASTM A 370*, or by in-place static tensile testing following the provisions of *ASTM E 488*, *Standard Test Methods for Strength of Anchors in Concrete and Masonry Elements*.

Prestressing Steel. Similar to conventional reinforcing, the yield and tensile strengths and modulus of elasticity of prestressing steels may be derived from testing in accordance with *ASTM A 370*. A maximum of one tendon per component shall be sampled, with a replacement tendon installed.

C6.3.2.4 Minimum Number of Tests

Determination of mechanical properties for use in the reanalysis of an existing building involves the completion of physical tests on *primary* component materials. Testing is not required on secondary components and other nonstructural elements, but may be performed to better analyze the building at the discretion of the design professional. The number of tests needed depends on many factors, including the type and age of construction, building size, accessibility, presence of degradation, desired accuracy, and cost. In particular, the costs for obtaining a statistically robust sample size and completing the destructive tests with a high level of confidence may be significant. A minimum level of testing for key properties that account for building size, concrete structure type, different classes of concrete, and variability was identified in *Guidelines* Section 6.3.2.4. It is recommended that a more comprehensive sampling program be established.

Minimum Sample Size. The minimum number of tests for determining material properties was identified from references including *ACI 228.1R* (concrete), various ASTM publications, and CRSI (reinforcing steel) guidelines. Typical coefficients of variation in concrete and steel materials were also cited from these references. In general, there is a statistical relationship between the minimum test quantity and the accuracy of the derived property. If prior information (e.g., design/construction records) exists, significantly higher confidence in the property of interest will be obtained with a reduced number of tests. Recent research (Bartlett and MacGregor, 1995) has shown that a minimum of three test sample should be taken if error is to be avoided, but at least six samples should be detected to identify outliers or specific values that deviate greatly from the others. Other documents (e.g., *ACI 228.1R*) have suggested that at least 12 cores be taken and tested to assess strength. The number of tests prescribed in the *Guidelines* was established with these reports as a basis. For small residential buildings, it is considered practical to obtain the expected strength from a small number of samples (such as three) as long as the coefficient of variation (*C.O.V.*) is low. However, with a larger tall building the number of tests may well exceed the minimum.

For reinforcing and prestressing steels, the minimum sample size is smaller than for concrete, because of material homogeneity, lower property variability, common material grades typically used throughout buildings, damage caused by sampling and need for repair, and ability to use samples to derive multiple properties. The sample size for prestressing steel shall be based on design information. If these data do not exist, sampling and testing are required. Because of the prestress, extreme care must be taken during disassembly.

Increased Sample Size. A higher degree of accuracy in material properties may be acquired by increasing the number of tests performed, supplementing required sampling/laboratory testing with rapid nondestructive methods, or using Bayesian statistics to gain further confidence.

Conventional statistical methods, such as those presented in *ASTM E 122* may also be used to determine the number of tests needed to achieve a specific confidence level. In general, these practices typically lead to a sample size much larger than the minimum number prescribed in the *Guidelines*. For reasons including access restrictions and cost, the design professional should consider using *ASTM E 122* or similar references to establish the actual sample sizes for a particular building.

Several nondestructive methods, including ultrasonic pulse velocity testing, may be effectively used to estimate concrete compressive strength and other in situ properties. Calibration of these methods with core test results is necessary for desired accuracy. The results may be used to improve confidence in representation of the core test results.

Bayesian statistics provide a means for improving confidence in material properties derived from a sample when prior information is available (e.g., design drawings, construction test records). A combination of strength data from cores and nondestructive methods may also be systematically combined via Bayes' theorem to obtain mean and standard deviation of compressive strength. This approach may also be used to justify use of a smaller sample size (e.g., minimum number of tests), especially if prior knowledge exists and a single concrete class was used in construction. Further information on the use of Bayesian statistics in material property selection is contained in Kriviak and Scanlon (1987) and Bartlett and Sexsmith (1991).

C6.3.2.5 Default Properties

Default values for key concrete and reinforcing steel mechanical properties were identified from the literature (e.g., CRSI, 1981; Merriman, 1911) in the Section 6.2 tables. Default values are provided for situations in which the design professional does not have materials test data from which in-place strengths may be derived. While these values have been further reduced in *Guidelines* Section 6.3.2.5, the design professional is cautioned against their use, as lower-strength or poorer quality materials may exist in the specific building in question. Concrete compressive strength in particular may be highly variable, even within a specific building. It is highly recommended that at least the minimum amount of testing in *Guidelines* Section 6.3.2.4 be carried out for confirmation of properties.

Another common condition in historic concrete construction was the use of contractor-specific proprietary systems, including floors and decks. Material properties in these proprietary designs may have been published in trade publications or other texts. The design professional is encouraged to research such references if the use of a proprietary system in the building is identified. Use of default values for these proprietary systems is not recommended. Also, as noted in CRSI (1981), it is recommended that a 50% reduction in effectiveness be applied to the reinforcing steel systems in historic construction.

C6.3.3 Condition Assessment

C6.3.3.1 General

The scope of the condition assessment effort—including visual inspection, component property determination, and use of supplemental testing—shall be developed by the design professional. The recommended scope of work includes all primary vertical- and lateral-load-resisting elements and their connections. Procedures for conducting the assessment and methods for use in assessing physical condition are referenced in the following section.

C6.3.3.2 Scope and Procedures

A condition assessment following the recommended guidelines of *ACI 201.2R* is recommended to be performed on all primary and secondary concrete elements of a building. The following steps should be considered.

1. Retrieve building drawings, specifications, improvement or alteration records, original test reports, and similar information.

2. Define the age of the building (e.g., when the building materials were procured and erected).

3. Compare age and drawing information to reference standards and practices of the period.

4. Conduct field material identification via visual inspection and in-place nondestructive testing of concrete.

5. Obtain representative samples from components and perform laboratory tests (e.g., compression, tensile, chemical) to establish in-place material properties per *Guidelines* Section 6.3.2.3. Samples shall be taken at random throughout the concrete building and elements. Test methods identified in Section 6.3.2 shall be used.

6. Determine chloride content and depth profile in concrete, if reinforcing steel corrosion is suspected, and determine the amount of loss of reinforcement due to corrosion, where applicable.

7. Visually inspect components and connections of the structural system to verify the physical condition.

Further information regarding the condition assessment of concrete structures may be found in *ACI 364.1R-94, Guide for Evaluation of Concrete Structures Prior to Rehabilitation*, and *ACI 201.2R-92, Guide for Making a Condition Survey of Concrete in Service*.

The samples removed for material property quantification may also be used for condition assessment. Significant data relative to the condition and quality of concrete (through petrographics and other tests) and reinforcing steel (degree of corrosion) may be established. In the event that degradation is observed in the visual assessment or review of retrieved samples, additional nondestructive and destructive tests should be used to quantify the extent. Such testing, referenced in the following paragraphs, should be performed by qualified personnel and testing firms.

Supplemental Test Methods for Concrete. Numerous nondestructive and destructive test methods have been developed for the examination and mapping of degradation and damage in concrete structures. Nondestructive methods (NDE) that may be used and their capabilities include:

Method	Capability/Use
Ultrasonic pulse-echo and pulse velocity	Indication of strength, uniformity, and quality; presence of internal damage and location; density and thickness estimation; location of reinforcing.
Impact-echo	Presence and location of cracking, voids, and other internal degradation.
Acoustic tomography	Presence and accurate location of cracking, voids, and other internal degradation.
Infrared thermography	Detection of shallow internal degradation and construction defects, delaminations, and voids.
Penetrating radar	Same as thermography; greater depth of inspectability.
Acoustic emission	Real-time monitoring of concrete degradation growth and structural performance.
Radiography	Location, size, and condition of reinforcing steel, and internal voids and density of concrete.
Chain-drag testing	Presence of near-surface delaminations and other degradation.
Crack mapping	Surface mapping of cracks to determine source, dimensions, activity level, and influence on performance.
Surface methods	Estimation of compressive strength and near-surface quality (methods such as Windsor probe, rebound hammer).

The practical application and usefulness of these methods is defined in numerous ACI and ASCE publications, including *ASCE Standard 11-90*, which compares and contrasts method capabilities for concrete element and damage types.

Additional physical properties for concrete may also be determined through use of other laboratory tests. Petrography (*ASTM C 856*) includes a series of laboratory tests performed on samples to assess concrete condition. These properties include entrained air quantity, depth of carbonation, degree of hydration, aggregates used, unit weight estimate, permeability, cement-aggregate reaction, and others.

Reinforcing System Assessment. The configuration and condition of reinforcing steel (conventional or prestressed) is especially critical to the future performance of the lateral- and vertical-force-resisting structural elements. The reinforcing steel is necessary to perform a variety of load resistance and transfer functions; to provide suitable ductility to the component and its connections; to prevent excessive straining, tensile stress development, and cracking in concrete from occurring; and for other purposes. Several means of evaluating the existing reinforcing steel system exist, including:

- Removal of cover concrete and direct visual inspection
- Local core sampling through a reinforcing bar(s)
- Nondestructive inspection using electromagnetic, electrochemical, radiographic, and other methods

Each method has positive and negative aspects. The greatest assurance of conventional or prestressed steel condition and configuration is gained through exposure and inspection. Critical parameters such as lap splice length, presence of hooks, development with concrete, and degree of corrosion can all be addressed in this manner. Of particular value is the ability to assess existing reinforcing detailing at critical component connections (for comparison to drawings and current code provisions). However, the expense, damage, and debris generated by this effort may be significant and disruptive to building use. The design professional should consider exposing a percentage of connections and the local reinforcing steel system to confirm drawing details and integrity of construction per the *Guidelines*.

Local core sampling through reinforcing steel is generally not a recommended practice because of the damage caused to the particular bar. However, during removal of cores for concrete strength testing, a sample containing portions of a bar may be inadvertently obtained. Such samples often allow direct visual inspection of local bar condition and interaction with surrounding concrete, and this information should be recorded.

Improvements in the area of nondestructive testing continue to be made. Existing proven technologies to identify bar location and approximate size include electromagnetic methods (via pachometers, profometers, and similar equipment), radiography, penetrating radar, and infrared thermography. To assess the activity level of corrosion in conventional reinforcing steel, half-cell potential (*ASTM C 876*), electrochemical impedance, and electrical resistivity methods have been used with some success. Electromagnetic methods have enjoyed the most use and have a good accuracy for round cross-section bars in uncongested areas (e.g., outer longitudinal steel in component spans). Reduced accuracy is demonstrated for locating square and other bar shapes, and at connections. Radiography, radar, and thermography have specific applications for which they provide important bar location information; however, available equipment capability, geometry, bar congestion, and component thickness present limitations to practical application.

To obtain details of prestressing steel location, remaining prestress, and physical condition requires direct exposure and inspection of anchorages, ducts (unbonded), and tendons (bonded). Measurement of remaining prestress in unbonded systems may be physically possible, depending on the system used and the end connection configuration. For accessible unbonded tendons, measurement of remaining prestress force may occur through use of calibrated hydraulic jacks and a lift-off procedure at one anchorage point, or through magnetic methods. Several nondestructive tests, including “coring stress relief,” have also been used to assess existing prestress levels (Brooks et al., 1990). Observation of corrosion in prestressing systems must also be carefully treated, as prestressing steel is susceptible to sudden fracture from hydrogen (corrosion byproduct) embrittlement, and often requires its full cross-sectional area to sustain applied loads. Widespread corrosion is indicative of a need for major rehabilitation.

Identification of the steel used in reinforcing systems may also necessitate the use of chemical testing on removed samples. The provisions of *ASTM A 751, Methods, Practices, and Definitions for Chemical Analysis of Steel Products* should be followed in this regard. If the carbon equivalent must be calculated to support welded attachment, the methodology in *AWS D1.4-92* (AWS, 1992) shall be followed.

Additional details on NDE and destructive testing are contained in *ASCE Standard 11-90* (ASCE, 1990).

Load Testing. A more thorough understanding of individual concrete components or elements may be gained through the performance of in-place load testing. Simulated gravity or lateral loads may be applied to an exposed component or element, with the response to loading measured via instrumentation (e.g., strain gauges, transducers, deflectometers) and data collection means. The measured results may be used to define structural performance under future load events and improve knowledge of condition and configuration. The aspect of performing load tests on concrete components is well defined in *ACI 437-94* and Chapter 20 of *ACI 318-95*. Load test results are also an acceptable means of establishing component capacity as stated in the model building codes (e.g., UBC), especially for elements constructed with alternative materials or techniques, and those with questionable capacity.

Limitations related to load testing include the expense of test performance, access requirements to the component(s), potential damage inflicted during the test, and difficulties posed by load application (e.g., high magnitude) and interpretation. In general, load testing has limited practicality in an existing, occupied building. However, it remains a viable option for certain components and building types.

Summary. The design professional of record is responsible for establishing the condition assessment and testing methods to be used as part of a seismic rehabilitation effort. Experienced personnel, proper equipment and procedures, accurate testing, and prudent interpretation of results are imperative to the determination of component/element structural capacity and deformation limits.

C6.3.3.3 Quantifying Results

The quantitative results from the condition assessment—such as component dimensions, significance of damage, and connection continuity—must be factored into the structural analysis and rehabilitation planning. Few resources exist that provide the design professional with assistance in quantifying the effects of damage on performance. If significant reinforcement corrosion or concrete loss is observed, it may be necessary to use load testing to assess in-place strength. If degraded elements are to be reused in the building, special attention should be given to mitigation of the degradation mechanism and stabilization of the element(s).

C6.3.4 Knowledge (κ) Factor

As noted in *Guidelines* Section 2.7.2 and the *Commentary* on it, a factor (κ) associated with the relative knowledge of as-built configuration and condition is used in component capacity and allowable deformation calculations. For concrete components, including foundations and columns, complete knowledge of reinforcing configuration and continuity is not likely to exist even if the original drawings are located. Other factors, such as actual material strength and resistance to applied loads, may not be completely understood. It is recommended that the lower κ factor of 0.75 be used if any concerns about condition or performance exist. This will provide a further factor of safety against unknown conditions.

C6.3.5 Rehabilitation Issues

After structural analysis of the building is completed, it may be determined that parts or all of the structure are seismically deficient. If rehabilitation is planned, a number of concrete materials issues must be considered in the design. Of paramount importance to concrete structure rehabilitation are the size, condition, location, and continuity of the reinforcing steel system, especially at element connections. It is recommended that the design professional pay significant attention to the reinforcing system in existing structures for reuse, attachment, treatment, and modification. If the strength, ductility, or confinement provided by the existing reinforcing system is in question, further examination of in-place conditions shall be performed. Section 6.3.6 of the *Guidelines* further addresses connection issues.

If a rehabilitation program is selected and attachment to the existing structure is required, a number of factors that may influence behavior must be addressed, including:

- Attachment to existing reinforcing steel, including required development, splicing, and mechanical or welded attachment
- Level of steady-state stress present in the components to be reinforced, and its treatment
- Elastic and strain-hardening properties of existing components and preservation of strain compatibility with any new reinforcement materials
- Confinement reinforcing steel and ductility requirements for existing and new components and their connections
- Prerequisite efforts necessary to achieve appropriate fit-up, continuity, and development
- Historic preservation issues
- Load flow and deformation at connections (especially beam-column joints, diaphragm, and shear wall connections where significant load transfer occurs)
- Treatment and rehabilitation of existing damage found during the condition assessment (e.g., concrete cracks, corrosion damage)

Many other material-related issues must be considered when planning seismic rehabilitation efforts. Increased attention should be paid to primary components and those with limited redundancy.

The design of all new components in the rehabilitation program shall be in accordance with the applicable state and local building codes and industry-accepted standards. Compatibility between new and existing components must be maintained at all times.

C6.4 General Assumptions and Requirements

C6.4.1 Modeling and Design

C6.4.1.1 General Approach

Procedures in the *Guidelines* for analysis and design of concrete components and elements are based on the analysis and design procedures of *ACI 318-95* (ACI, 1995). Those provisions govern, except where these *Guidelines* specify different procedures and where it is shown by rational analysis or experiment that alternate procedures are appropriate. Some modifications to the procedures of *ACI 318-95* are necessary because, whereas *ACI 318-95* covers new construction, these *Guidelines* cover existing construction and its seismic rehabilitation.

ACI 318-95 is a design document for new materials that includes proportioning and detailing requirements intended to produce serviceable and safe structures. Many of the rules of *ACI 318-95* are designed to automatically preclude certain types of nonductile failure modes for the design loading. An existing building structure may not have been designed according to the current requirements of *ACI 318-95*, and its design may not have considered the currently recognized seismic loading. Therefore, it is possible that seismic response may be controlled by brittle or low-ductility failure modes. The engineer is cautioned to examine all aspects of possible building response—including, but not limited to, response modes associated with flexure, axial load, shear, torsion, and anchorage and reinforcement development.

Commonly used Analysis Procedures identify design actions only at specific locations of a component, typically at sections where maximum design actions are expected. When this is the case, it is necessary to check separately that design strengths are not exceeded at other sections. Figure C6-1 illustrates how this may be done for a beam component of a beam-column moment frame analyzed by the linear procedures of Chapter 3. In Figure C6-1a, the calculated design moments at the component ends do not exceed the design moment strengths. These design beam end moments can be used, along with the known gravity load and beam geometry, to determine design moments and shears at all sections along the component length, which can then be compared with design strengths at all sections. In Figure C6-1b, the calculated design moments at the beam ends exceed the design moment strengths, indicating inelastic response of the component. To determine the internal beam actions corresponding to this loading case, the design end moments are replaced with the design moment strengths (the maximum moments that can be developed at the beam ends). With this information, statics can again be used to construct the internal shear and moment diagrams, which can in turn be compared with design strengths at all sections along the length. For the case shown, the design moment diagram lies within the design strengths, so it is assured that inelastic action occurs by flexure at the beam ends. If the design shear or moment diagram at any section exceeds the design strength at that section, then inelastic action at that section would be identified, and the design actions would have to be adjusted accordingly or the component would have to be rehabilitated to prevent inelastic action.

Inelastic response along the length of a component is most likely if there are changes in design strength along the length or if gravity load effects are relatively large. Figure C6-2 illustrates these for a beam. Because of either large gravity loading or long beam span, the maximum positive design moment occurs away from the beam end. Coupled with reductions in longitudinal reinforcement, positive plastic moment flexural hinging along the span is likely under the design earthquake plus gravity loading.

C6.4.1.2 Stiffness

Stiffness of a reinforced concrete component depends on material properties (including current condition), component dimensions, reinforcement quantities, boundary conditions, and stress levels. Each of these aspects should be considered and verified when defining effective stiffnesses.

Reinforced concrete texts and design codes prescribe precise procedures for stiffness calculation. Most of these procedures were developed from tests of simply-supported reinforced concrete flexural members, loaded to relatively low stress levels. The results often have little relation to effective stiffness of a reinforced concrete component that is interconnected with other components, and subjected to high levels of lateral load. Actual boundary conditions and stress levels may result in significantly different effective stiffnesses. Experience in component testing suggests that the variations in stiffness from one component to another are largely indeterminate. The engineer carrying out an evaluation of an existing building needs to be aware that

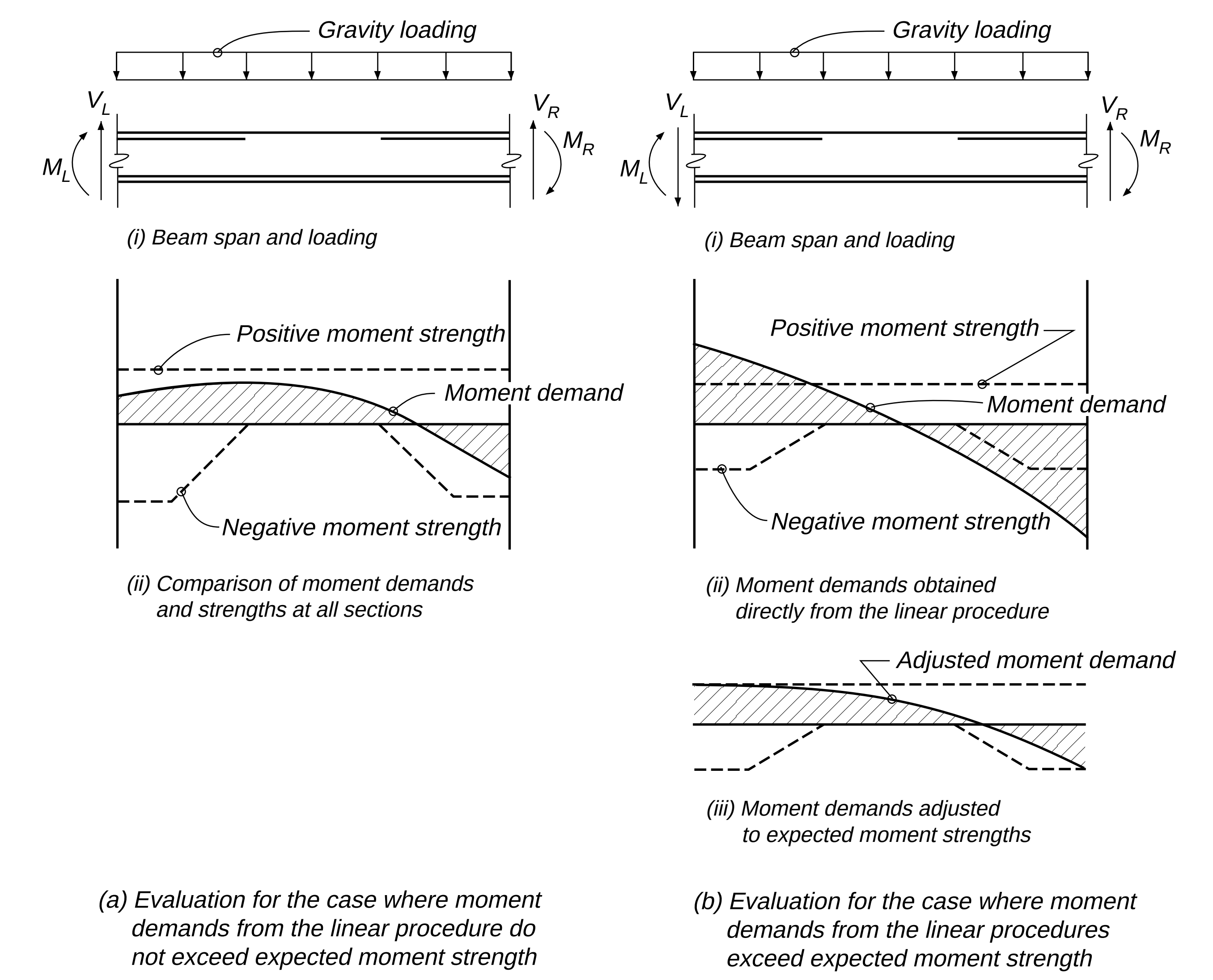


Figure C6-1 Evaluation of Beam Moment Demands of All Sections Along Span

a range of stiffnesses is possible for any set of nominal conditions, and that variations within the range may have a considerable impact on the final assessment.

The typical sources of flexibility for a relatively squat reinforced concrete cantilever wall are illustrated in Figure C6-3. These include flexure, shear, and reinforcement slip from adjacent connections (e.g., foundations, beam-column joints, walls). Flexure tends to dominate for relatively slender components (*h/l* exceeding about five). Shear and reinforcement slip tend to dominate for relatively lower aspect ratios. Whereas flexure and shear rigidities can be estimated acceptably with available mechanics procedures, the effects of reinforcement slip—which can be appreciable or even dominant—cannot be predicted accurately. For columns and shear walls subjected to appreciable axial stress variations under earthquake loading, it is important to also model axial flexibility.

A. Linear Procedures

The linear procedures of Chapter 3 were developed under the assumption that the stiffness of the analysis model approximates the stiffness of the building as it oscillates at displacement amplitudes near an effective yield condition. While this is an imprecise definition, it is clear that the target stiffness in many cases will be considerably less than the gross-section stiffness

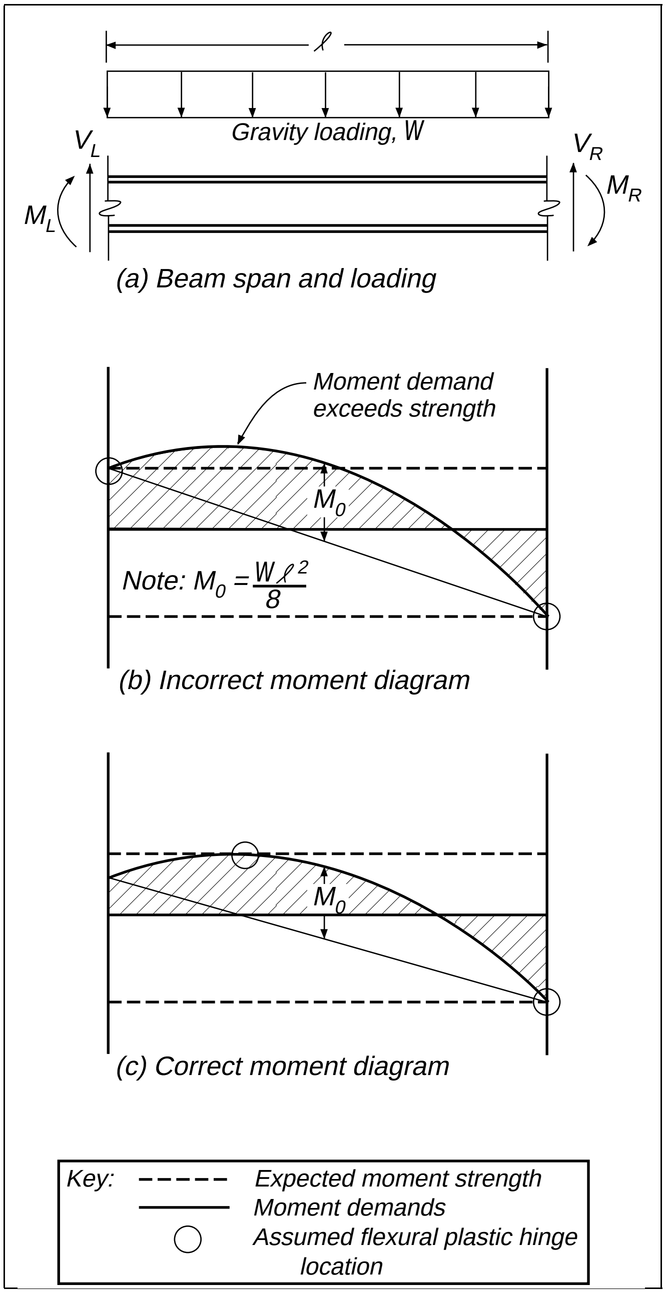


Figure C6-2 Determination of Correct Locations of Beam Flexural Plastic Hinges

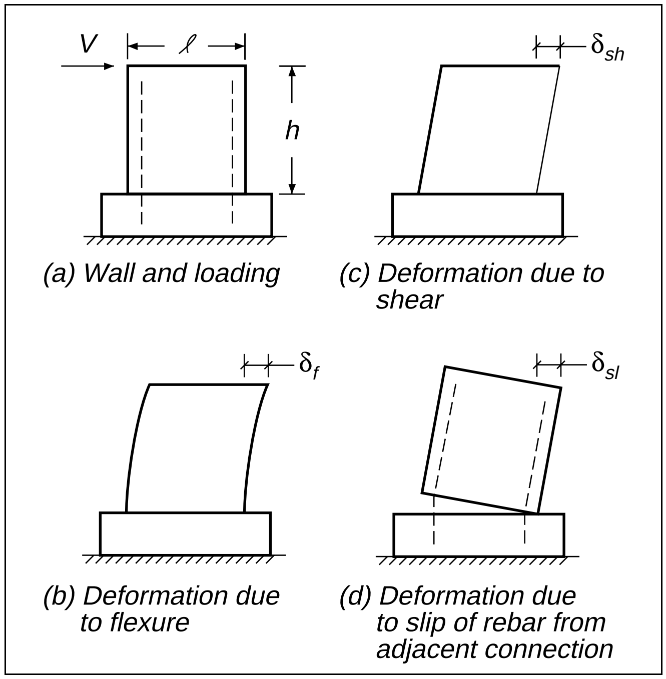


Figure C6-3 Sources of Flexibility in a Wall

commonly used in conventional design practice. The target stiffness for a given component will depend somewhat on the sources of deformation and the anticipated stress levels, as suggested by the following.

- For a **flexure dominated component**, effective stiffness can be calculated considering well-developed flexural cracking, minimal shear cracking, and partial slip of reinforcement from adjacent joints and foundation elements. Flexural stiffness can be calculated according to conventional procedures that take into consideration the variation of flexural moment and cracking along the component length. Shear stiffness may be approximated based on the gross section. Reinforcement slip (which may as much as double the overall flexibility) can be calculated by assuming appropriate stress-slip relations. Where stress levels under design load combinations are certain to be less than levels corresponding to significant cracking, uncracked flexural stiffness may be appropriate.

- For a **shear dominated component**, the onset of shear cracking commonly results in a dramatic reduction in effective stiffness, and may be considered to represent the end of elastic behavior for the component. Therefore, for shear-dominated components the effective stiffness may be based on the gross-section properties, considering flexure and shear. Stiffness reduction to account for reinforcement slip from foundation elements may be appropriate.

- For an **axial dominated component**, the appropriate stiffness depends on whether the axial load is tensile or compressive under the design load combinations.

Where it is compressive, the stiffness can be derived from the gross-section or uncracked transformed-section properties. Where it is tensile, and of sufficient magnitude to result in cracking, stiffness based on the reinforcement only should be used.

In most cases it will be impractical to calculate effective stiffnesses directly from principles of basic mechanics. Instead, the effective stiffness for the linear procedures of Chapter 3 may be based on the approximate values of Table 6-4.

Some of the stiffness values given in Table 6-4 vary with the level of axial load, where axial load is a force-controlled action including gravity and earthquake loading effects calculated according to the procedures specified in Chapter 3. In statically indeterminate structures, the calculated actions will depend on the assumed stiffness, and in certain cases it will not be possible to identify a stiffness from Table 6-4 that results in an action that is consistent with the assumed stiffness. For example, a column may be assumed to be in compression, resulting in a flexural stiffness of $0.7E_cI_g$; the analysis with this stiffness produces column tension. On the other hand, if the same column is assumed to be in tension, resulting in a flexural stiffness of $0.5E_cI_g$, the analysis indicates that the column is in compression. For this column, it is acceptable to assume an intermediate stiffness of $0.6E_cI_g$.

B. Nonlinear Procedures

The nonlinear procedures of Chapter 3 require definition of nonlinear load-deformation relations. For the NSP it is usually sufficient to define a load-deformation relation that describes behavior under monotonically increasing lateral deformation. For the NDP it is also necessary to define load-deformation rules for multiple reversed deformation cycles.

Figure C6-4 illustrates load-deformation relations that may be appropriate to the NSP of Chapter 3. Figure C6-4a is identical in content to Figure 6-1. The following aspects of these relations are important.

- **Point *A*** corresponds to the unloaded condition. The analysis must recognize that gravity loads may induce initial forces and deformations that should be accounted for in the model. Therefore, lateral loading may commence at a point other than the origin of the load-deformation relation.

- **Point *B*** has resistance equal to the nominal yield strength. Usually, this load is less than the nominal strength defined in Section 6.4.2.

- **The slope from *B* to *C***, ignoring effects of gravity loads acting through lateral displacements, is usually taken as equal to between zero and 10% of the initial slope. Strain hardening, which is observed for most reinforced concrete components, may have an important effect on redistribution of internal forces among adjacent components.

- **The ordinate at *C*** corresponds to the nominal strength defined in Section 6.4.2. In some computer codes used for structural analysis it is not possible to specify directly the value of resistance at point *C*. Rather, it is possible only to define the ordinate at *B* and the slope for loading after *B*. In such cases, results should be checked to ensure that final force levels following strain hardening are consistent with expected resistance for that deformation level. Strain hardening to values considerably in excess of the nominal strength should be avoided.

- **The drop in resistance from *C* to *D*** represents initial failure of the component. It may be associated with phenomena such as fracture of longitudinal reinforcement, spalling of concrete, or sudden shear failure following initial yield.

- **The residual resistance from *D* to *E*** may be non-zero in some cases, and may be effectively zero in others.

- **Point *E*** is a point defining the useful deformation limit. In some cases, initial failure at *C* defines the limiting deformation, in which case *E* is a point having deformation equal to that at *C* and zero resistance. In other cases, deformations beyond *C* will be permitted even though the resistance is greatly reduced or even zero-valued.

Many currently available computer programs can only directly model a simple bilinear load-deformation relation. For this reason it is acceptable for the NSP to represent the load-deformation relation by lines connecting points *A*-*B*-*C* as shown in Figure C6-4(b). Alternatively, it may be possible and desirable to use more detailed load-deformation relations such as the relation illustrated in Figure C6-4(c).

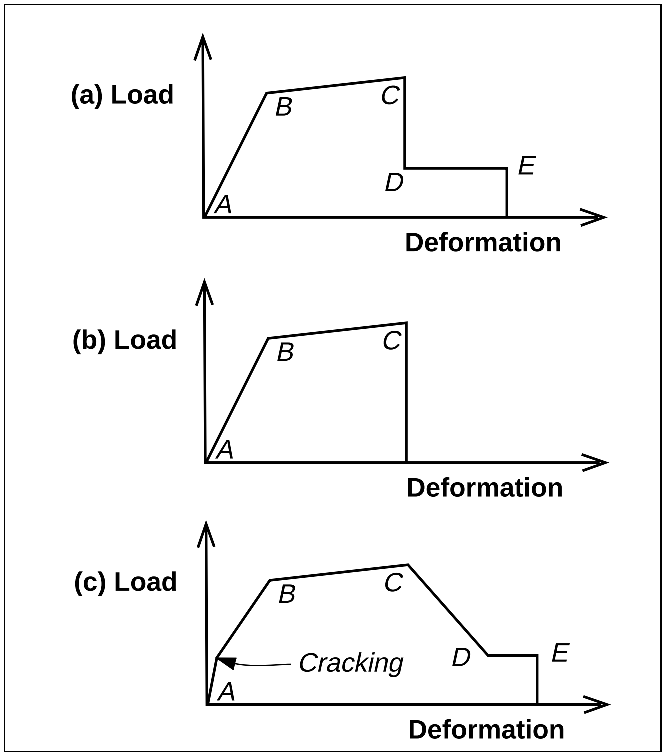


Figure C6-4 Typical Load-Deformation Relations Suitable for Nonlinear Static Procedure

Sections 6.5 through 6.13 present guidelines for specific concrete elements. These sections provide numerical recommendations for defining the nonlinear load-deformation relations.

C6.4.1.3 Flanged Construction

Tests and analysis show that both concrete and reinforcement within the monolithic flange of a beam or wall component act to resist tension and compression forces associated with flexure and axial load on the component (French and Moehle, 1991; Thomsen and Wallace, 1995). The effective flange width specified here is a crude measure of the effectiveness of the flange, to be used with the conventional Bernoulli assumption that plane sections remain plane. Action of the flange in tension—not included in current codes such as *ACI 318-95*—should not be overlooked. In general, the effect of the flange on the component is to increase bending and axial stiffness, increase bending and axial strength, and either increase or decrease flexural deformability depending on whether the flange is in compression or tension. The effects on the structure depend on details of the structure, but could include increased overall stiffness and strength, and modification of the yielding or failure mechanism. Consistent with conventional practice, a flange is considered ineffective in resisting shear out of its plane.

C6.4.2 Design Strengths and Deformabilities

C6.4.2.1 General

Acceptability criteria and strength specifications depend on whether a component has low, moderate, or high ductility demand, and whether the action is considered, according to Chapter 3, to be deformation-controlled or force-controlled.

Strength and deformability of reinforced concrete components are sensitive to details of geometry, reinforcement, materials, and load history including simulated gravity and earthquake loading. For example, flexural deformability is known to decrease with increasing nominal shear stress, all other factors being equal. Experiments must be designed to properly simulate important conditions. Expected variability in test results may sometimes be simulated analytically where suitable analytical models of the physical phenomena are available.

Reinforced concrete component resistance and deformation capacity tend to degrade with an increasing number of cycles and deformation levels. Degradation effects should be accounted for where numerous reversed loading cycles to large deformation levels are expected. These may be expected for structures with short periods and for structures subjected to long-duration ground motions. This effect should be considered primarily for deformability of deformation-controlled actions and for deformability and strength of force-controlled actions. Although strength degradation of deformation-controlled actions may occur, it usually is safer to disregard this degradation. The reason is that the forces in the deformation-controlled actions determine the design forces on the more brittle, force-controlled actions, and upper bound forces should be sought for design.

C6.4.2.2 Deformation-Controlled Actions

Deformation-controlled actions in reinforced concrete construction typically are limited to flexure and to shear in members with low aspect ratio. Flexure generally is the more ductile of the two, and resistance in flexure usually can be determined with greater accuracy. For this reason, deformation-controlled actions preferably will be limited to flexure.

As a flexurally-dominated component is flexed into the inelastic range, the longitudinal reinforcement in tension may be stressed to yield and beyond. The actual yield stress of reinforcing steel typically ranges from the nominal yield value up to about 1.3 times the nominal value, with average values about 1.15 times the nominal value. Tensile strength, which may be approached in components having high ductility demand, is typically 1.5 times the actual yield value. Therefore, the minimum recommended tensile stress of 1.25 times the nominal yield strength should be considered a low estimate suitable only for components with low and intermediate ductility demands.

C6.4.2.3 Force-Controlled Actions

In general, strengths Q_{CL} should be determined as realistically low estimates of component resistance over the range of deformations and coexisting actions to which the component is likely to be subjected. Where strengths are calculated, use low estimates of material strengths; however, assumed material strengths should be consistent with quantities assumed for deformation-controlled actions in cases where the same materials affect both strengths. For example, consider a reinforced concrete beam where flexural moment is the deformation-controlled action, and shear is the force-controlled action. In this case, beam flexural strength and beam shear strength are affected by concrete and reinforcement properties. It would be reasonable to calculate flexural strength assuming estimated concrete strength, and reinforcement stress equal to 1.25 times the nominal value. Shear strength would be calculated using the same assumed concrete strength and the same assumed nominal yield stress for the reinforcement, but without strain hardening. It would be unreasonable to assume a high compressive strength for flexure and a low compressive strength for shear, because the same concrete resists both actions.

C6.4.2.4 Component Ductility Demand Classification

Deformation ductility may be taken as displacement ductility, although it is conservative to use rotation or curvature ductility instead.

C6.4.3 Flexure and Axial Loads

Flexural strength calculation follows standard procedures, except that in contrast with some procedures, the developed longitudinal reinforcement in the effective flange width is to be included as tensile reinforcement. In existing construction, the longitudinal reinforcement may not be adequately developed at all sections. Where development length measured from a section is less than the length required to develop the yield stress, the stress used for strength calculation shall be reduced in proportion with the available length. Furthermore, the flexural deformability shall be based on the assumption of development failure, rather than flexural failure.

Flexural strength and deformation capacity of columns need to be calculated considering the axial forces likely to be coexisting with the design flexural demands. Except for conforming columns supporting discontinuous walls, where the column is in compression the flexural moment is a deformation-controlled action and the axial load is a force-controlled action. Where practicable, the column axial load should be determined by limit analysis or nonlinear analysis, as described in Chapter 3. The column flexural moment strength and corresponding acceptance criteria are then determined for this axial load. Where lateral loading in different directions results in different design axial loads, flexural strength and acceptability should be checked for both extremes and for critical cases in between. Special attention is required for corner columns, which may experience very high axial tension or compression for lateral loading along a diagonal of the building.

ACI 318-95 limits the maximum concrete compression strain for flexural calculations to 0.003. The same limit is permitted in the *Guidelines*. However, larger strains at the onset of concrete spalling are commonly achievable for components with significant strain gradients and components framing into adjacent blocks of concrete (for example, a column framing into a footing). The upper limit of 0.005 for unconfined sections is based on observed performance of components in laboratory tests. Larger calculated deformation capacities will result using this limit.

The compression strain limit of 0.005 for unconfined concrete is based on judgment gained through laboratory testing experience. When a component has a moment gradient, or when it frames into an adjacent component, the concrete is confined by adjacent concrete so that larger compression strains can be developed. The value 0.003 specified in the *ACI 318-95* Building Code is a lower-bound value that is intended to give a conservative estimate of strength for design of new construction. Larger values are used in some other codes for design of new structures.

The *Guidelines* permit the engineer to take advantage of the beneficial effects of concrete confinement provided by properly detailed transverse reinforcement (Sheikh, 1982). Appropriate details include close longitudinal spacing, cross-ties or intermediate hoops for wide sections, and anchorage into the confined core (or other appropriate means of anchoring the transverse steel). The analytical model for confined concrete should be consistent with the materials and details. The maximum usable compression strain of confined concrete may correspond to loss of component resistance due to either degradation of the confined concrete, fracture of transverse reinforcement, or buckling and subsequent fracture of longitudinal reinforcement. Buckling and subsequent fracture of longitudinal reinforcement appear to depend on both the maximum tensile strain and the maximum compressive strain experienced by the longitudinal reinforcement. At the time of this writing, accurate models for predicting this type of failure are not available. The recommended strain limits of 0.05 (tension) and 0.02 (compression) are based on observed performance of reinforced concrete components in laboratory tests, and are associated primarily with the phenomenon of reinforcement buckling and subsequent fracture.

Laboratory tests indicate that flexural deformability may be reduced as the coexisting shear force increases. At the time of this writing, analytical methods for considering effects of applied shear on flexural deformability are not well developed. The engineer should exercise caution when extrapolating results for low applied shear force to cases with high applied shear force.

C6.4.4 Shear and Torsion

Strength in shear and torsion has been observed to degrade with increasing number and magnitude of deformation cycles. Relations between shear strength and deformation demand have been proposed based on test results (Priestley et al., 1994; Aschheim and Moehle, 1992), but these are valid only within the loading regime used during the tests. The sequence and magnitude of inelastic deformations that will occur in a given building during an unknown earthquake cannot be predicted. Therefore, shear strength cannot be predicted accurately. The *Guidelines* therefore prescribe a simple procedure whereby for low ductility demands the strength is assumed to be equal to the strength for a nonyielding structure, and for other cases the strength is assumed to be equal to the strength expected for structures experiencing large ductility demand. For yielding components, it is permitted to calculate the shear strength outside flexural plastic hinges, assuming values for low ductility demand. For this purpose, the flexural plastic hinge length should be taken as equal to the section depth in the direction of applied shear.

To be effective in resisting shear, transverse reinforcement must be properly detailed and proportioned. The *Guidelines* specify minimum requirements.

The recommendation for shear friction strength is based on research results reported in Bass et al. (1989). The reduced friction coefficient for overhead work is because of poorer quality of the interface at this joint.

Additional information on shear strength and deformability is presented in the sections on concrete elements.

C6.4.5 Development and Splices of Reinforcement

Development of straight and hooked bars, and strength of lap splices, are a function of ductility demand and number of yielding cycles. General trends are similar to those described for shear in Section C6.4.4. For this reason, the specifications for development and lap splices are organized according to ductility demand.

For bars that are not fully developed according to the specifications of *ACI 318-95*, the bar stress capacity for strength calculations can be calculated as a linear function of the provided development or splice length. Where a bar has less than the development or splice length required for yield at a given section, and the calculated stress demand equals or exceeds the available capacity, development or splice failure should be assumed to govern. Splice failure should be modeled as a rapid loss in bar stress capacity.

The embedment length used in Equation 6-2 was derived from design equations in *ACI 318-95* that relate to pullout of bars having sufficient cover or transverse reinforcement, so that splitting of cover concrete cannot occur. The expression may be applied to bottom beam reinforcement embedded a short distance into a beam-column joint. For an embedment of six inches into a joint, which is common for frames designed for gravity loads only, Equation 6-2 typically produces values of f_s = 20 ksi or lower. Experimental research on beam-column connections (Moehle et al., 1994) indicates higher stress capacities may be available when flexural tension stresses in adjacent column bar reinforcement (which acts to clamp the embedded bar) are low. The available data support use of Figure C6-5 to estimate the stress capacity of the embedded bars. In Figure C6-5, the column longitudinal reinforcement stress is calculated based on column actions coexisting with the embedded bar tensile force.

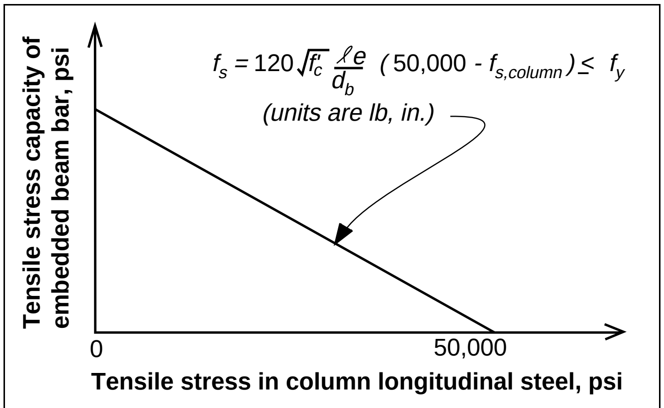


Figure C6-5 Relation Between Beam Embedded Bar Stress Capacity and Coexisting Tensile Stress in Adjacent Column Longitudinal Reinforcement

The specification for doweled bars is based on tests reported in Luke et al. (1985). Other suitable methods of anchoring new concrete to existing concrete are acceptable.

C6.4.6 Connections to Existing Concrete

Many different devices are used for attaching structural and nonstructural items to concrete. The design of anchorages has generally been based on engineering judgment, proprietary test data, manufacturers' data, and code requirements. Anchorage systems can be classified as either cast-in-place systems or post-installed systems.

C6.4.6.1 Cast-in-Place Systems

Anchors of this general classification come in a wide range of types and shapes, and utilize numerous attachment mechanisms. Typical examples are common bolts, hooked J or L bolts, threaded rod, reinforcing steel, threaded inserts, stud welded plates, and embedded structural shapes. The design of these anchoring components must consider the overall behavior of the connected components or elements and must consider the overall behavior of the anchorage. Anchorages are not only subject to shear and tensile forces, but also to bending and prying actions. The ductility and capacity of these connections should exceed the associated ductility of the connecting action as well as the magnitude of the action.

The location of the anchor with respect to potential cracking of the host concrete must be considered in the design. Edge distances, depth of embedment, spacing, and flexural cracking may reduce the capacity of the anchor by a factor of 0.5 or less. Consideration of the service environment is essential to reduce the potential of corrosion-induced failure.

ACI 355.1R-91 contains state-of-the-art information on anchorage to concrete. It is the first of a two-volume project being undertaken by ACI Committee 355; the referenced document emphasizes behavior, while the second volume is to be a design manual. Suggestions for design consideration and construction quality control are provided in the first volume. Designers are strongly encouraged to utilize this document in developing their anchorage designs. While this is not a code-like document, it provides a single point of reference for information needed for appropriate design.

C6.4.6.2 Post-Installed Systems

Anchors of this general classification include grouted anchors, chemical anchors, and expansion anchors. Excluded from consideration are powder-actuated fasteners, light plastic or lead inserts, hammer-driven concrete nails, and screen-driven systems. These are excluded because there is little test data to recommend their use.

The commentary for this section includes the material in Section C6.4.6.1. An additional item to be considered is that anchors of this type generally have little ductility associated with their behavior. They therefore should be designed for the total unreduced demand associated with the connected components.

Test data and design values for various proprietary post-installed systems are available from various sources. Because there commonly is a relatively wide scatter in ultimate strengths, common practice is to define working loads as one-quarter of the average of the ultimate test values. Where working load data are defined in this manner, it may be appropriate to use a design strength equal to twice the tabulated working load. Alternatively, where ultimate values are tabulated, it may be appropriate to use a design strength equal to half the tabulated average ultimate value. The implicit objective of these suggestions for design strengths is to define the design strength as the lower-bound strength of Chapter 3. Accordingly, where statistical data are available the design strength may be taken at the lower five-percentile value.

C6.4.6.3 Quality Control

Connections between seismic resisting components must be subjected to a high level of installation inspection and testing. Many different installation factors can greatly reduce the expected capacities of all connection systems. ACI Report 355.1R-91 provides guidance with respect to this issue. Special care must be taken by the design professional specifying the inspection and testing of anchorage and connection systems.

The design of post-installed systems is susceptible to being altered in the field, due to existing reinforcing steel. Magnetic and radiographic procedures are available to help in locating conflicting reinforcing steel during the design stage, but all conditions and variations are difficult to predetermine. Contingency plans should be made as to how to deal with conflicts in anchor placement. Rebar should rarely be cut and then only under the direction of the engineer of record.

C6.5 Concrete Moment Frames

C6.5.1 Types of Concrete Moment Frames

Properly-proportioned and detailed reinforced concrete frames can provide an efficient system for resisting gravity and lateral loads, while providing maximum flexibility for use of interior spaces. To function properly in resisting earthquake effects, the framing system should provide at least the following:

- **Adequate stiffness.** Stiffness is important in controlling lateral displacements during earthquake response to within acceptable limits. While the *Guidelines* do not impose general limits on lateral drift ratios for all materials of construction, some guidance on target drift levels is provided in Table 2-4. The target drift levels suggested in the table are derived from experience with successful performance of buildings in past earthquakes; significant deviations above these limits should only be accepted after careful consideration. Lateral drift also needs to be limited to avoid pounding with adjacent structures, per Section C6.2. As noted in Section C6.2, pounding of adjacent buildings, especially when floor levels for the pounding buildings do not align, may lead to severe damage to impacted columns, and may cause collapse. Excessive lateral drift may also contribute to second-order P-Δ effects associated with gravity loads acting through lateral displacements. Some additional restrictions on lateral drifts are imposed in Chapter 11, because of the potential for damage to nonstructural components and contents.

- **Proper relative proportions of framing components.** To function properly, it is desirable that inelastic action, if it occurs, be distributed throughout the structure rather than being concentrated in a few components. In reinforced concrete frames, this usually is achieved by providing a stiff, nonyielding spine throughout the building height. This spine can be either a stiff reinforced concrete wall that is continuous through the building height, or the columns themselves if they are sufficiently strong. If the columns are made stronger than the horizontal framing members, yielding will tend to occur primarily in the beams, ideally resulting in a beam sway mechanism in which horizontal framing components yield throughout the building height (Figure C6-6b). On the other hand, if the columns are weaker than the horizontal framing components, yielding will tend to concentrate in a single story, possibly leading to a column sway mechanism (Figure C6-6a). This latter failure mechanism is one of the prominent causes of collapse in reinforced concrete building construction. Attention also must be paid to strength of beam-column connections. In general, it is desirable that connections be made stronger than the adjacent framing components. Beam-column joint failures, especially for exterior and corner

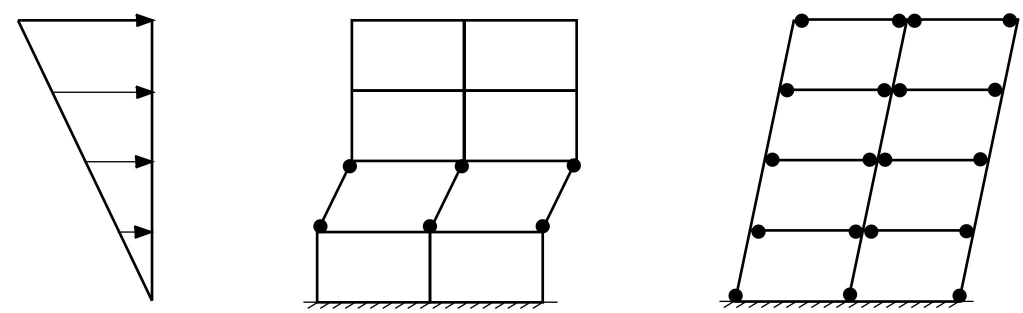


Figure C6-6 Flexural Failure Mechanisms of Reinforced Concrete Frames

connections, have contributed to many building collapses in past earthquakes.

- **Adequate detailing.** Framing components need to be detailed with reinforcement that provides them with adequate toughness. In both columns and horizontal framing components, the longitudinal reinforcement needs to be reasonably continuous and well-anchored, so that flexural tension stresses can be resisted under the full range of flexural moments that will be experienced during a design-level event. Lap splices preferably will be located away from locations of inelastic flexural action, or will be confined by closely spaced, well-detailed transverse reinforcement. Transverse reinforcement spacing and detailing should be adequate to confine wherever compression strains are large (that is, where axial loads are high or where flexural plastic hinges require large rotation capacity). Transverse reinforcement also should be proportioned and detailed to prevent shear failures in columns and beams. Where joints are heavily stressed, joint transverse reinforcement also is an essential element of a tough framing system. The literature abounds with documentation of building collapses associated with failures of inadequately detailed columns and joints. Beam failures do not appear to have been a major cause of building collapse in past earthquakes, but adequate attention to their details is nonetheless important in design.

C6.5.1.1 Reinforced Concrete Beam-Column Moment Frames

Where new frames are added as part of a seismic rehabilitation, it is preferable that they satisfy the requirements for Special Moment Frames, Intermediate Moment Frames, or Ordinary Moment Frames, whichever is appropriate according to definitions and requirements of the *NEHRP Recommended Provisions* (BSSC, 1995). However, because of constraints imposed by existing conditions, it may not be possible to satisfy all requirements for these predefined framing types. Because design requirements have evolved continually, it is unlikely that any existing frame will fully comply with the requirements of modern codes. For example, many older existing frames will satisfy many—but not all—of the provisions required for new ordinary moment frames. For these reasons, the terms “Special Moment Frame,” “Intermediate Moment Frame,” or “Ordinary Moment Frame” are not used broadly in the *Guidelines*.

Some existing bearing wall buildings may rely on wall resistance for loading in the plane of the wall, and on slab-wall framing for loading out of the plane of the wall (the wall acts as a wide column in this loading direction). The slab-wall frame, loaded out of the plane of the wall, may be classified as a beam-column moment frame.

C6.5.1.2 Post-Tensioned Concrete Beam-Column Moment Frames

This classification excludes precast construction that is pretensioned or post-tensioned, which is covered by Section 6.6 of the *Guidelines*.

C6.5.1.3 Slab-Column Moment Frames

In certain parts of the United States, it is common practice to design slab-column frames for gravity loads alone and to assign lateral load resistance to other elements, such as beam-column moment frames and shear walls. Slab-column frames designed according to this practice are included within the scope of Section 6.5, as it may be possible to derive some benefit in lateral load resistance from these frames, and because these frames should be analyzed to ensure that they continue to support gravity loads under the design lateral deformations.

C6.5.2 Reinforced Concrete Beam-Column Moment Frames

C6.5.2.1 General Considerations

The main structural components of beam-column frames are beams, columns, and beam-column connections. The beam may be cast monolithically with a reinforced concrete slab, in which case the slab should be considered to act as a flange of the beam.

Experience in earthquakes demonstrates that frames, being relatively flexible, may be affected negatively by interaction with stiff nonstructural components and elements. The analytical model should represent this interaction.

Provisions for design of new buildings (e.g., ACI 318) are written so that inelastic action ideally is restricted to flexure at predetermined locations. Inelastic action in an existing building may be by flexure at sections other than the component ends, by shear or bond failure, or by some combination of these. The analytical model should be established recognizing these possibilities. Usually it is preferable to establish the likely inelastic response of a component using free-body diagrams of the isolated component rather than relying on the complete structure analysis model for this purpose. This approach is illustrated in Section C6.4.1.1.

The recommendations for eccentric connections are based largely on practical considerations and engineering judgment. Some tests have investigated this condition (Joh et al., 1991; Raffaelle and Wight, 1995).

Some tests on beam-column joints having beams wider than columns have been reported (Gentry and Wight, 1994). These indicate that wide beams can be effectively connected to columns, given certain details.

The restrictions on types of inelastic deformation are based on the observation that lateral load resistance cannot be sustained under repeated loadings for frame members whose strength is controlled by shear, torsion, or bond. Some inelastic response in shear, torsion, or bond may be acceptable in secondary components, which by definition are required only for gravity load resistance.

C6.5.2.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

No commentary is provided for this section.

B. Nonlinear Static Procedure

Available inelastic models for beams include concentrated plastic hinge models, parallel component models, and fiber models (Spacone et al., 1992). With plastic hinge models, inelastic behavior is restricted to those locations where the analyst has placed nodes in the analytical model, typically at beam ends adjacent to the columns. If inelastic response is possible at other locations along the beam span, it is necessary to subdivide the beam into shorter segments having potential plastic hinges located at the end of each segment. Usually a beam can be evaluated separately before assembling the complete structure model to determine if internal plastic hinges are likely (see Section C6.4.1.1).

Reinforced concrete columns can be modeled using the same models identified for beams, except that where there are significant axial force variations under the action of earthquake loading, the model should also represent the effects of that variation on stiffness and strength properties. This is possible using interaction surfaces for plastic hinge models. Fiber models usually can represent this effect directly.

C. Nonlinear Dynamic Procedure

Hysteretic relations used for the NDP should resemble the response obtained for reinforced concrete components. It is preferable that nondegrading bilinear relations not be used. Simple stiffness degrading component models such as the Takeda and Modified Clough relations (Saiidi, 1982) are preferred.
Figure C6-7 is a sample of a load-deformation relation produced by the Takeda model. The model features reduced stiffness beyond yield and stiffness degradation with increasing displacement amplitude. For existing construction with inadequate details, there may be strength degradation in addition to stiffness degradation. Some hysteretic models including stiffness and strength degradation have been reported (Kunnath

et al., 1992). The rate of strength degradation for these models needs to be calibrated with experimental data.

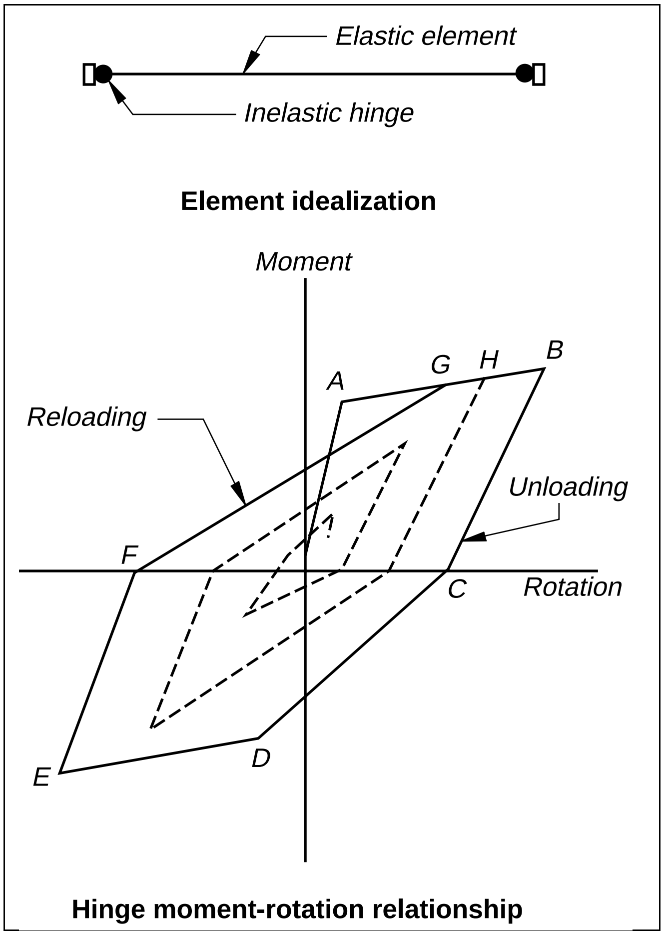


Figure C6-7 Takeda Hysteresis Model

Figure C6-8 presents some typical load-deformation relations measured during laboratory tests of reinforced concrete components. These illustrate a range of performances that might be anticipated. The relations shown should not be construed as being representative of components in existing construction, but should be used only as a guide in selecting general characteristics of hysteretic models.

C6.5.2.3 Design Strengths

As described in Section 6.4.2, component strengths are calculated based on procedures from *ACI 318-95*, with some modification to reflect differences in details and proportions, as well as to reflect the different purposes of the *ACI 318-95* document and the *Guidelines*.

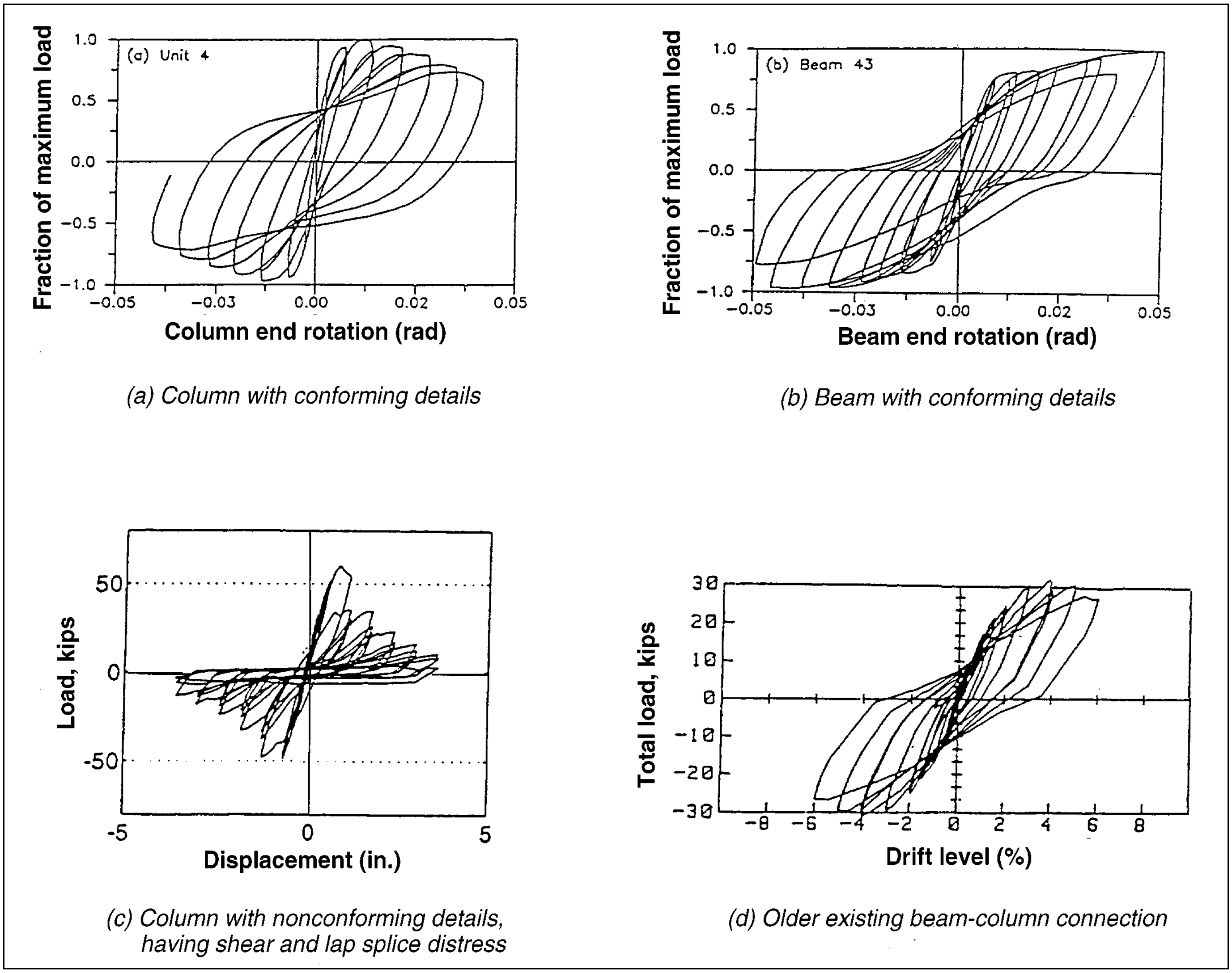
The engineer is reminded that inelastic response and failure may occur in any of a number of different modes, and may occur at any section along the length of the component, including its connections.

Experiments on columns subjected to axial load and reversed cyclic lateral displacements indicate that *ACI 318-95* design strength equations may be excessively conservative for older existing columns, especially those with low ductility demands (Priestley et al., 1994; Aschheim and Moehle, 1992). The recommended column shear strength equation is based on a review of the available test data. The available strength in older columns is strongly related to ductility demand; therefore, conservative procedures should be used to determine whether ductility demands will reach critical levels. The distinction between low ductility demand and moderate or high ductility demand is discussed in Section 6.4.2.4. The restriction on axial loads calculated using the linear procedures of Chapter 3 is based on the understanding that the axial load calculated using linear procedures may overestimate the axial load in a yielding building. The restriction will produce conservative effects. The axial load preferably should be calculated using limit analysis procedures as described in Section 3.4.2.1B. Simple procedures involving summation of the beam plastic shears are appropriate for this purpose.

Shear failure in columns is a common source of damage and collapse in older buildings. Engineering judgment should be applied—in addition to the specifications of the *Guidelines*—to determine the proper course of action for buildings with columns having widely-spaced ties and moderately high shear stresses.

The specification for beam-column joint shear strength is developed from various sources. Kitayama et al. (1991) and Otani (1991) present data indicating that joint shear strength is relatively insensitive to the amount of joint transverse reinforcement, provided there is a minimum amount (a transverse steel ratio equal to about 0.003). Beres et al., (1992a) report on shear strengths of joints without transverse reinforcement. Although some researchers report that increased column axial load results in increased shear strength, the data do not show a significant trend.

Design actions (axial loads and joint shears) on beam-column joints preferably should be calculated from consideration of the probable resistances at the locations for nonlinear action. Procedures for



(a) Column with conforming details

(b) Beam with conforming details

(c) Column with nonconforming details, having shear and lap splice distress

(d) Older existing beam-column connection

Figure C6-8 Sample Load-Deformation Relations for Reinforced Concrete Beams, Columns, and Beam-Column Connections

estimating joint shear are the same as those specified in *ACI 318-95*.

C6.5.2.4 Acceptance Criteria

A. Linear Static and Dynamic Procedures

The basic acceptance criteria of Chapter 3 require that all actions be classified as either displacement-controlled or force-controlled actions. For beam-column moment frames, it is preferred that deformation-controlled actions be limited to flexure in beams, although some flexural yielding in columns (at least at the foundation level) is usually inevitable. This preference lies in the observation that beams yielding in flexure usually have moderate to high ductility capacities. Column flexural yielding is usually less ductile because of the detrimental effects of axial loads on deformability, and because excessive yielding in columns may lead to story sway mechanisms (see Section C6.5.1). Low-ductility capacity response modes—such as shear, torsion, or reinforcement development or splicing of beams or columns, and shear in beam-column joints—are to be avoided in primary components designed using the linear procedures of Chapter 3. Yielding in some low-ductility capacity response modes is permitted in secondary components where gravity loads are likely to be sustained through moderate levels of ductility demand.

Tables 6-10 through 6-12 present allowable values for these secondary component cases.

Ideally, where linear procedures are used for design, the actions obtained directly from the linear analysis will be used only for determining design values associated with yielding actions in the structure. The design actions in the rest of the structure should be determined using limit analysis procedures considering the gravity forces plus the yielding actions acting on a free body diagram of the component or element. The *Guidelines* specify actions that should be designed on this basis.

Reinforced concrete components whose design forces are less than force capacities can be assumed to satisfy all the performance criteria of the *Guidelines*. However, it is still necessary to check performance of all other components and the structure as a whole.

Beam-column frames with widely-spaced column transverse reinforcement may be susceptible to story collapse due to column failure. Column shear failure can initiate the collapse if shear capacity is less than shear strength demand. Flexural failure can initiate the collapse if inelastic column flexural demands lead to strength degradation. Frames having columns with flexural strengths less than the adjoining beam flexural strengths are particularly vulnerable to this latter type of failure. To minimize the likelihood of this type of failure in new construction, codes for new building construction require that column end regions contain copious amounts of transverse reinforcement, and that the sum of strengths of columns exceed the sum of strengths of beams at each joint. With a similar objective, the *Guidelines* specify that DCR values for beams and columns be checked (which is similar to checking relative strengths) and that DCR values be compared with DCR capacities (a conservative measure of $m/2$ is specified). The check is carried out as an average for all components at the floor level being checked, rather than at each connection as specified in *ACI 318-95*. Where an element fails the check, either: (1) the check is repeated for all elements of the system, since story collapse is likely to involve more than one frame; (2) the structure is reanalyzed by one of the nonlinear approaches, which is likely to provide an improved measure of the actual demands; or (3) the structure is rehabilitated to remove the deficiency.

The m values in Tables 6-6, 6-7, and 6-8 were developed from the experience and judgment of the project team, guided by available test data (Aycardi et al., 1994; Beres et al., 1992; Lynn et al., 1994; Pessiki et al., 1990; Qi and Moehle, 1991).

B. Nonlinear Static and Dynamic Procedures

Inelastic response preferably will be limited to flexure in beams and columns. For components whose strength is limited by shear, torsion, and reinforcement development and splicing, the deformability usually is less than for flexure, and stability under repeated deformation cycles is often questionable. Where inelastic action other than flexure is permitted, it is preferable that it be limited to a few components whose contribution to total lateral load resistance is a minority.

Inelastic action is not desirable for actions other than those listed in Tables 6-6, 6-7, and 6-8. Where inelastic response is acceptable, calculated deformations should not exceed the deformation capacities listed in Tables 6-6, 6-7, and 6-8.

C6.5.2.5 Rehabilitation Measures

The rehabilitation strategies and techniques listed in the *Guidelines* are intended to provide guidance on procedures that have been successfully used for seismic rehabilitation of reinforced concrete beam-column moment frames. The list is not intended to exclude alternate procedures that are demonstrated to be effective in satisfying the Rehabilitation Objective. A summary of past research on rehabilitation techniques for reinforced concrete frames is provided by Moehle et al. (1994); Sugano (1981); and Rodriguez and Park (1991).

Commentary on the noted rehabilitation schemes is provided below.

- **Jacketing existing beams, columns, or joints with new steel or reinforced concrete overlays.** Jacketing may serve to increase flexural strength and ductility, and shear strength; to improve longitudinal reinforcement development or splicing; and to combine these effects. Although jacketing can be a technically effective procedure, when several components must be jacketed, it may not be cost-effective, and it can also be very disruptive to building occupants.

  Where jackets are used to increase flexural strength, and in some other cases requiring composite action, appropriate measures should be implemented to provide shear transfer between new and existing materials. These measures may include:

– For concrete jackets, roughening the surface of the existing concrete prior to concrete placement, and using dowels to improve shear transfer strength when the jacket does not surround the component

– For steel jackets, using epoxy to effectively bond the steel to the concrete, and nonshrink grout or dry pack plus bolts or other anchorage devices

Where the objective is to increase component flexural strength, the technique must provide continuity across beam-column connections so that the enhanced strength can be transferred to adjacent framing components (Alcocer and Jirsa, 1993; Corazao and Durrani, 1989; Rodriguez and Park, 1992; Krause and Wight, 1990; Stoppenhagen and Jirsa, 1987). For columns, approaches include the following:

– New longitudinal reinforcement can be passed through the floor system and encased in a reinforced concrete jacket.

– Steel sections flanking the existing column can be connected to it to ensure composite action, and pass through the floor system to provide continuity. Similar approaches may be used for beams, including the addition of straps or continuous reinforcement across joints where beam bottom reinforcement is discontinuous.

Where the objective is to increase flexural ductility, either reinforced concrete or steel jackets can be added to deficient sections (Aboutaha et al., 1994). If the jacket completely surrounds the component or, in the case of beams, the jacket surrounds three faces and is anchored into the slab, only a nominal connection is required between existing and new materials. Concrete jackets should be reinforced with transverse reinforcement and nominal longitudinal reinforcement. Steel jackets may comprise bands or full-height jackets made of steel plates or shells; anchorage may be necessary along the side face of flat steel plates to improve confining action, and stiffeners may be required for thin plates. The space between steel jackets and existing concrete should be filled with nonshrink grout. If the purpose of the jacket is to increase the flexural ductility but not increase the flexural strength, the longitudinal reinforcement in concrete jackets and steel in steel jackets should be discontinued a short distance (about 50 mm) from the connection with adjacent components. Concrete jackets placed to improve ductility may also enhance flexural strength, which may shift the ductility demands to adjacent sections, and this aspect should be checked and appropriate actions taken. In general, a jacket should extend from critical sections a distance equal to at least 1.5 times the cross-sectional dimension measured in the direction of the lateral load.

Where the objective is to increase shear strength, steel, concrete, or other types of jackets can be added to deficient sections (Bett et al., 1988; Katsumata et al., 1988; Aboutaha et al., 1993). The general approach to designing the jacket and its connection with the existing concrete is similar to that described in the preceding paragraph. When proper connections between old and new materials are achieved, it is usually appropriate to calculate the nominal shear strength as if the section were composite.

Where the objective is to improve performance of inadequate reinforcement development or splicing, either reinforced concrete or steel jackets may be used (Aboutaha et al., 1994). The jackets should be designed to restrain splitting action associated with development or splice failure. Concrete jackets require transverse reinforcement and may require cross ties; steel jackets may require bolts anchored into the concrete core.

Where the objective is to improve continuity of beam bottom reinforcement, supplementary reinforcement may be added to improve continuity (Beres et al., 1992b).

- **Post-tensioning existing beams, columns, or joints using external post-tensioned reinforcement.** Post-tensioning may serve to increase flexural strength and shear strength of beams and columns. It may also reduce deficiencies in reinforcement development and splicing if tension stress levels are reduced. Joint shear strength may also be enhanced by joint post-tensioning.

  Usually it is preferable to not bond the post-tensioned reinforcement in regions where inelastic response is anticipated. Bonded reinforcement is more likely to undergo inelastic strain that may relieve the post-tensioning stress. Anchorage zones should also be placed away from inelastic regions because of the potential for anchorage damage in these regions.

- **Modifying of the element by selective material removal from the existing element.** Partial or full-height infills in existing beam-column frames may have inadequate separation between the infill and the concrete frame. In some cases, it is desirable to use the infill as a structural component (see Section 6.7). In other cases, it is desirable to separate the infill from the concrete frame so that lateral resistance is provided by beam-column framing. Either the infill can be entirely removed, or the joint between the infill and the frame can be cleaned and filled with flexible jointing material. In the latter case, the joint dimension should be at least equal to the interstory drift calculated using the Analysis Procedures of Chapter 3.

  Other architectural components that may affect the structural framing include stairs and nonstructural exterior curtain walls. In some cases, gaps can be increased or rigid connections can be replaced with flexible connections to reduce the interaction with the structural framing.

  Beams and columns can also be selectively weakened to improve structural performance. For example, beam longitudinal reinforcement or section depth can be reduced to weaken the beam, thereby promoting development of a strong column-weak beam framing action. Beam and column longitudinal reinforcement can also be severed to decrease shear demands associated with flexural hinging of these components. Weakening of existing structural components is often considered unacceptable, even if this action promotes improved overall behavior of the building. When considering weakening of a structural component, the impact on safety and serviceability under design load combinations—including gravity load, and gravity load plus design lateral loads—should be evaluated.

- **Improving deficient existing reinforcement details.** This approach does not include jacketing, which is covered elsewhere. As with jacketing, this approach may not be cost-effective, and may be overly intrusive.

  This approach may be effective where reinforcement lap splices or anchorages are inadequate. The approach in this case is to remove cover concrete, lap weld existing reinforcement together or weld auxiliary reinforcement between adjacent inadequately developed bars, and replace concrete cover.

  This approach has also been used to add transverse reinforcement to confine inadequately confined lap splices, but tests have shown that this technique may be ineffective. Transverse reinforcement can be added effectively to improve shear strength.

- **Changing the building system to reduce the demands on the existing element.** This approach involves reducing the displacement demands on the existing element by adding new vertical elements (such as moment frames, braced frames, or walls), by adding seismic isolation or supplemental damping, or by otherwise modifying the building. Approaches to changing the building system to reduce seismic demands are discussed in Chapter 2.

- **Changing the frame element to a shear wall, infilled frame, or braced frame element by addition of new material.** This approach usually involves filling openings with reinforced concrete (Altin et al., 1992) or adding steel bracing components to convert the existing moment frame to a shear wall or braced frame (Bush et al., 1991; Goel and Lee, 1990). Where wall openings are filled with concrete, two approaches have been considered. In the first, the entire opening is filled, converting the panel to a structural wall. In the second, a portion of the opening on each side of the existing column is filled to transform the existing column to a wall pier (the added portions of concrete are commonly referred to as wing walls—see Bush et al., 1990). Decisions about how to modify frames, and which ones to modify, depend partly on technical issues and partly on nonstructural considerations.

Where openings in frames are filled with reinforced concrete, at least the following aspects should be considered:

– The wall panel should be designed according to requirements for new wall construction. Wall panel reinforcement should be doweled into existing beam and column sections, to transfer tensile forces from wall reinforcement and to provide shear transfer between new and old concrete.

– Wall boundary reinforcement should be provided where necessary (Jirsa and Kreger, 1989). Where the infill fills the entire opening and the wall panel is adequately connected to the columns, the columns may act as boundary elements. The adequacy of column transverse reinforcement and longitudinal reinforcement (including lap splices) to transfer required forces and sustain required deformations should be checked. Columns may be jacketed to improve their adequacy. Additional wall vertical reinforcement (distributed or concentrated near the boundaries) can be provided. Usually the additional reinforcement can pass through the floor system adjacent to the beam webs.

– If some of the openings in the frame are not filled, the effect of the new wall panel on the existing unfilled portions should be checked.

– The floor diaphragm, struts, and collectors are to be checked to ensure that there is an adequate system to transfer lateral forces to the new wall element. They may be enhanced if necessary.

– The foundation is to be checked to be certain it is capable of resisting both the extra weight of the new material and the increased overturning and shearing actions beneath the rehabilitated element.

• Where steel bracing is provided in existing concrete moment frames, at least the following aspects should be considered:

– The bracing components should be designed according to accepted practices for steel bracing.

– Steel braces should be connected to the existing concrete frame to transfer the design forces. The attachment details should be designed to minimize the impact on the existing concrete materials.

– Adequacy of the existing concrete frame components (beams and columns) to transfer actions developed in the rehabilitated element should be evaluated. Adequacy of column transverse reinforcement and longitudinal reinforcement (including lap splices) to transfer required forces and sustain required deformations should be checked. Columns may be jacketed to improve adequacy. Steel strapping to supplement capacity is permitted.

– The effects of the new bracing system on the existing frame, including portions not provided with braces, should be checked.

– Collectors and floor diaphragms are to be checked to ensure that they are capable of transferring lateral forces to the new braced frame element. They may be enhanced if necessary.

– The foundation is to be checked to be certain it is capable of resisting both the extra weight of the new material and the increased overturning and shearing actions of the rehabilitated element.

Post-tensioning steel can also be considered for lateral bracing of deficient buildings (Miranda and Bertero, 1991; Pinchiera and Jirsa, 1992).

C6.5.3 Post-Tensioned Concrete Beam-Column Moment Frames

C6.5.3.1 General Considerations

The limiting conditions presented in Section 6.5.3.1 are the same as those described in the *NEHRP Recommended Provisions* (BSSC, 1995) for new buildings with prestressed and nonprestressed reinforcement. As documented by Ishizuka and Hawkins (1987), if these conditions are met in new buildings the seismic design provisions for nonprestressed moment frames apply. The recommendation of the *Guidelines* is to extend this finding to existing construction. Satisfactory seismic performance can be obtained in frames using prestressing amounts greater than those listed in

Section 6.5.3.1, but reductions in allowable *m* values or inelastic deformation values may be required. Relevant discussion may be found in Park and Thompson (1977) and Thompson and Park (1980).

BSSC (1995) recommends for new buildings that anchorages for tendons be capable of withstanding, without failure, a minimum of 50 cycles of loading ranging between 40 and 85% of the minimum specified tensile strength of the tendon. It also recommends that tendons extend through exterior joints and be anchored at the exterior face or beyond. These recommendations apply also to the *Guidelines*.

C6.5.3.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

No commentary is provided for this section.

B. Nonlinear Static Procedure

It is assumed that a prestressed concrete beam behaves in a manner equivalent to a nonprestressed beam when conditions (1), (2), and (3) of Section 6.5.3.1 are satisfied. When these conditions are not satisfied, behavior parameters are to be derived from experiments or rational analysis.

C. Nonlinear Dynamic Procedure

Prestressing may result in component hysteresis that is markedly different from that for nonprestressed reinforced concrete components. Figure C6-9 presents some examples. The analytical model should represent the relevant characteristics of the load-deformation response.

C6.5.3.3 Design Strengths

A yielding prestressed concrete flexural member will develop strength associated with force levels developed in prestressed and nonprestressed reinforcement. Yielding of prestressed reinforcement may result in loss of prestress upon load reversal. The effects of this loss on the strength of force-controlled actions should be considered.

C6.5.3.4 Acceptance Criteria

No commentary is provided for this section.

C6.5.3.5 Rehabilitation Measures

The general rehabilitation procedures of Section 6.5.2.5 apply to prestressed concrete frames. Where seismic rehabilitation involves modification of the existing prestressed frame, including attachment of new materials, care should be taken to avoid damage to existing prestressing tendons and anchorages.

C6.5.4 Slab-Column Moment Frames

C6.5.4.1 General Considerations

The main structural components of slab-column frames are slabs, columns, slab-column joints, and the slab-column connection. In most cases, slab-column joints are not critical; therefore, no further discussion on slab-column joints is included in Section 6.5.4. Relevant material on beam-column joints should be referred to for special cases where slab-column joints may have high shear stresses. The slab-column connection commonly is a critical component in the system. It comprises the region of slab immediately adjacent to the column. Shear failure of the slab associated with shear and moment transfer can result in progressive collapse in cases where slab bottom reinforcement (or post-tensional strand) is not continuous through the column (see the report *ACI 352* [ACI, 1988] for further information). Beams are common around the perimeter of buildings that otherwise have predominantly slab-column framing. This case is covered in Section 6.5.4.

As with beam-column frames, experience indicates that slab-column frames may be affected negatively by interaction with nonstructural components and elements. The analytical model should represent this interaction.

Provisions for design of new buildings (e.g., *ACI 318-95*) are written so that inelastic action is restricted, ideally, to flexure at predetermined locations. Inelastic action in an existing building may be by flexure at sections other than the component ends, by shear or bond failure, or by some combination of these. The analytical model should be established recognizing these possibilities. Usually it is preferable to establish the likely inelastic response of a component using free-body diagrams of the isolated component rather than relying on the complete structure analysis model for this purpose. This approach is illustrated in Section C6.4.1.1.

Analytical models for slab-column frames usually are one of three types, illustrated in Figure C6-10. The effective beam width model (Figure C6-10b) represents the slab as a flexural member having stiffness reduced to represent the indirect framing between slab and

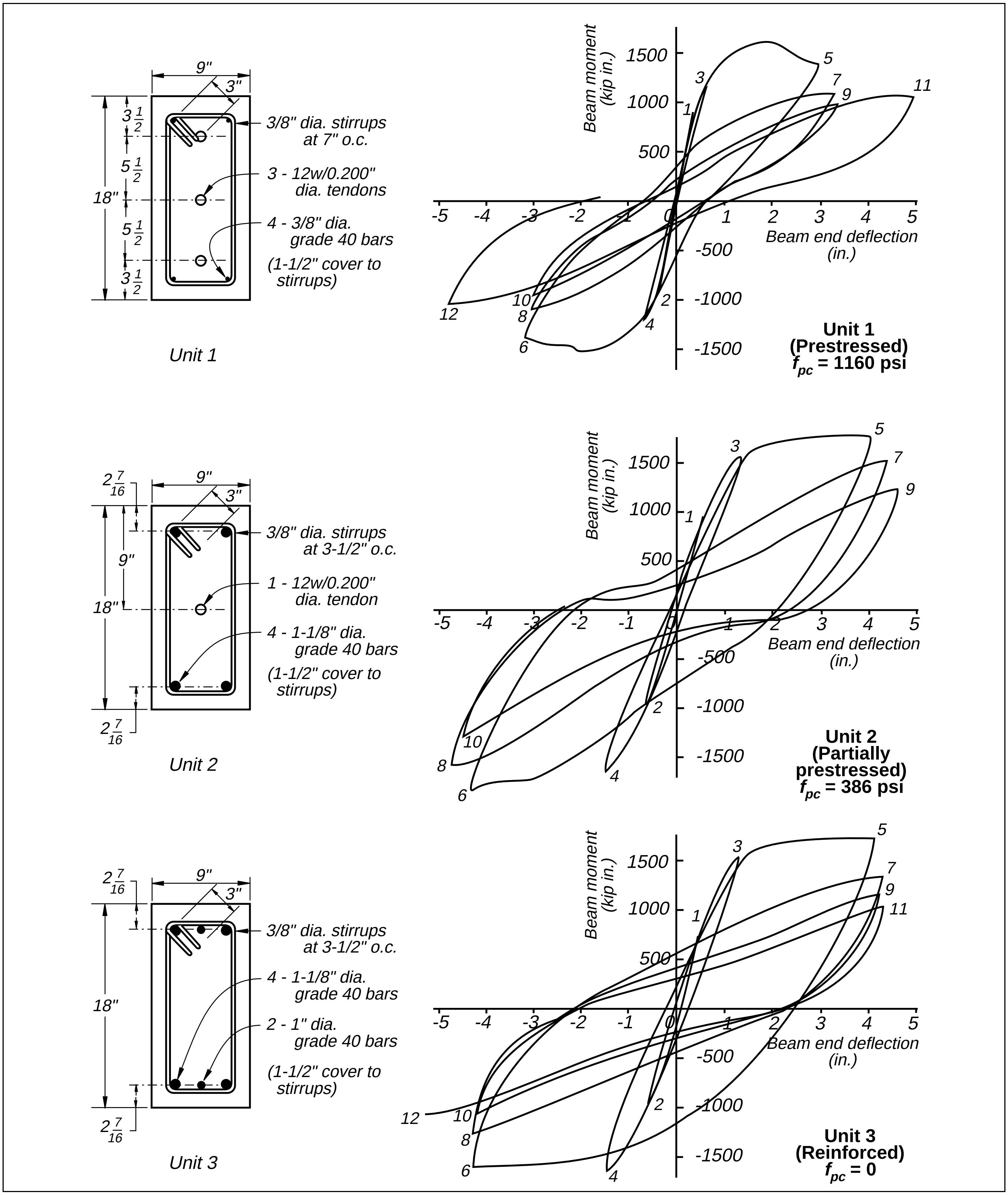


Figure C6-9 Sample Load-Deformation Relations for Prestressed, Partially-Prestressed, and Reinforced Beams column, as well as slab cracking. The equivalent frame model (Figure C6-10c) represents the slab by a flexural member that connects to the column through a transverse torsional member. Direct finite element models (Figure C6-10d) represent the flexural, shear, and torsional response of the slab directly. For each of the three models, the stiffness should be adjusted from theoretical values based on the gross cross section because of the significant effects of slab cracking on response (Vanderbilt and Corley, 1983). The effective beam width model, while simple to use, has a drawback in that there is no component to monitor directly the shear and moment transfer between slab and column, and this is an important aspect in checking performance. The finite element model has certain advantages, but has a relatively high computational cost. In most cases, it may be preferable to use an equivalent frame model because it provides a component to directly monitor shear and moment transfer.

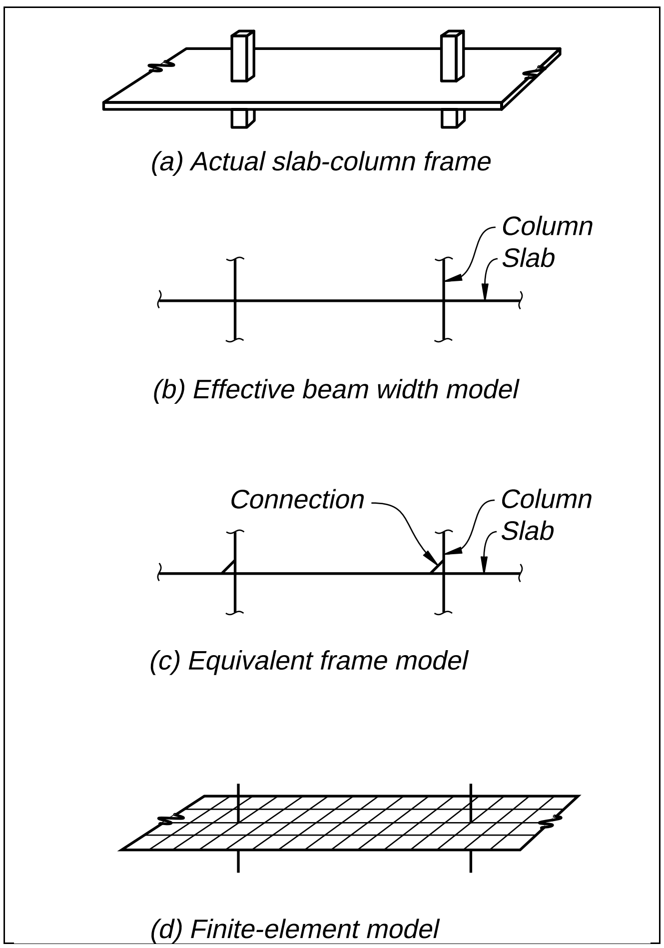


Figure C6-10 Models for Slab-Column Framing

The restriction on types of inelastic deformation are based on the observation that lateral load resistance cannot be sustained under repeated loadings for frame members whose strength is controlled by shear, torsion, or bond. Some inelastic response in shear, torsion, or bond may be acceptable in secondary components, which by definition are required only for gravity load resistance.

C6.5.4.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

Any of the three models depicted in Figure C6-10, and other validated models, may be used to represent the slab-column frame. Whatever the model, it is essential to take into account the reduction in framing stiffness associated with cracking of the slab near the column. This cracking can reduce the effective linear elastic stiffness to as little as one-third the uncracked value (Vanderbilt and Corley, 1983; Pan and Moehle, 1992; Hwang and Moehle, 1993). Further discussion follows.

Various approaches to representing effects of cracking on stiffness of reinforced concrete slabs have been proposed and verified. Vanderbilt and Corley (1983) recommend modeling the slab-column frame using an equivalent frame model (Figure C6-10c) in which the slab flexural stiffness is modeled as one-third of the gross-section value. Hwang and Moehle (1993) recommend an effective beam width model (Figure C6-10b) having an effective width for interior framing lines equal to $\beta(5c_1 + 0.25l_1)$, where β represents cracking effects and ranges typically from one-third to one-half, c_1 = column dimension in the direction of framing, and l_1 = center-to-center span in the direction of framing. For exterior frame lines, half this width should be used. Note that this effective width applies only where the analysis model represents the slab-column joints as having zero horizontal dimension. Alternate approaches may be used where verified by tests.

For prestressed slabs, less cracking is likely, so it is acceptable to model the framing using the equivalent frame model without the factor one-third, or the effective width model with $\beta = 1.0$.

B. Nonlinear Static Procedure

It is essential that the nonlinear analysis model represent the behavior of the slab-column connection in addition to the slab and column components. Nonlinear response of slab-column connections is a complex

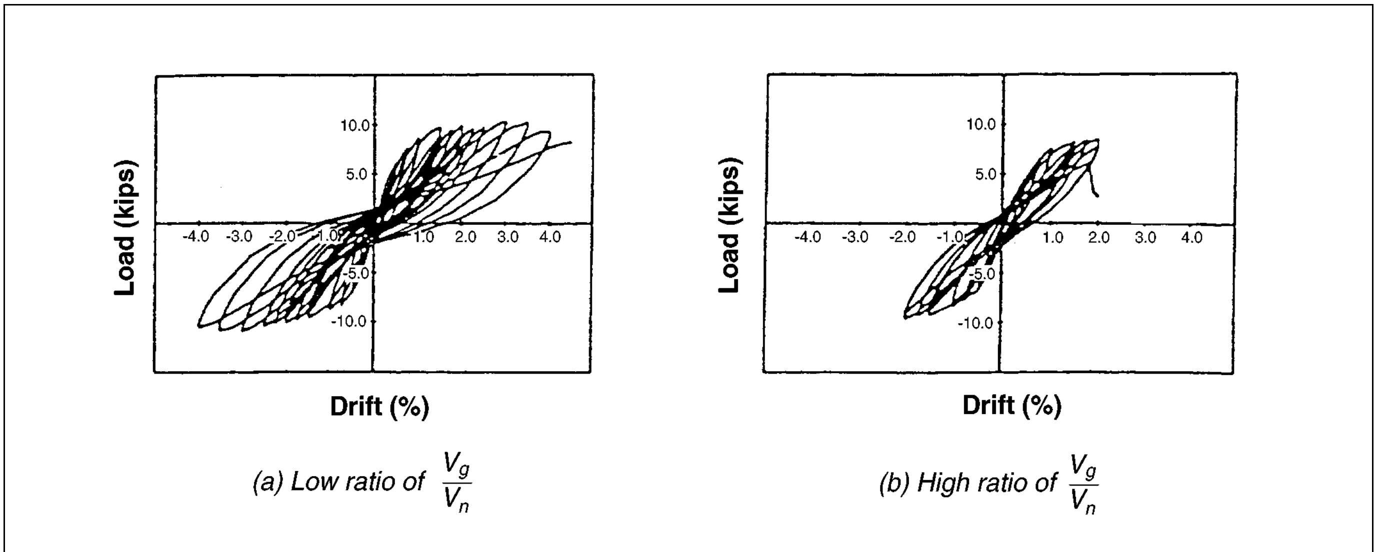


Figure C6-11 Sample Load-Deformation Relations for Reinforced Concrete Slab-Column Connections

function of flexural, shear, torsion, and bond actions. Some detailed models have been reported in the literature (Hawkins, 1980; Luo et al., 1994). A simplified approach, described here, is to model the slab-column frame using the equivalent frame of Figure C6-10c. The column is modeled as described in Section C6.5.2.2B. The slab is modeled according to the general procedures of Section C6.5.2.2B, with initial stiffness according to Section C6.5.4.2A and plastic hinge rotation capacity according to Table 6-14. The connection element between slab and column is modeled as an elasto-plastic component (moderate strain hardening is acceptable) with ultimate rotation capacity according to Table 6-14.

C. Nonlinear Dynamic Procedure

See Section C6.5.2.2C.

Figure C6-11 presents some typical load-deformation relations measured during laboratory tests of slab-column connections where the column did not yield. These illustrate a range of performances that might be anticipated.

C6.5.4.3 Design Strengths

See Section C6.5.2.3 for general discussion on strength of moment frames.

Current technology does not provide accurate strength estimates for slab-column frames. This can be a critical shortcoming, as less-ductile failure modes may in fact predominate even though calculations indicate otherwise. The design of critical structures should take this additional uncertainty into account.

Flexural action of a slab connecting to a column is nonuniform, as illustrated in Figure C6-12. Portions of the slab nearest the column yield first, followed by gradual spread of yielding as deformations increase. The actual flexural strength developed in the slab will depend on the degree to which lateral spread of yielding can occur. The recommendation to limit effective width to the column strip is the same as the design requirement of ACI 318, and represents a lower bound to expected flexural strength. In some cases the full width of the slab will yield. If a greater portion of slab yields than is assumed, the demand on the slab-column connection and the columns will be increased. Nonductile failure modes can result. Shear and moment

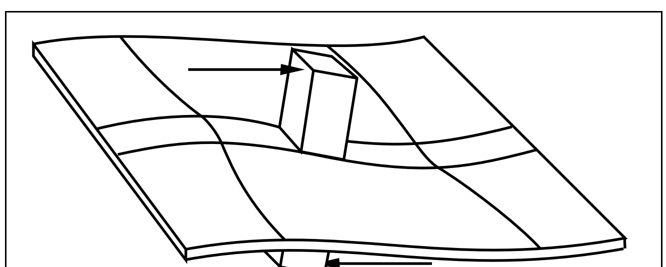


Figure C6-12 Slab Distortion in Flat-Plate Connection under Lateral Load transfer strength of interior slab-column connections may be calculated using any models that are verified by experimental evidence (Hwang and Moehle, 1993; Hawkins, 1980). It is permissible to use a simplified approached that follows the concepts of *ACI 318-95* (ACI, 1995). According to this approach, connection design strength is the minimum of two calculated strengths. One is the strength corresponding to developing a nominal shear stress capacity on a slab critical section surrounding the column (Figure C6-13). All definitions are according to *ACI 318-95*. In applying this procedure, tests indicate that biaxial moment transfer need not be considered (Pan and Moehle, 1992; Martinez-Cruzado et al., 1991). The second strength corresponds to developing flexural capacity of an effective slab width. The effective width is modified from *ACI 318-95* based on results reported by Hwang and Moehle (1993). Both top and bottom reinforcement are included in the calculated strength.

Shear and moment transfer strength for exterior connections without beams is calculated using the same procedure as specified in *ACI 318-95*. Where spandrel beams exist, the strength should be modified to account for the torsional stiffness and strength of the spandrel beam.

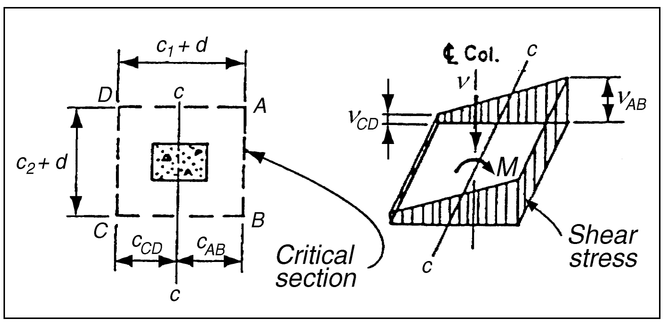


Figure C6-13 Eccentric Shear Stress Model

C6.5.4.4 Acceptance Criteria

A. Linear Static and Dynamic Procedures

For slab-column moment frames, it is preferred that deformation-controlled actions be limited to flexure in slabs, although it may be necessary and acceptable to permit inelastic action in columns and slab-column connections. This preference is partially explained in Section C6.5.2.4. Inelastic response of slab-column connections can be ductile if the level of vertical shear carried from the slab to the column is relatively low (Pan and Moehle, 1989).

Ideally, where the linear procedures of Chapter 3 are used for design, the actions obtained directly from the linear analysis will be used only to determine design values associated with yielding actions in the structure. The design actions in the rest of the structure should be determined using limit analysis procedures considering the gravity forces plus the yielding actions acting on a free body diagram of the component or element. The *Guidelines* specify actions that should be designed on this basis.

Reinforced concrete components whose design forces are less than force capacities can be assumed to satisfy all the performance criteria of the *Guidelines*. However, it is still necessary to check performance of all other components and the structure as a whole.

Slab-column frames with weak columns having widely-spaced transverse reinforcement may be susceptible to story collapse due to column failure. The specified procedure is the same as that specified for beam-column frames in Section 6.5.2.4.

The *m* values were developed from experience and judgment of the project team, guided by available test data (Pan and Moehle, 1989; Martinez-Cruzado et al., 1991; Hwang and Moehle, 1993; Goel and Masri, 1994; Graf and Mehrain, 1992; Meli and Rodriguez, 1979; Durrani et al., 1995).

B. Nonlinear Static and Dynamic Procedures

It is preferred that inelastic response be limited to flexure in beams and columns, or inelastic rotation of slab-column connections. For components whose strength is limited by shear, torsion, and reinforcement development and splicing, the deformability usually is less than for flexure, and stability under repeated deformation cycles is often questionable. Where these latter forms of inelastic action are permitted as part of the design, they should preferably be limited to a few components whose contribution to total lateral load resistance is a minority.

C6.5.4.5 Rehabilitation Measures

The rehabilitation strategies or techniques are similar in principle to those described for beam-column frames in Section 6.5.2.5. The *Commentary* to that section provides general information. In addition, the following aspects apply specifically to slab-column construction.

Jacketing existing slabs, columns, or joints with new steel or reinforced concrete overlays. Where the objective is to improve the strength or ductility of the slab-column connection region, reinforced concrete or steel capitals may be added. These approaches are described by Martinez-Cruzado et al. (1991) and Lou and Durrani (1994). Alternatively, steel plates can be epoxied to both sides of the slab, around the column with through-bolts added to act as plate stiffeners and shear reinforcement (Martinez-Cruzado et al., 1991).

C6.6 Precast Concrete Frames

C6.6.1 Types of Precast Concrete Frames

Many types of precast concrete frames have been constructed since their inception in the 1950s. Some have inherent limited lateral-load-resisting capacity because of the nature of their construction details and because they were consciously designed for wind or earthquake loads. Except for emulated systems and braced systems (Section 6.6.1.1), these frames have capacities to resist lateral loads that are limited by elastic level deformations. In many double tee and single tee systems, as well as others, there is a lack of a complete load path. Brittle welded connections are very common. Many columns and beams lack sufficient confinement steel to provide ductility, and some column systems have inadequate shear capacity as well as base anchorage. Other columns have moment capacity at the base plate that is far beyond their ability to accept the deformations imposed by the global system. Each system may contain details or configuration characteristics that make it unique. Careful study of each unique system is required. In addition, Section C6.12 should be carefully reviewed.

C6.6.2 Precast Concrete Frames that Emulate Cast-in-Place Moment Frames

Frames of this type have been used intermittently since the mid-1950s. Columns with beam stubs are precast with rebar extending from beam or column ends that are connected to other precast members. The joint region has reinforcing extending into it from each of the common members. The joint is “tied” with confining stirrups and then completed by casting the concrete into the gap.

Deficiencies of this type of frame are consistent with those of traditional cast-in-place frames. Additional concerns are for the shear transfer across the joint, confinement of the joint, and tensile steel lap lengths in the joint. The system also requires dowels through the interface between the precast components and the horizontal framing. In many cases this was accomplished using threaded inserts that may or may not have ductile-force-resisting characteristics.

C6.6.3 Precast Concrete Beam-Column Moment Frames Other than Emulated Cast-in-Place Moment Frames

There is a wide variation of frames in this category. The common characteristic is potentially brittle connections that were constructed to resist gravity and wind loads. The addition of shear walls and or steel bracing systems is a primary means for seismically rehabilitating buildings. When employing this or any other approach, a complete load path must be established, with each joint in the system being analyzed for its ability to transmit the required forces and deform appropriate amounts.

C6.6.4 Precast Concrete Frames Not Expected to Resist Lateral Loads Directly

Frames of this category are similar to those of Section C6.6.3, except that it is assumed that other elements resist the lateral loads. Refer to Sections C6.6.3 and C6.6.2.

C6.7 Concrete Frames with Infills

C6.7.1 Types of Concrete Frames with Infills

These types of frames were common starting around the turn of the century. The infill commonly was provided along the perimeter of the building, where it served to clad the building and provide required fire resistance. Design of both the infill and the concrete frame in older buildings typically did not include consideration of the interaction between the frame and infill under lateral loads.

C6.7.1.1 Types of Frames

Infilled frames in older construction almost universally are of cast-in-place construction, and usually are of beam-column construction. However, the general principles of infilled frames as presented in the *Guidelines* are applicable to other types of concrete frames as well. The engineer should anticipate that the frame was designed primarily or exclusively as a gravity-load-carrying frame. The infill probably was not designed to be load-bearing. Frame girders commonly may have been designed without consideration of framing continuity; therefore, only nominal negative moment reinforcement may be present. Beam bottom reinforcement may or may not be continuous into supports. Column longitudinal reinforcement typically was spliced with laps or dowels at or near the floor level. Transverse reinforcement is likely to be relatively light by current standards.

C6.7.1.2 Masonry Infills

No commentary is provided for this section.

C6.7.1.3 Concrete Infills

Concrete infills in existing construction commonly are of cast-in-place concrete. Concrete was used as the infill because of lower cost and because the architectural requirements did not mandate masonry. Concrete infills may be mixed with masonry infills, the concrete infills being used in less visible bays of the framing. The concrete infill in existing buildings commonly was about eight inches thick. Most walls contain some reinforcement, but it may be as light as 3/8-inch bars at 24 inches on centers in one layer in each principal direction. Reinforcement may not extend into the surrounding frame, resulting in a plane of weakness around the perimeter of the infill. Infills may vary over building height, resulting in structural irregularities.

C6.7.2 Concrete Frames with Masonry Infills

C6.7.2.1 General Considerations

This section is concerned primarily with the overall element model, and the behavior and evaluation of the concrete frame. Behavior and evaluation of the masonry infill is covered in detail in Chapter 7.

Infilled frames have demonstrated relatively good performance, although there are some notable exceptions. Lack of toughness in the reinforced concrete framing elements can be a cause of severe damage and collapse, especially for older construction lacking details to provide ductility and continuity. The analysis model should be able to identify deficiencies in the concrete frame related to interaction with the masonry panels. For relatively undamaged infills, the columns act essentially as tension and compression chords of the infilled frame, with relatively large tension and compression forces possible along a substantial length of the column. Adequacy of splices to resist tension, and adequacy of concrete to sustain potentially large compression strains, needs to be considered. As the masonry infill becomes more heavily damaged, in addition to the action as a boundary element, the columns may be loaded locally by large forces from the masonry panel, with the centroid of those forces being eccentric from the beam-column joints. Severe damage to the columns can result. Details of this interaction are in Chapter 7.

C6.7.2.2 Stiffness for Analysis

Chapter 7 contains details on modeling of infilled frames.

The literature contains numerous reports of simulated earthquake load tests on concrete frames with masonry infills; these may provide insight on behavior and modeling issues. Refer to Abrams and Angel (1993), Altin et al. (1992), Fiorato et al. (1970), Gavrilovic and Sendova (1992), Klingner and Bertero (1976), Schuller et al. (1994), and Zarnic and Tomazevic (1985).

C6.7.2.3 Design Strengths

No commentary is provided for this section.

C6.7.2.4 Acceptance Criteria

The acceptance criteria were developed from experience and judgment of the project team, guided by available test data. The strain limits in Table 6-15 are based on experience with axially-loaded columns.

For columns in compression, confinement enables the concrete to sustain load for strains well beyond the crushing strain of 0.002 to 0.003. Ultimate limits for confined columns in compression may be limited by reinforcement buckling. For poorly confined columns, compressive resistance may drop rapidly following initial crushing of concrete. The capacity to sustain gravity loads beyond this point depends on the level of gravity load, and on the capability to redistribute gravity loads to other components, including the masonry infill. Further discussion of compressive strain capacity is provided in Section 6.4.3.

For columns in tension, stress and strain capacity may be limited by the capacity of lap splices. In primary components, failure of a lap splice effectively signals the end of reliable lateral force resistance. In secondary components, splice failure may result in significant loss of lateral load resistance, but gravity load resistance is likely to be sustained; an exception is where axial loads approach the axial load capacity, in which case concrete splitting associated with splice failure may result in reductions in axial compression capacity of the column. Additional discussion on splice strength is provided in Section 6.4.5.

A. Nonlinear Static and Dynamic Procedures

The numerical model should properly represent the load-deformation response of the infilled frame. Figure C6-14 presents some typical load-deformation relations measured in laboratory tests.

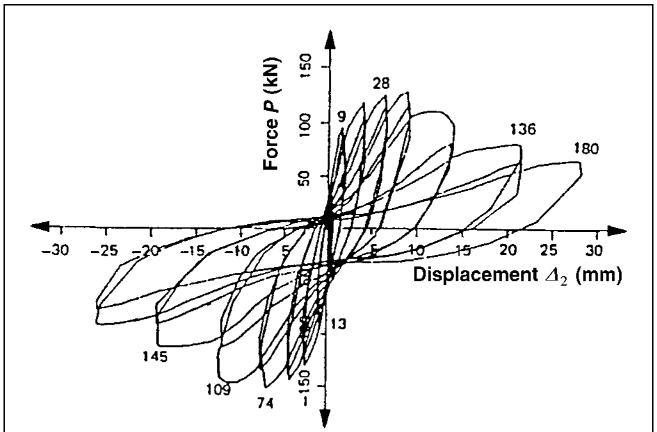


Figure C6-14 Load-Deformation Relation for Masonry-Infilled Reinforced Concrete Frame

C6.7.2.5 Rehabilitation Measures

In addition to the specific procedures listed in this section, the engineer should refer to additional procedures for infills in Chapter 7.

- **Jacketing existing beams, columns, or joints with new reinforced concrete, steel, or fiber wrap overlays.** This approach is especially suitable when overlays are placed over the masonry infill to achieve improved strength and toughness. Overlays may include reinforced concrete, fiberglass, carbon fiber, kevlar, or other materials. Examples are provided in Ehsani and Saadatmanesh (1994) and Zarnic et al. (1986). Jacketing of beams, columns, and joints is not likely to be a primary approach to rehabilitation of existing infilled frames, because it is not possible to fully encase beams or columns due to the presence of the infill.

- **Post-tensioning existing beams, columns, or joints using external post-tensioned reinforcement.** Lateral deformations of slender walls may result in significant tension force requirements for boundary columns, which may lead to unacceptable behavior of reinforcement splices. Post-tensioning can be considered as an option for precompressing columns to avoid excessive tension forces. When this approach is adopted, the design needs to also consider the possible negative effects on column behavior when the lateral forces reverse and the column becomes loaded in compression.

- **Modifying of the element by selective material removal from the existing element.** This is a primary method of rehabilitating existing infilled frames. In general, removal of existing infills should not result in vertical or plan irregularities in the structural system.

- **Improving of deficient existing reinforcement details.** This approach may be useful for improving tension lap strength of existing column lap splices. When this option is selected, chipping of concrete cover may be required; care should be exercised to ensure that core concrete, and bond with existing transverse reinforcement, are not damaged excessively.

- **Changing the building system to reduce the demands on the existing element.** This is a primary method of rehabilitating existing infilled frames. By adding sufficiently stiff elements, it may be possible to reduce design demands on the infills to acceptable levels. Concrete walls may be particularly suitable for this purpose; steel braced frames, and especially eccentrically-braced frames, may lack adequate stiffness to protect the infill from damage. Where new elements are added, the design must ensure adequate connections with adjacent elements. Seismic isolation and supplemental damping may also be used to reduce demands to acceptable levels.

C6.7.3 Concrete Frames with Concrete Infills

C6.7.3.1 General Considerations

Traditionally, a variety of analysis models have been used to model concrete frames with concrete infills. One approach has been to assume that the frame is sufficiently flexible and weak that framing action does not appreciably influence behavior. In this extreme, the frame with infill is modeled as a solid shear wall. This approach is often suitable in cases where the frame is relatively flexible, but may not be suitable for walls with openings, or for stiff frames (typically those with deep spandrels and short columns). Another extreme has been to completely ignore the infill in the numerical model. This approach is often unsuitable because it overlooks potentially significant interaction effects. These effects include overall element strength and stiffness, as well as potentially detrimental effects on columns acting as boundary elements or otherwise interacting with the frame. Detailed discussion of this interaction is provided in Section 6.7.2 and Chapter 7. Braced-frame analogies may be used to identify some aspects of the interaction.

The current state of knowledge does not justify recommendation of generally applicable modeling rules. Engineering judgment provides the only rule of general application. Engineering judgment may be guided by detailed finite-element solutions of subassemblies. Experimental data are lacking; therefore, testing of subassemblies is encouraged where feasible.

C6.7.3.2 Stiffness for Analysis

Because of the lack of experimental data, engineering judgment is required when establishing modeling parameters. Where the frames are relatively flexible and weak, and the infills are in good condition and adequately connected with the frame, the general procedures for walls in Sections 6.8 and 6.9 may provide guidance. Where the frames are relatively stiff and strong, and the infills are relatively weak, the general procedures for concrete frames with masonry infills in Section 6.7.2 may provide guidance.

C6.7.3.3 Design Strengths

Shear strength provided by a concrete infill is likely to depend on the shear strength of the infill itself, and the interface between the infill and the surrounding frame. In existing construction, the infill reinforcement is likely to be relatively light, and is likely to not be anchored into the surrounding frame. As noted in Section 6.8.2.3, where the reinforcement ratio is low, the shear strength is to be calculated using procedures that differ from those in *ACI 318-95* (ACI, 1995). Where the infill reinforcement is not anchored in the surrounding frame, sliding along the interface may occur during lateral loading. In this case, shear is introduced to the frame primarily by direct bearing (lug action) between the infill and the surrounding frame. In this case, shear strength may be limited by direct shear strength of the infill, by local crushing of the infill where it bears against the surrounding frame, or by shear failure of the surrounding frame because of the eccentric bearing of the infill against the frame. These basic behaviors are similar to those described for masonry infills in Chapter 7. Lacking experimental data, the *Guidelines* assume the strength to be limited by direct shear strength of the infill.

Similarly, flexural strength of an infilled frame is likely to be influenced by continuity of the longitudinal reinforcement. Lap splices in the boundary columns may limit strength and deformation capacity. If the infill reinforcement is not anchored in the surrounding frame, it should not be included in the design strength.

C6.7.3.4 Acceptance Criteria

Engineering judgment is required in establishing the acceptance criteria because of the lack of relevant test data. In general, the following aspects should be considered.

- The surrounding frame should be checked for action in tension and compression as described in Section 6.7.2.4. Where portions of the frame are not infilled, the relevant criteria of Section 6.5 should be checked.
- The infilled frame should be checked according to criteria in Section 6.7.2.4.
- Where the relative stiffnesses and strengths of the frame and infill result in effectively composite action, the relevant criteria of Sections 6.8 and 6.9 should be considered.

C6.7.3.5 Rehabilitation Measures

Tests on walls thickened by jacketing have been reported by Goto and Adachi (1987) and Motooka et al. (1984). Infills have also been used to retrofit existing frame construction. Relevant test data on frames rehabilitated by concrete infills can be found in Aoyama et al. (1984), Hayashi et al. (1980), Jirsa and Kreger (1989), and Kahn and Hanson (1979).

C6.8 Concrete Shear Walls

C6.8.1 Types of Concrete Shear Walls and Associated Components

Due to their high initial stiffness and lateral load capacity, shear walls are an ideal choice for a lateral-load-resisting system in a reinforced concrete (RC) structure. Slender walls will normally exhibit stable ductile flexural response under severe lateral loading, but squat walls are more likely to be governed by shear response, so they must be designed for lower ductilities. In residential construction, the generous use of walls provides ample redundancy and load capacity to keep seismic forces and deformation demands relatively low. However, due to architectural restraints in office buildings, there tend to be fewer shear walls, and horizontal spans are kept as short as possible. Thus, these walls are usually slender, and seismic deformation demands tend to be high.

There are three general structural classifications in which shear walls are used as the primary lateral-load-resisting elements. In bearing wall systems, shear walls serve as the primary members for both gravity and lateral load resistance. Such structures have often been considered to behave in a nonductile manner when subjected to large lateral loads, but studies of several bearing wall buildings following the 1985 Chile earthquake have shown that such structures may be very reliable for seismic resistance if there is a high percentage of wall area to total floor area (Wood et al., 1987; Sozen, 1989; Wood, 1991b; Wallace and Moehle, 1992 and 1993; Wight et al., 1996).

When a shear wall is assumed to be the only lateral-load-resisting system and a space frame is provided to carry most of the gravity load, the resulting structural system is commonly referred to as a shear wall system. In such systems the shear walls often form the perimeter of an interior core that contains the elevator shaft and stairways. In some cases the core walls will work in combination with isolated walls that are distributed around the perimeter of the building, to increase the torsional stiffness of the building.

Where shear walls are combined with a space frame that carries most of the gravity load and also assists in resisting lateral loads, the structure is referred to as a dual (wall-frame) system. Again, the most common use for the shear walls in such a system would be to form an interior core. Because of the different elastic displacement modes for walls and frames, the dual system offers significant stiffness benefits in the elastic range of lateral loading. For inelastic lateral loading, the frame offers a second line of defense, which provides significant lateral stiffness and strength after initial yielding at the base of the shear walls.

For any one of these three general structural systems, shear walls that are in the same plane may be joined together at each floor level with coupling beams to form a coupled-wall system. As with a wall-frame system, the coupled-wall system offers a significant increase in lateral stiffness compared to the algebraic sum of lateral stiffnesses of isolated shear walls. Under inelastic lateral loadings, the coupling beams can provide significant energy absorbtion if they are properly detailed.

In bearing wall systems, the shear walls may have a pattern of large openings in both the horizontal and vertical directions. Such walls are commonly referred to as either a “framed-wall” or a “perforated-wall system.” Perforated walls are typically used along the exterior of buildings to form a repetitive pattern of window openings. The behavior of such a wall system is more often dominated by the behavior of the individual vertical and horizontal wall segments, than by the overall proportions of the wall. The vertical wall elements are commonly referred to as “wall piers.” There is no common terminology for the horizontal wall segments that resemble deep beams. For all the tables presented in the *Guidelines*, the term “wall segments” refers to both the horizontal and vertical members of a perforated wall.

Although they are frame elements, coupling beams and columns that support discontinuous shear walls are included in this section of the *Guidelines*. When these elements are used in a shear wall system, they will commonly have large ductility demands under large lateral load reversals. Therefore, the detailing of the reinforcement in such members, particularly the transverse reinforcement, is critical to the behavior of these elements. Of course, the inelastic behavior of these elements will strongly influence the lateral load response of the shear wall system in which they are located.

C6.8.1.1 Monolithic Reinforced Concrete Shear Walls and Wall Segments

A slender shear wall will commonly have longitudinal reinforcement concentrated either along its horizontal edges or within a boundary element. For both cases, the percentage of longitudinal steel concentrated at the wall edge and the amount and spacing of the transverse reinforcement used to confine that steel will have a significant influence on the inelastic lateral load response of the shear wall. A large percentage of longitudinal reinforcement will increase the shear required to cause flexural yielding under lateral loading, and will increase the compressive strains along the compression edge of the wall. The increased shear could either trigger an early shear failure or cause a more rapid deterioration of stiffness under lateral load reversals. The high compression strains could lead to concrete crushing and rebar buckling unless the compression edge of the wall is well confined by transverse reinforcement.

Squat shear walls normally have a uniform distribution of vertical and longitudinal steel. If the percentage of vertical steel is low, flexural behavior may govern inelastic response under lateral loads. If shear governs the lateral load behavior, either the available shear ductility should be assumed to be a small value, or the shear strength of the wall should be designated as a force-controlled action. Details are given in Section 6.8.2.4 of the *Guidelines*.

C6.8.1.2 Reinforced Concrete Columns Supporting Discontinuous Shear Walls

RC columns that support discontinuous shear walls are subjected to large force and displacement demands during severe ground shaking. The damage inflicted on the first story columns of the Olive View Hospital during the 1971 San Fernando earthquake is an often-used example of the demands placed on such columns. Modern building codes have limitations on stiffness discontinuites that tend to eliminate this type of construction. However, there are existing RC buildings with shear walls that are not continuous to the foundation level. For these buildings, the columns that support the discontinuous walls will need to be carefully analyzed.

In most cases, the shear strength of columns supporting discontinuous shear walls will be a force-controlled action. These columns should be analyzed as displacement-controlled members only if they have transverse reinforcement that satisfies ductile detailing requirements of modern codes. Even in these cases, the permitted ductility values will be very low. Following the 1995 earthquake in Kobe, Japan, there have been reports (Bertero et al., 1995; Watabe, 1995) of damage to RC columns supporting discontinuous shear walls in very modern RC structures. The columns were well detailed, but the displacement demands were excessive.

C6.8.1.3 Reinforced Concrete Coupling Beams

RC coupling beams are normally deep with respect to their span. Observations of post-earthquake damage to concrete shear wall buildings have repeatedly shown diagonal tension failures (severe X-cracking) in coupling beams. The most common cause of this damage is insufficient shear strength to develop the beam's flexural strength under repeated cyclic loading. Any contribution from the concrete to the shear capacity should be ignored and closed stirrups should be provided at a close spacing (ð d/4). However, even these measures may only delay, and will not necessarily prevent, an eventual shear failure under repeated large load reversals (Paulay, 1971a).

Research (Paulay, 1971b) has shown that coupling beams designed with primary reinforcement arranged in a diagonal pattern over the length of the beam will exhibit more stable behavior under large load reversals than will conventionally reinforced coupling beams. When diagonal reinforcement is used, it should be designed to resist the vertical shear forces that accompany flexural yielding of the reinforcement.

C6.8.2 Reinforced Concrete Shear Walls, Wall Segments, Coupling Beams, and RC Columns Supporting Discontinuous Shear Walls

C6.8.2.1 General Modeling Considerations

Using equivalent beam-column elements to model the elastic and inelastic response of slender shear walls is a fairly common practice. A primary reason for using equivalent beam-column models for shear walls is because numerous frame analysis programs are available to a structural engineer. The use of an equivalent beam-column model to represent inelastic behavior of shear walls and wall segments is normally acceptable for slender elements with aspect ratios above those stated in the *Guidelines*, where flexural response will dominate. However, in all these cases the equivalent beam-column must incorporate shear deformations and the beams connecting to the equivalent beam-column element must have long rigid end zones to properly simulate the horizontal dimension of the shear wall. Results from a large number of shear wall tests have been summarized by Wood (1991a).

For squat shear walls, or other walls where shear deformations will be significant, a more sophisticated wall model should be used. This model should incorporate both elastic and inelastic shear deformations, as well as the full range of flexural behavior. Researchers have suggested the use of multiple spring models (Otani, 1980; Otani et al., 1985; Alama and Wight, 1992), and multi-node link models to represent an RC shear wall (Charney, 1991).

Most coupling beams have small span-to-depth ratios, so any beam element used to model a coupling beam must incorporate shear deformations. Several researchers have developed special beam elements specifically for simulating the response of an RC coupling beam (Saatcioglu, 1991).

Columns that support discontinuous shear walls can be modeled with a beam-column element similar to that used in most frame analysis programs. However, the element should include shear deformations and the possible rapid loss of shear strength under large lateral deformations and high axial load.

C6.8.2.2 Stiffness for Analysis

Typical sources of flexibility in RC members were discussed in Section C6.4.1.2.

A. Linear Static and Dynamic Procedures

The linear procedures of Chapter 3 assume that the element stiffness used in analysis approximates the stiffness of that element at displacement amplitudes near its effective yield displacement. At such displacement levels, the effective element stiffness will be significantly less than the gross stiffness commonly used in conventional design practice. A discussion of how the effective stiffness may vary as a function of the source of deformation and level of stress is given in Section C6.4.1.2. In lieu of a more precise analysis, the effective element stiffnesses for linear procedures should be based on the approximate values given in Table 6-4.

B. Nonlinear Static and Dynamic Procedures

The nonlinear procedures of Chapter 3 require the definition of the typical nonlinear load-deformation relationship for each displacement-controlled action. For the NSP, it is sufficient to define a load-deformation relationship that describes the behavior of an element under monotonically increasing lateral deformations. For the NDP, the same basic load-deformation relationship can be used as a backbone curve, but it is also necessary to define rules for the load-deformation relationship under multiple reversed deformation cycles. Figure 6-1 shows typical load-deformation relationships that may be used for the NSP. Definitions of the key points in this figure are given in the *Guidelines*.

When using the basic load-deformation curves given in Figure 6-1, the ordinates (loads) are to be a function of the member strengths defined in Section 6.8.2.3. The deformation values (x-axis) are to be defined as either plastic hinge rotations, drifts, or chord rotations, depending on the type of element involved and whether the element's inelastic response is governed by flexure or shear. Plastic hinge rotations are used where flexure governs the inelastic response for shear walls and wall segments, and for RC columns supporting discontinuous shear walls. It should be clear that RC columns that have shear strengths below the shear required to develop flexural hinging are not included in this discussion.

A sketch of the first story of a deformed shear wall governed by flexure is given in Figure C6-15. The length of a plastic hinging region in an RC member is typically defined to be somewhere between 0.5 and 1.0 times the effective flexural depth of the member. For RC members where shear deformations are significant, the plastic hinging length tends toward the upper end of this range, and vice versa. Therefore, for the shear walls the plastic hinging region will extend very close to, if not beyond, one story height of the member. In these cases it is appropriate to limit the length of the plastic hinging zone to one story height. For wall segments that often have small length-to-depth ratios, the plastic hinging zone may extend to mid-length of the member. For those cases, the length of the plastic hinging zone is limited to one-half the length of the member. For RC columns that support discontinuous shear walls, the plastic hinge length is taken to be taken as one-half the effective flexural depth, as is done for typical RC frame members.

For members whose inelastic response is controlled by shear, Figure 6-1(b) should be used to characterize the inelastic behavior of the member and *drift* should be used as the deformation value. Drift for shear walls is defined as the lateral displacement over one story height, divided by story height (Figure C6-16). For wall segments, drift is defined as the transverse displacement of the member over its length, divided by the member length.

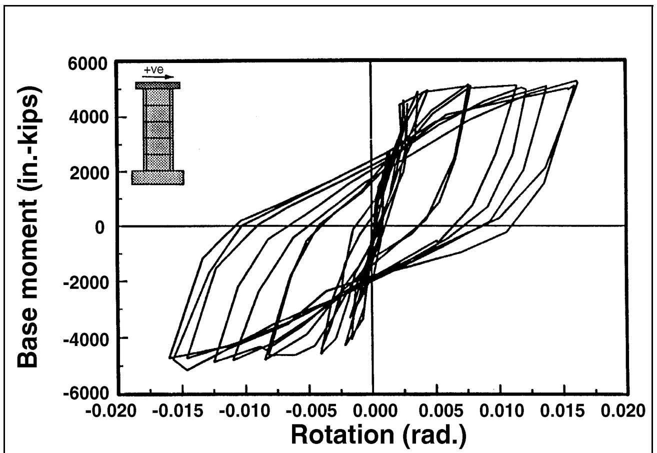


Figure C6-15 Shear Wall Base Moment versus First-Story Rotation Relationship (Specimen W-1, Ali and Wight, 1991)

Figure 6-1(b) is also used to characterize the inelastic behavior of coupling beams, whether their inelastic response is governed by flexure or by shear. Chord rotation, as defined in Figure 6-4, is considered to be the most appropriate deformation measure for inelastic response of coupling beams.

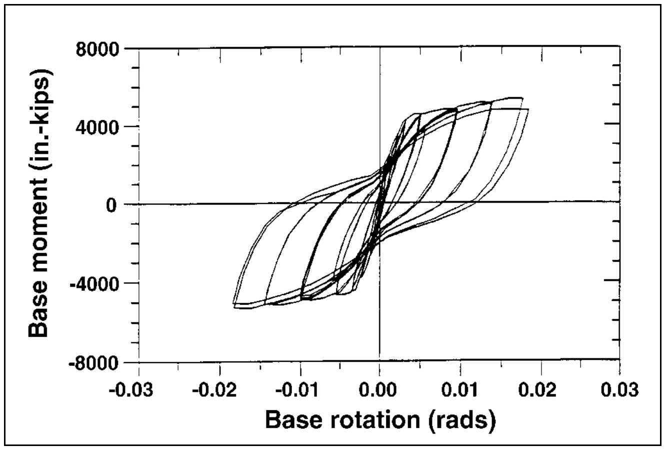


Figure C6-16 Shear Wall Base Moment versus Base Rotation Relationship (Specimen RW1, Thomsen and Wallace, 1995)

Values for the hinge rotation values *a* and *b* (which are described in Figure 6-1(a) and given in Table 6-17) and the drift or chord rotation values *d* and *e* (which are described in Figure 6-1(b) and given in Tables 6-17 and 6-18) are based on experimentally observed behavior of RC members and the engineering judgment of the project team. Experimental results of the inelastic behavior of elements defined in Tables 6-17 and 6-18 are described in the following sections of the *Commentary.*

C6.8.2.3 Design Strengths

Component strengths are to be calculated based on the principles and procedures from *ACI 318-95* (ACI, 1995) and the 1994 *NEHRP Recommended Provisions* (BSSC, 1995), with some modification to reflect different purposes of the *Guidelines* and those documents. The design engineer must consider all potential failure modes that may occur at any section along the length of the member under consideration.

When calculating the nominal flexural yield strength of a shear wall or wall segments, it is assumed that only the longitudinal steel in the outer portions of the wall will yield initially. As lateral deformations increase, section rotations in the plastic hinging region will increase to the point that essentially all the longitudinal steel will be yielding. This point is assumed to represent the nominal flexural strength of the member. For both the yield strength and nominal flexural strength calculation, the value for the yield strength of the reinforcement should be increased by 25% to account for actual yield strengths exceeding the specified yield strength, and the onset of strain hardening in the reinforcement at rotations beyond the yield rotation.

For shear-controlled shear walls and wall segments, no difference is assumed between the shear yield strength and the nominal shear strength of the element. Also, the reinforcement strength is set equal to the specified yield strength. These conservative assumptions are used for additional safety because shear-controlled members have less ductility and are usually more brittle that flexure-controlled members.

Similar procedures are used to evaluate the nominal flexure and shear strengths of coupling beam elements. For RC columns supporting discontinuous shear walls, nominal strengths are based on the procedures developed in Section 6.5.2.3 of the *Guidelines*.

C6.8.2.4 Acceptance Criteria

A. Linear Static and Dynamic Procedures

The acceptance criteria of Chapter 3 require that all component actions be classified as either displacement-controlled or force-controlled actions. For most RC members, it is preferable that deformation-controlled actions be limited to those members where flexural actions govern the nonlinear response. However, for some of the RC members covered in this section, shear may govern the strength and nonlinear response. Therefore, Table 6-20 includes *m* values for members controlled by shear. For RC columns that support discontinuous shear walls, *m* values are only given for members governed by flexure. Shear-critical RC columns must be considered as force-controlled components.

Where the linear procedures of Chapter 3 are used for design, they should be restricted to determining design values for yielding parts of the structure. The design actions for the force-controlled portions of the structure should be determined by statics considering the gravity forces plus the yielding actions for the deformation-controlled components in the structure. Members whose design forces are less than their respective capacities can be assumed to satisfy all the performance criteria of the *Guidelines*.

One example of laboratory data used to determine *m* values is given in Figure C6-15 (Ali and Wight, 1991). The figure shows the base moment versus base rotation relationship for a one-fifth scale five-story shear wall specimen. The specimen generally satisfies the conditions listed in the first row of Table 6-19. The wall reinforcement was symmetrical and the axial load was approximately equal to $0.1 t_w l_w \sqrt{f'_c}$. The wall had confined boundary elements and the maximum shear stress recorded during the test was approximately $3\sqrt{f'_c}$. A single lateral load was applied at the top of the specimen, so the base moment in Figure C6-15 was the lateral load multiplied by the height of the specimen. The base rotation was measured over one story height, which was approximately 0.55 times the length of the wall.

The general results given in Figure C6-15 indicate that this specimen was able to achieve base rotations exceeding 0.015 radians without any loss of strength. Clearly, the specimen could have achieved higher base rotations, but the testing was terminated because the maximum displacement capacity of the testing equipment had been reached. Although the interpretation of the yield point is somewhat subjective, it appears that the base rotation at yield for this specimen was approximately equal to 0.0025 radians. Thus, this specimen achieved a base rotational ductility of 6.0, without any indication of strength deterioration.

Similar test results have been reported by other researchers (Thomsen and Wallace, 1995; Paulay, 1986) for shear walls that also generally fit the conditions listed in the first row of Table 6-19. The base moment versus base rotation results from Thomsen and Wallace are shown in Figure C6-16, and the lateral load versus top lateral displacement results from Paulay are shown in Figure C6-17. The results in Figure C6-16 are remarkably similar to those in Figure C6-15, and actually indicate a maximum base rotation approaching 0.020 radians. The results given in Figures C6-15 and C6-16 were used to determine appropriate *m* values for the first row of Table 6-19. The test results from Paulay are presented as further confirmation of the available ductility in shear walls satisfying the listed conditions.

Two other sets of test results from Thomsen and Wallace for shear walls governed by flexure are given in Figures C6-18 and C6-19. Both of these results are for walls with T-shaped cross sections. Positive moment corresponds to putting the flange of the section into compression, and negative moment corresponds to putting the stem of the section into compression. Before conducting these tests, Thomsen and Wallace had done analytical studies of T-shaped cross sections (Figure C6-20). The results of those studies had clearly

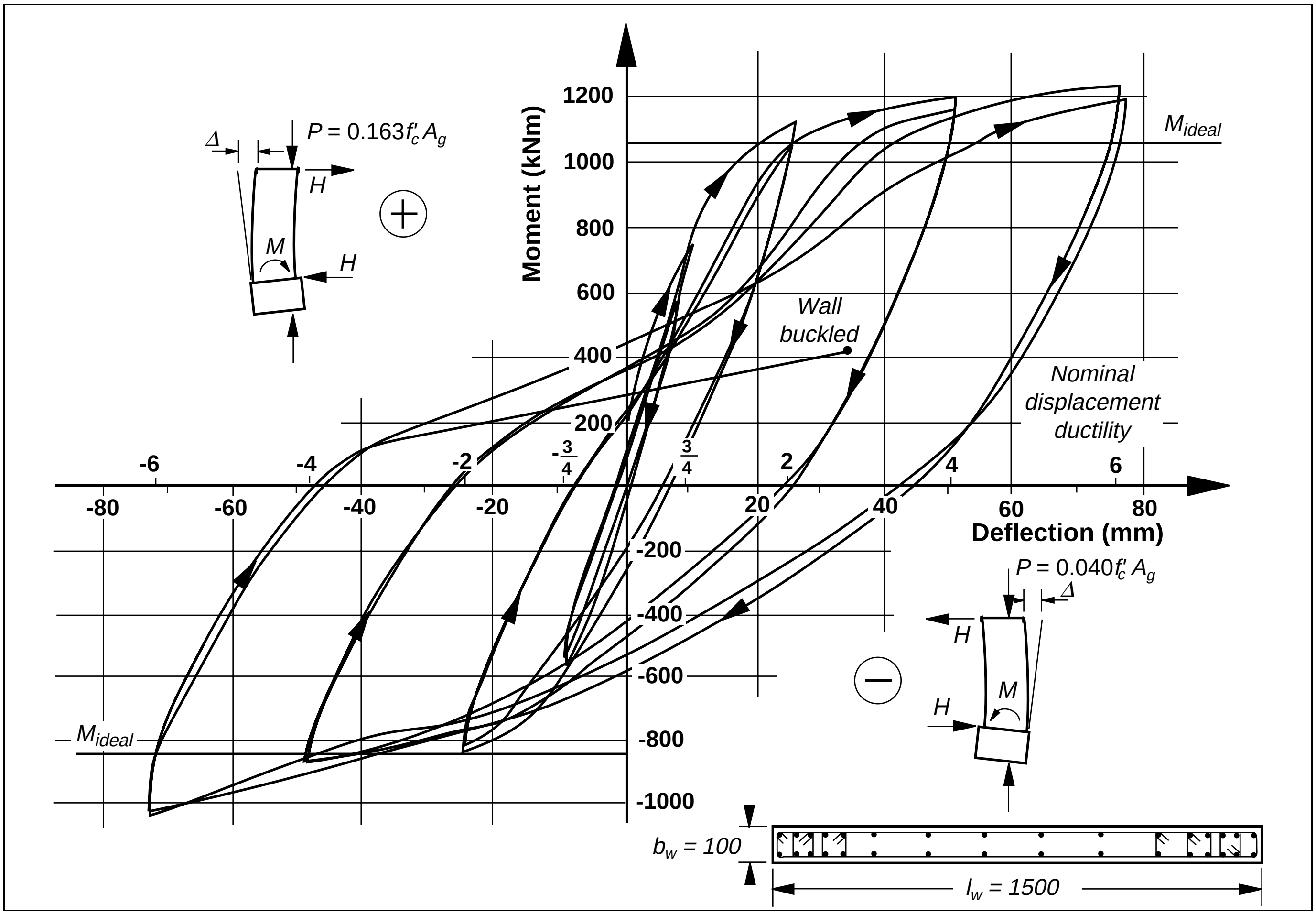


Figure C6-17 Lateral Load versus Top Displacement Relationship (Paulay, 1986)

indicated that less ductility should be expected when the stem of the T-section is subjected to compression.

The results shown in Figures C6-18 and C6-19 for negative bending should correspond to the conditions represented by rows three and seven, respectively, of Table 6-19. The axial load acting on the specimens was low, but the large difference between the tension steel area from the flange versus the compression steel area from the stem put the coefficient for the parameter in the first column of Table 6-19 above the given limit of 0.25. The results in Figure C6-18 represent a well-confined boundary region, and those in Figure C6-19 represent a poorly confined boundary region. The shear stress in both specimens was below $3\sqrt{f'_c}$.

The specimen shown in Figure C6-18 demonstrates a reasonable amount of ductility and reaches a maximum

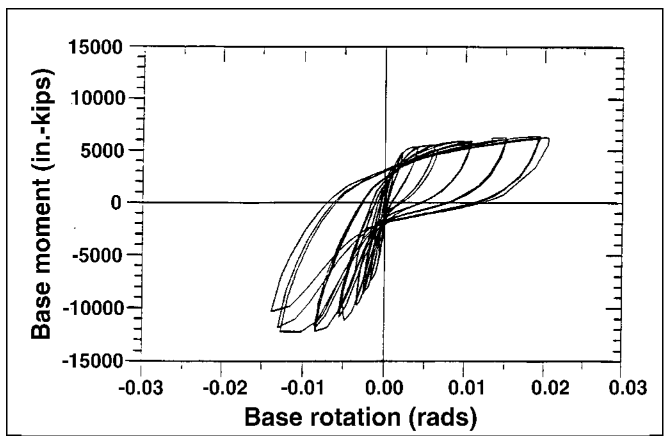


Figure C6-18 Shear Wall Base Moment versus Base Rotation Relationship (Specimen TW2, Thomsen and Wallace, 1995)

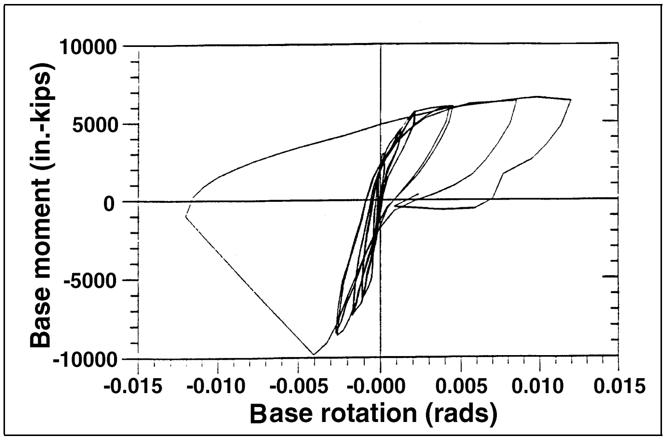


Figure C6-19 ***Shear Wall Base Moment versus Base Rotation Relationship (Specimen TW1, Thomsen and Wallace, 1995)***

base rotation of approximately 0.013 before experiencing a substantial loss in capacity. Because this specimen has less ductility and a more rapid strength loss at higher rotation values than shown by the specimens in Figures C6-15 and C6-16, lower *m* values are used in the third row of Table 6-19. The specimen in Figure C6-19 shows a very low amount of ductility, and this result is reflected in the *m* values used in the seventh row of Table 6-19.

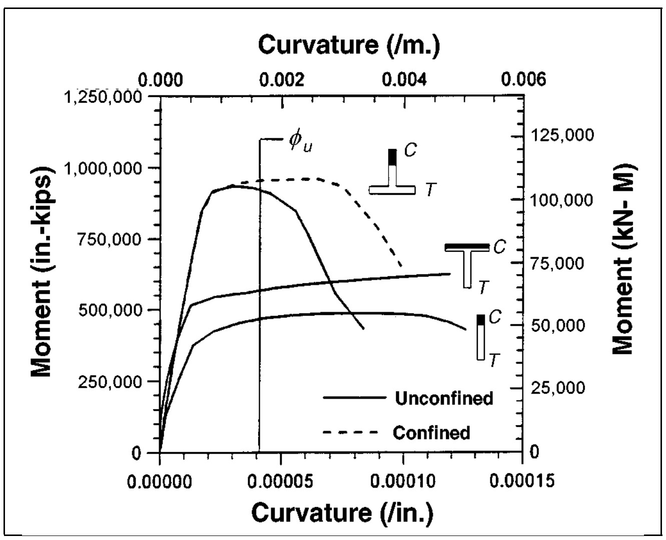


Figure C6-20 ***Analytical Moment-Curvature Relationship for Rectangular and T-Shaped Wall Sections (Thomsen and Wallace, 1995)***

Design engineers must use some judgment when interpreting test results for isolated specimens similar to those shown in Figure C6-19. When the compression zone of this specimen becomes unstable, the specimen fails immediately because there is nowhere else for the load to go. However, if this wall were contained within a building structure consisting of several walls and columns, its response would be much more stable. When the compression zone of this specimen started to deteriorate, it would become much less stiff, and loads in the structure would redistribute to stiffer lateral-load-resisting members. This wall could then be subjected to larger deformations while carrying less load. This assumed behavior is reflected in the *m* values of Table 6-19 and the residual strength values listed in Table 6-17.

Although flexure is the preferred mode of inelastic response for RC members (elements and components), shear will control the inelastic response of certain shear wall, wall segment, and coupling beam elements. Test results for a shear wall controlled by shear are shown in Figure C6-21 (Saatcioglu, 1991). This was a one-story specimen with a height of 1000 mm. Thus, a lateral top deflection of 10 mm corresponds to a 1% story drift.

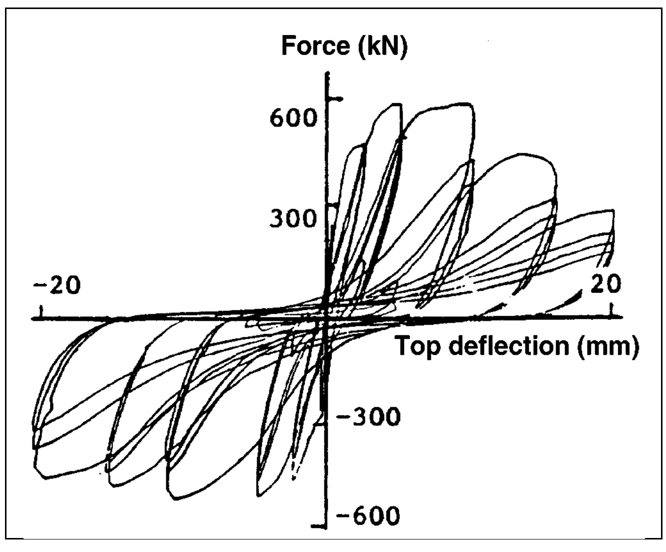


Figure C6-21 ***Lateral Shear Force versus Top Displacement of Shear Wall Specimen 1 (Saatcioglu, 1995)***

As stated previously, the determination of the yield point is somewhat subjective, but could be assumed to occur at a top displacement of approximately 2.5 mm (drift of 0.25%). Beyond this point, the specimen exhibited the development of diagonal cracks that continued to open wider as the lateral displacements were increased. The specimen reached a top displacement of 10 mm (drift of 1.0%) before it experienced a significant deterioration of its shear capacity. The specimen was able to achieve top displacements of 20 mm (drift of 2%) without a dramatic failure.

The results of another shear wall test by Saatcioglu are given in Figure C6-22. This specimen had more shear reinforcement, but suffered a shear sliding failure along its base. Although the hysteresis loops are much more pinched than those shown in Figure C6-21, the specimen still has significant deformation capacity without experiencing a sudden failure.

Again, judgment must be used with these test results to determine the *m* values given in the first row of Table 6-20 and the residual capacity given in the first row of Table 6-18.

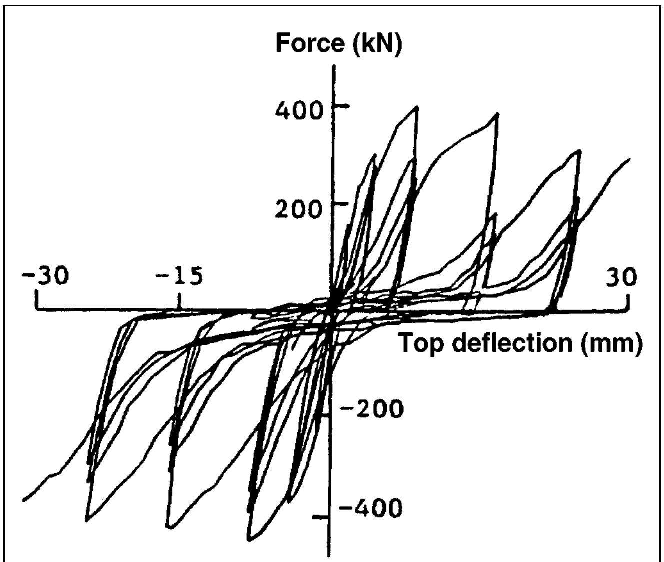


Figure C6-22 Lateral Shear Force versus Top Displacement of Shear Wall Specimen 4 (Saatcioglu, 1995)

Coupling beams are another RC element whose inelastic response is often controlled by shear. Measured lateral load versus chord rotation results for RC coupling beam specimens tested in New Zealand (Paulay, 1971a and 1971b) are presented in Figure C6-23. Both specimens had conventional longitudinal reinforcement and carried maximum shear stresses that exceeded $6\sqrt{f_c'}$. The results for a specimen with conforming transverse reinforcement are given in Figure C6-23; the results for a specimen with nonconforming transverse reinforcement are given in Figure C6-24. Thus, these results should correspond to conditions given in the second and fourth rows, respectively, of Part ii of Table 6-20.

The results shown in Figure C6-23 indicate that the specimen was subjected to only one load reversal after the yield capacity of the specimen was achieved. Thus, the test results are more of a monotonic backbone type curve. However, some required information can be obtained from these results. If one assumes that yield occurred at a chord rotation of approximately 0.004 radians, it then appears that the specimen achieved a rotational ductility of approximately three in each direction. The amount of strength deterioration that would have occurred at this ductility level cannot be determined because of the vary large displacement excursion in the negative direction. However, that large excursion does indicate that rotational ductilities as large as four are possible with little or no loss in capacity for monotonic loading.

Test results for the specimen with nonconforming transverse reinforcement are shown in Figure C6-24. Although the scale for the chord rotation axis has been expanded, it is clear that this specimen had a lower stiffness and a more pinched hysteretic response than was obtained for the specimen that had conforming transverse reinforcement. Thus, the *m* values that are given in the fourth row of Part ii of Table 6-20 are reduced from those given in the second row of Part ii.

A third set of test results from same series of RC coupling beam tests is given in Figure C6-25. This specimen's primary reinforcement was diagonal reinforcement, so it corresponds to the last row of Table 6-19. Clearly, the test results for this specimen indicate that larger rotational ductilities can be obtained and that the lateral load versus rotational hysteresis loops are fuller than obtained for the specimens with conventional longitudinal and transverse reinforcement. This improved behavior is reflected in the large *m* values given in the last row of Table 6-19.

B. Nonlinear Static and Dynamic Procedures

Inelastic response is only acceptable for those actions listed in Tables 6-17 and 6-18. Deformations

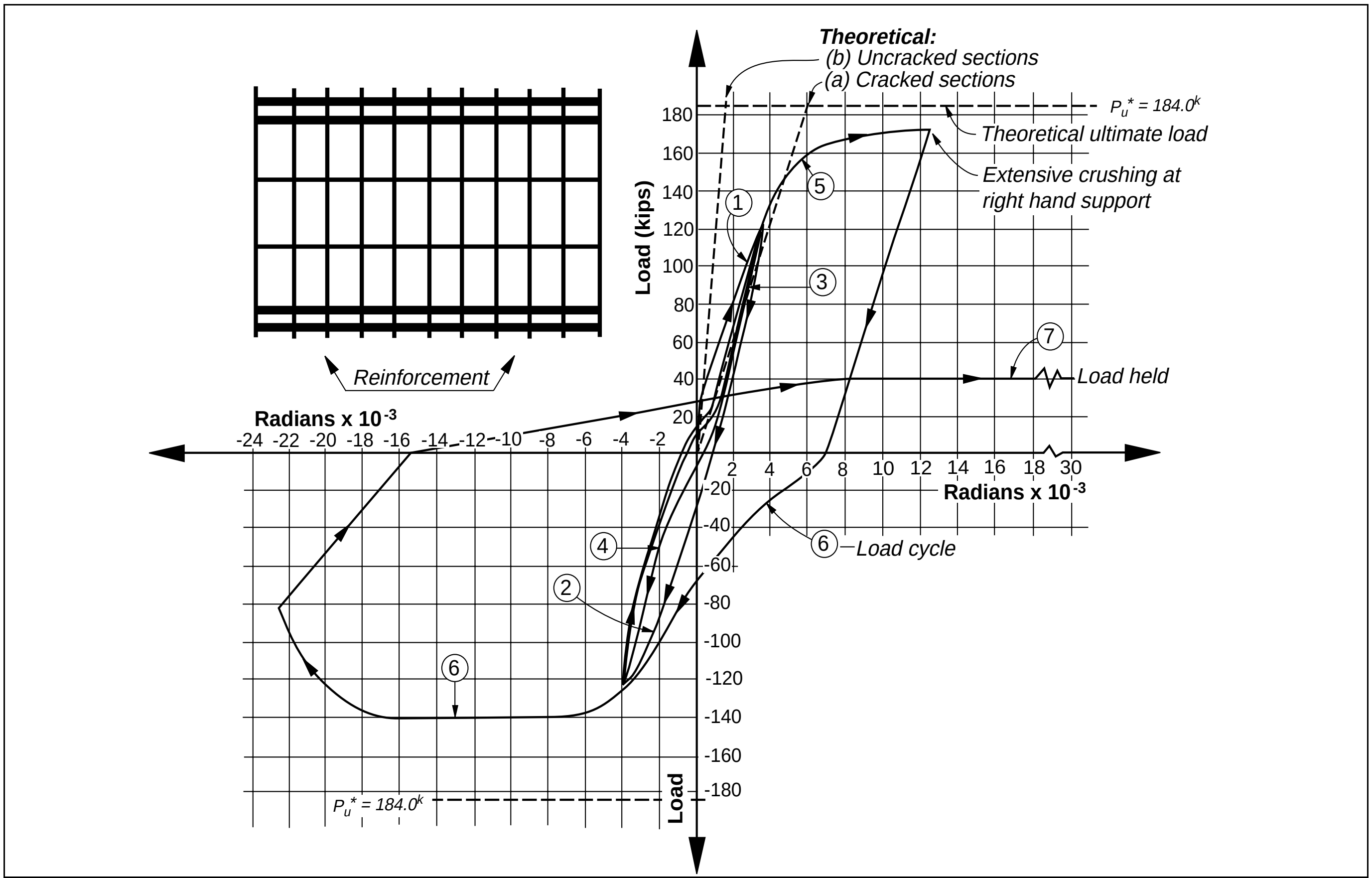


Figure C6-23 Lateral Load versus Chord Rotation Relationship Beam 315 (Paulay, 1971b)

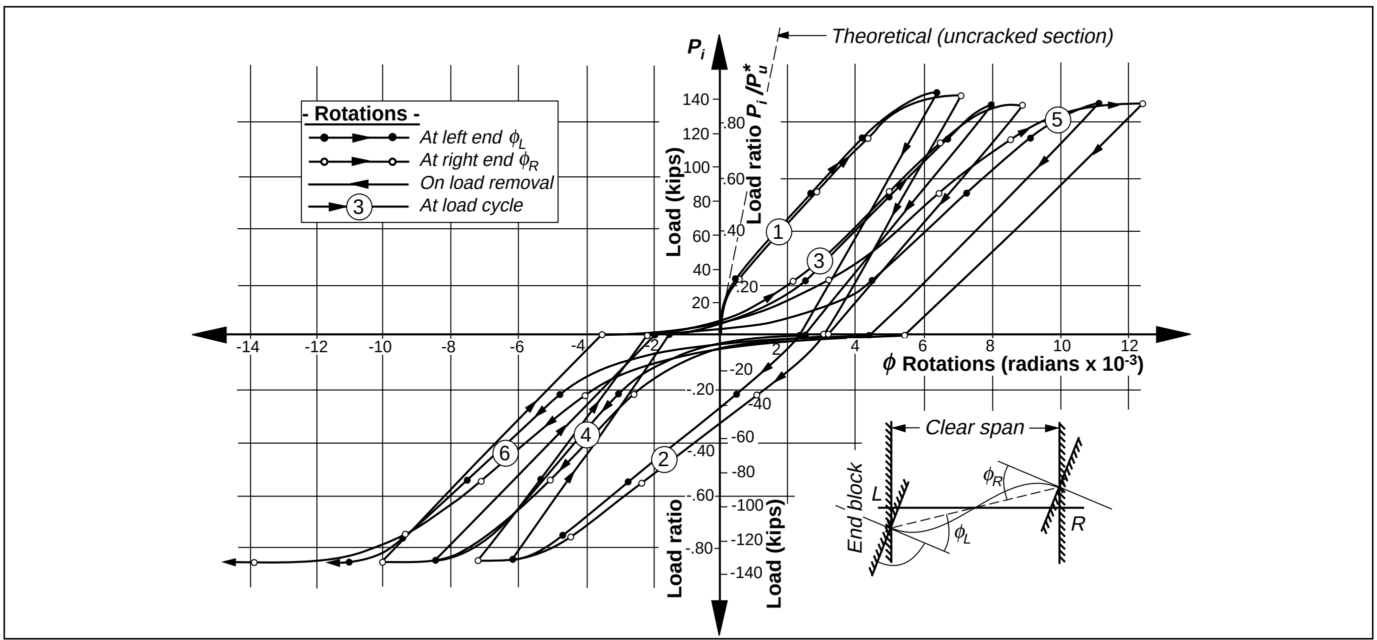


Figure C6-24 Lateral Load versus Chord Rotation Relationship Beam 312 (Paulay, 1971b)

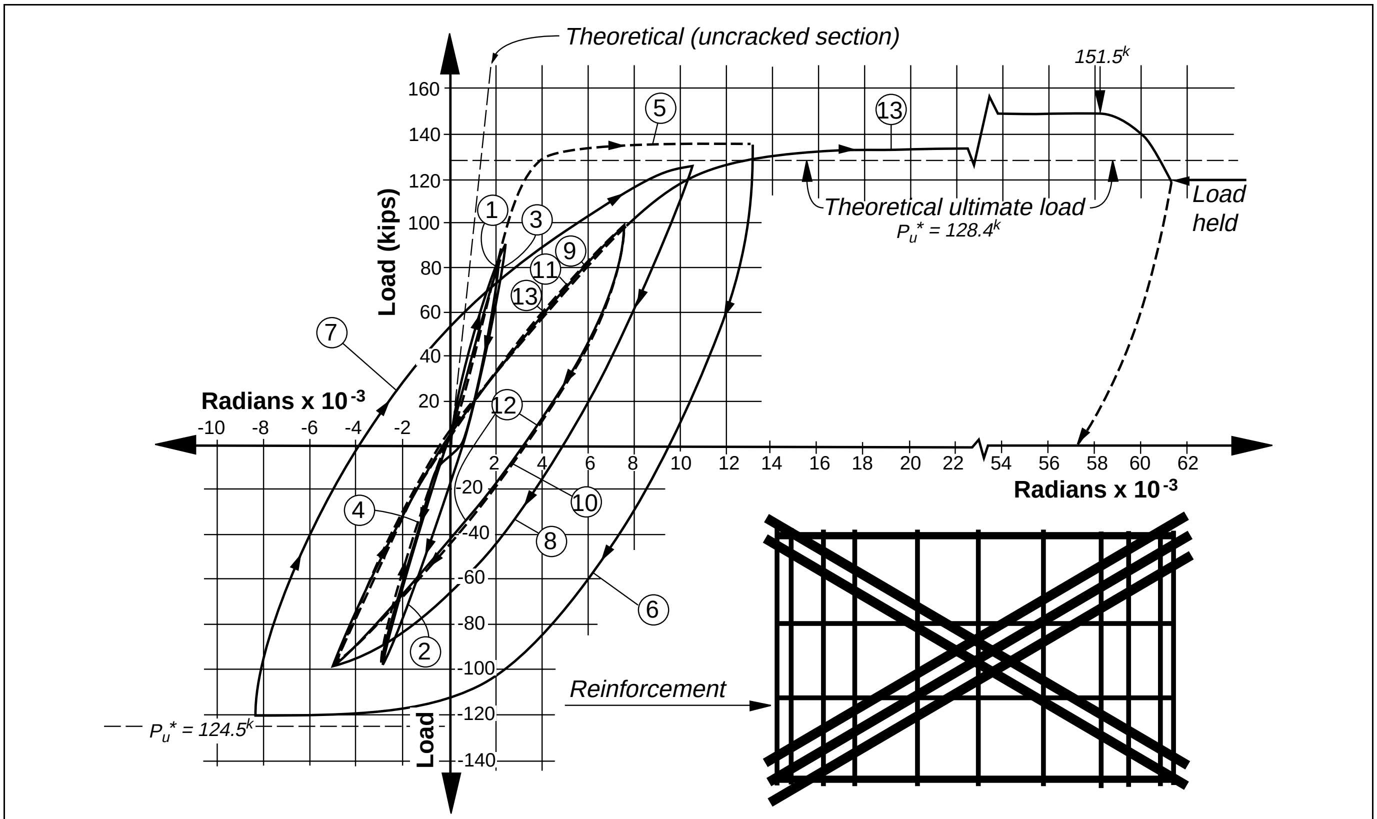


Figure C6-25 Lateral Load versus Chord Rotation Relationship Beam 316 (Paulay, 1971b)

corresponding to these actions shall not exceed the plastic hinge rotation, drift, or chord rotation capacities given in these tables. The deformation values for the nonlinear procedures given in these tables were developed from the experience and judgment of the project team, guided by available test results. The various experimental results referred to in the previous paragraphs are also used here to justify the deformation values given in Table 6-17 and 6-18.

The shear wall test results given in Figures C6-15 and C6-16 correspond to the first row of Table 6-17, and were used to develop the values of *a*, *b*, and *c* required to define the load versus deformation curve given in Figure 6-1(a) of the *Guidelines*. If it is assumed that yielding occurred at a hinge rotation of 0.0025 radians, both specimens reached a plastic rotation (inelastic rotation beyond the yield rotation) of 0.015 radians (value of *a*) without a significant loss in strength.

Because both of the tests referred to here were terminated before the shear wall specimen demonstrated a significant loss in strength, judgment is required to determine what plastic hinge rotations could be reasonably obtained and what residual strength the specimen would have at that deformation state. Both sets of researchers were reporting distress in the wall compression zones at the end of the tests, and the last deformation cycle in Figure C6-17 does show some drop in lateral load capacity. Thus, it was assumed that the plastic rotations could have increased to 0.020 radians (value of *b*), and the specimen could still have carried 75% (value of *c*) of its maximum loads.

The test results shown in Figures C6-18 and C6-19 were used to justify values in the third and seventh rows, respectively, of Table 6-17. Specifically, loading in the negative direction corresponded to the tabulated values. Again, if the yield rotation is taken to be approximately 0.0025 radians, the specimen in Figure C6-18 obtains a plastic rotation of 0.009 radians without an apparent loss in lateral load capacity, and could probably obtain a plastic rotation of 0.012 radians and still maintain 60% of its lateral load capacity. The results in Figure C6-19 indicate that this specimen did not have much deformation capacity beyond yield. However, as noted previously, these test results are for an isolated specimen; the shear wall behavior would be different if the wall was contained within a building structure with several lateral-load-resisting elements. Thus, it was assumed that the wall specimen could have obtained a plastic rotation of 0.003 radians without a significant loss in lateral load capacity. At higher rotations, the load capacity will quickly deteriorate. Thus, at a plastic rotation of 0.005 radians it was assumed that the lateral load capacity would have dropped to 25% of its maximum value.

For shear walls and wall segments controlled by shear, drift was selected as the appropriate deformation parameter, and the *d* and *e* parameters defined in Figure 6-1(b) of the *Guidelines* were selected as the appropriate measures of inelastic deformation.

Test results given in Figure C6-21 are for a shear wall specimen whose inelastic behavior was governed by shear. The web reinforcement ratio used in this specimen was approximately 0.0025, so these results should correspond to the entries in the first row of Table 6-18. Recalling the previous discussion of these test results, a lateral deflection of 10 mm corresponds to a 1.0% story drift. The test results indicate that the specimen could have been cycled at a maximum story drift of 0.75% (*d* value) without a significant loss in lateral load capacity. At the end of the test the specimen was cycled to story drifts of 2.0% (*e* value) and still maintained approximately 40% (*c* value) of its maximum lateral load capacity.

It should be noted that the test results in Figure C6-22 are for a specimen with a large web reinforcement ratio, so the failure of this specimen was due to sliding shear failure at the base of the structure. Thus, it is more appropriate to use the results given in Figure C6-21 for determining the values in Table 6-18.

Chord rotations were selected as the appropriate deformation parameter for shear wall coupling beams, and the backbone curve given in Figure 6-1(b) of the *Guidelines* was used to define the inelastic behavior of coupling beams. The test results shown in Figures C6-23 and C6-24 represent RC coupling beams whose inelastic behavior was governed by shear; these results correspond to rows two and four, respectively, of Part ii of Table 6-18. The results given in Figure C6-23 indicate that the specimen reached chord rotation angles of 0.012 radians (*d* value) in each direction without a significant decrease in lateral load capacity. Not many load cycles were completed for this specimen, but it probably could have maintained at least 30% (*c* value) of its maximum lateral load capacity at chord rotations of 0.020 (*e* value) in each direction.

The results shown in Figure C6-24 indicate that the specimen maintained its lateral load capacity at relatively large chord rotations, but the hysteretic response was very pinched. To account for the low stiffness of this member and its poor hysteresis response, *d* was set equal to 0.008 radians, *e* was set equal to 0.012 radians, and *c* was set equal to 25%.

The lateral load versus chord rotation test results for a shear wall coupling beam with diagonal reinforcement, which corresponds to the conditions for the last row in Table 6-17, are given in Figure C6-25. Again, this specimen was not subjected to many loading cycles, or to large levels of chord rotation, but the given test results indicate a very ductile response that is stable at large chord rotation values. Based on the given test results, *d* was selected to be 0.030 radians, *e* was selected to be 0.050 radians, and *c* was selected to be 0.80.

C6.8.2.5 Rehabilitation Measures

When strengthening or stiffening a shear wall, the designer is reminded to evaluate the strength and stiffness of floor diaphragms and their connections to the shear wall. Also, the strength and stiffness of the foundation supporting the shear wall must be evaluated. All connections between new and existing structures should satisfy the requirements in Section 6.4.6 of the *Guidelines*.

The addition of wall boundary elements to increase the flexural strength of a shear wall requires a careful evaluation of the ratio between the wall's shear strength and the increased shear forces required to develop the flexural strength of the wall. In several cases the wall shear strength will need to be increased to ensure that the shear wall will exhibit ductile flexural behavior if it is overloaded.

Confinement jackets may be added to shear wall boundaries to either increase the deformation capacity of the wall, or increase both the wall flexural strength and deformation capacity. In the latter case, the shear capacity of the wall must be checked as noted above. Research results have shown that effective confinement of wall boundaries can be achieved by the use of concrete jackets, steel jackets, or fiber wraps (Iglesias, 1987; Aguilar et al., 1989; Jirsa et al., 1989; Aboutaha et al., 1994; Katsumata et al., 1988; Priestley et al., 1992).

For shear walls that have a shear capacity less than the shear required to develop the flexural capacity of the wall, a designer may elect to reduce the flexural capacity of the wall. A decision to reduce the lateral load capacity of a structure should be carefully evaluated to be sure that the improved ductile behavior of the structure more than compensates for its reduced strength.

In shear critical walls where the designer does not want to reduce the flexural strength of the wall, the shear capacity of the wall can be enhanced by increasing the thickness of the web of the wall. The extra web thickness should be reinforced with horizontal and vertical steel. Before casting the new concrete, the surface of the existing wall should be roughened and dowels should be placed to ensure that the old and new concrete will work together. In lieu of increasing the wall thickness, recent research (Ehsani and Saadatmanesh, 1994) has shown that the addition of carbon fiber bands is effective in increasing the shear strength and stiffness of existing walls.

As discussed in Section 6.5 of the *Guidelines*, steel or reinforced confinement jackets can be used to increase the shear capacity and confinement in beams and columns. These same procedures are effective for improving the inelastic behavior of coupling beams and RC columns supporting discontinuous shear walls. Even though these members may not initially appear to be shear critical, their shear strength may decrease under reversed cyclic loading. The use of a confinement jacket will either prevent or at least significantly delay the decrease in the member's shear strength with cycling.

Even the addition of confinement jackets may not be sufficient to improve the response of an RC column supporting a discontinuous shear wall to a satisfactory level. In such cases, it may be necessary to significantly change the demands placed on those columns by changing the layout of the structure. Shear walls could be added at other locations in the structure, but a more effective means will be to add new elements below the discontinuous wall. One procedure is to add a concrete infill between the existing columns (Kahn and Hanson, 1979; Jirsa et al., 1989; Valluvan et al., 1994). A second procedure is to add steel bracing members between the columns (Bush et al., 1991; Goel and Lee, 1990). For both cases, the new members will need to be evaluated by the procedures given in the *Guidelines* for new construction.

C6.9 Precast Concrete Shear Walls

C6.9.1 Types of Precast Shear Walls

In the past, precast wall systems have seldom been used as primary lateral-load-resisting elements for structures located in high seismic risk zones of the United States. There has been a general belief that precast construction was inherently less ductile than monolithic construction, and thus should not be used for structures that may experience moderate or severe earthquake excitation during their service life.

In more modern seismic building codes, precast shear wall construction is permitted in high seismic risk zones if it can be shown by experiment or analysis that the lateral-load-resisting characteristics of the precast system are at least equal to those of a similar cast-in-place shear wall system. This design requirement has led to a type of precast shear wall construction known as cast-in-place emulation. For this design approach, the connections between the precast components are detailed such that inelastic action will occur away from the connections. Since the precast components can be reinforced and detailed similarly to monolithic walls, then the inelastic response of the precast system should be identical to that of a cast-in-place system. Although this emulation design approach may be effective and predictable, this approach has a tendency to undermine the cost-effectiveness of precast concrete systems.

As a result of the recent National Science Foundation-sponsored research program entitled PRESSS (PREcast Seismic Structural Systems) (Priestley, 1995), there is now some experimental and analytical evidence to indicate that precast structures that do not emulate monolithic cast-in-place construction may be used to resist severe earthquake loading. In this new design philosophy, known as "jointed construction," some of the joints between precast members are designed to deform inelastically under large lateral loads, thereby providing ductility and energy dissipation to the structural system. These ductile joints between precast elements may consist of both vertical and horizontal connections between panels.

Precast shear walls in several older structures cannot be classified as cast-in-place emulation because the joints were not designed to force all inelastic action away from the connection region. Also, these older precast walls would not satisfy the more modern definition of jointed construction because the connections were not designed with special elements intended to absorb energy in a stable ductile manner. This older type of jointed construction was not permitted in high seismic zones, and the designer will need to be careful when assessing its deformation capacity. For these older precast shear walls, continuity splices between the horizontal and vertical web reinforcement of the wall panels was normally obtained by a simple interconnection of the bars protruding from adjacent wall segments. Because the cast-in-place connections between panels are too short to satisfy the requirements for a tension lap splice, the bars may have been either hooked around each other to create a mechanical interlock, or fillet welded along their short lap length. The larger vertical bars commonly used along the vertical edges of a wall panel would have required special splicing hardware. A variety of proprietary rebar splice connectors have been used in older construction and may still be used in modern precast wall construction.

Tilt-up walls are considered to be a special case of jointed construction. The in-plane shear strength of these walls should be evaluated as a force-controlled action. Failure of the connection between the tilt-up wall and the roof diaphragm has been the most common type of failure observed for these types of structures during significant seismic loading. If that connection fails, the wall panel is subjected to out-of-plane forces and deformations that could cause it to collapse. Thus, the designer is cautioned to carefully check the connection between the wall and the roof diaphragm.

C6.9.2 Precast Concrete Shear Walls and Wall Segments

C6.9.2.1 General Modeling Considerations

The general analytical modeling considerations for precast concrete shear walls are very similar to those for monolithic cast-in-place shear walls. Therefore, the reader is referred to Section C6.8.2.1.

In addition to modeling the precast wall panels, the designer will need to include an analytical model to represent deformations in connections between the precast panels. Such connection models can only be avoided if the connections are designed and detailed to remain elastic, and all inelastic response of the precast wall system will take place in the precast panels.

C6.9.2.2 Stiffness for Analysis

The *Guidelines* offer two alternatives for including the stiffness of the connections between precast panels in the analytical model. One option would be to modify the analytical model used for the wall panels to represent the stiffness of the assembled wall panels and connections. The second option is to keep the same stiffness parameters as used for monolithic walls, but add a separate element to represent the stiffness of the connection.

A. Linear Static and Dynamic Procedures

No commentary is provided for this section.

B. Nonlinear Static and Dynamic Procedures

A general discussion of nonlinear procedures for shear walls and wall segments is given in Section C6.8.2.2B. Most of that discussion for monolithic concrete shear walls and wall segments is also applicable to precast walls and wall segments.

When using the basic load-deformation curves given in Figure 6-1, the deformation values (*x*-axis) are to be defined as either plastic hinge rotation or drifts, depending on whether the wall's (or wall segment's) inelastic response is governed by flexure or shear. Plastic hinge rotations are used where flexure governs the inelastic response for shear walls and wall segments. A sketch of the first story of a deformed shear wall governed by flexure is given in Figure 6-2. The length of a plastic hinging region in an RC member is typically defined to be somewhere between 0.5 and 1.0 times the effective flexural depth of the member. For RC members where shear deformations are significant, the plastic hinging length tends toward the upper end of this range, and vice versa. Therefore, for the shear walls the plastic hinging region will extend very close to, if not beyond, one story height of the member. In these cases it is appropriate to limit the length of the plastic hinging zone to one story height. For wall segments, which often have small length-to-depth ratios, the plastic hinging zone may extend to mid-length of the member. For those cases, the length of the plastic hinging zone is limited to one-half the length of the member.

For members whose inelastic response is controlled by shear, it is more appropriate to use drifts as the deformation value in Figure 6-1(b). For shear walls, this drift is actually the story drift as shown in Figures 6-3. For wall segments, the member drift is used.

For monolithic construction, values for the hinge rotation values *a* and *b*, described in Figure 6-1(a), are given in Table 6-17, and the drift values *d* and *e*, described in Figure 6-1(b), are given in Table 6-18. For cast-in-place emulation types of precast wall construction, the full tabulated values are used. For jointed construction, the tabulated values are to be reduced by 50%. This is a severe reduction, but the design engineer can use a smaller reduction if there is experimental evidence to support the use of higher values.

C6.9.2.3 Design Strengths

The discussion of the calculation of yield and nominal strengths given in Section C6.8.2.3 is applicable to precast shear walls and wall segments that are classified as cast-in-place emulation. For all types of jointed construction, the strength of the precast shear wall will be significantly affected by the strength of the connections. Thus, the connection strength must be evaluated as described in the *Guidelines*. Special consideration must be given to the type of splicing used for the reinforcement present in the connection. In many cases the strength of the splice will govern the strength of the connection.

C6.9.2.4 Acceptance Criteria

A. Linear Static and Dynamic Procedures

As previously stated, precast shear walls that emulate cast-in-place construction and wall elements with precast panels shall be evaluated by the same procedure as used for cast-in-place shear walls and wall elements. For jointed construction, the *m* values, which give a measure of a member’s ductility, shall be reduced to 50% of the values given in Tables 6-19 and 6-20. This severe reduction in the available ductility can be changed if the designer has access to experimental evidence that justifies a higher *m* value.

B. Nonlinear Static and Dynamic Procedures

Inelastic response is only acceptable for those actions listed in Tables 6-17 and 6-18. A detailed discussion of the deformation values given in these tables was presented in Section C6.8.2.4B. For jointed construction, the deformation values are reduced to 50% of the tabulated values because of uncertainty about the inelastic behavior of older versions of this type of construction.

C6.9.2.5 Rehabilitation Measures

As the *Guidelines* note, precast concrete shear walls may suffer from some of the same problems experienced by monolithic shear walls. Therefore, most of the rehabilitation measures described in Section 6.8.2.5 are applicable to precast shear walls.

Connections between precast panels and between the panels and the foundation offer an additional set of problems in precast walls. Most of the deficiencies in strength at the connections can be rehabilitated through the use of supplemental mechanical connectors or cast-in-place connections doweled into the adjacent members. Rather than add ductile supplemental connections, the designer should attempt to make the connections stronger than the adjacent panels, and thus force any inelastic behavior into those panels. The designer is cautioned to consider out-of-plane forces and deformations when designing and detailing supplemental panel-to-panel connections and panel-to-foundation or panel-to-floor diaphragm connections.

C6.10 Concrete Braced Frames

C6.10.1 Types of Concrete Braced Frames

Reinforced concrete braced frames are relatively uncommon in existing construction, and are seldom recommended for use as ductile earthquake resisting systems. They are sometimes used in the United States for wind-bracing systems, where inelastic response is not anticipated. Examples of concrete braced frames have been identified in other countries. These bracing systems may have provided necessary stiffness and strength that saved many concrete frames during the 1985 Mexico City earthquake, but there are also many examples of poor performance in the same systems during this earthquake. In general, these types of elements are not recommended for regions of moderate and high seismic activity.

C6.10.2 General Considerations in Analysis and Modeling

Braced frames resist lateral forces primarily through tension and compression in the beams, columns, and diagonal braces. Therefore, it is usually acceptable to model these frames as simple trusses. As with other reinforced concrete framing systems, the analysis model must recognize the possibility for failure along the length of the component (as in tension failure of reinforcement splices) or in connections.

C6.10.3 Stiffness for Analysis

C6.10.3.1 Linear Static and Dynamic Procedures

If the braced frame is modeled as a truss, it is acceptable for beams, columns, and braces to use the recommended axial stiffnesses for columns from Table 6-4. Joints may be modeled as being rigid.

C6.10.3.2 Nonlinear Static Procedure

The writers were unable to identify test data related to reinforced concrete braced frames. However, the braced-frame action of this element is expected to be similar in many regards to that for infilled frames modeled using the braced-frame analogy. Therefore, it is acceptable to use the general modeling parameters from Section 6.7.

C6.10.3.3 Nonlinear Dynamic Procedure

The writers were unable to identify test data related to reinforced concrete braced frames. The analyst must use engineering judgment in establishing the analysis model for the NDP.

C6.10.4 Design Strengths

The general procedures of ACI 318 for calculation of compressive and tensile strength are applicable, subject to the guidelines of Section 6.4.2.

C6.10.5 Acceptance Criteria

Existing construction of concrete braced frames is unlikely to contain details necessary for ductile response. These details include: (1) in compression members, adequately detailed transverse reinforcement to confine concrete and restrain longitudinal reinforcement from buckling; (2) in tension members, reinforcement splices having strength sufficient to develop post-yield tension behavior in longitudinal reinforcement; and (3) in connections, adequate anchorage for longitudinal reinforcement. Where these details are not provided, actions should be defined as being force-controlled.

C6.10.6 Rehabilitation Measures

Rehabilitation measures that are likely to improve response of existing concrete braced frames include the following:

- Jacketing of existing components, using steel, reinforced concrete, or composites to improve continuity and ductility
- Various measures to improve performance of lap splices, including chipping cover concrete and welding
- Removal of the diagonal bracing, leaving a moment-resisting frame, which must then be checked according to procedures in Section 6.5
- Addition of steel braces, walls, buttresses, or other stiff elements to control lateral drift and protect the existing braced frame
- Infilling of the braced frame with reinforced concrete, either with the brace in place, or after removal of the brace
- Modification of the structural system through such techniques as seismic isolation

C6.11 Concrete Diaphragms

Cast-in-place diaphragms have had a relatively good performance record in worldwide earthquakes when the configuration was not irregular and when the length-to-width ratio was relatively small (less than three to one). Thin concrete slabs associated with one-way beam and joist systems are limited in diaphragm shear capacity and become more suspect as the length-to-width ratio increases.

C6.11.1 Components of Concrete Diaphragms

No commentary is provided for this section.

C6.11.2 Analysis, Modeling, and Acceptance Criteria

No commentary is provided for this section.

C6.11.3 Rehabilitation Measures

No commentary is provided for this section.

C6.12 Precast Concrete Diaphragms

C6.12.1 Components of Precast Concrete Diaphragms

Precast concrete diaphragms contain a variety of different components that have been used at different times and in different geographic regions. The precast industry first began to produce components in the early 1950s. Many of the first components were reinforced with mild steel and utilized concrete strengths in the range of 3000 psi. Rectangular beam, inverted tee beam, L beam, column, channel shape, slab, double tee, and single tee components (reinforced, prestressed, and post-tensioned) were available in most regions of the United States by 1960. The connections utilized are generally brittle, with varying amounts of limited ductility. Concrete strengths were then routinely specified at 6000 psi or more to facilitate quick turnaround of casting facilities. Only a small percentage of these systems were designed with ultimate level seismic forces in mind. Diaphragms rarely had a composite topping slab poured on them if they were at the roof level, but most floor systems do have poured composite topping slabs.

Topped diaphragms may have the following seismic deficiencies:

- Inadequate topping thickness and general reinforcement
- Brittle connections between components
- Excessive diaphragm length-to-width rations
- Little or no chord/connector steel
- Inadequate shear transfer capacity at boundaries
- Inadequate connections and bearing length of components at supports
- Corrosion of connections

Whether or not the diaphragms were initially designed for seismic forces, the performance of precast diaphragms during the 1994 Northridge earthquake demonstrated that the following items should be reviewed as part of an evaluation/rehabilitation program.

- **Diaphragm Rigidity**. Diaphragms experience relatively large displacements due to the yielding of reinforcing used as temperature steel in the deck, the yielding of collectors and chords, and, in some cases, the long length-to-depth ratios. Brittle failure of individual component-to-component connections will also contribute to greater-than-expected displacements. Diaphragm displacements may be much larger than associated shear wall drifts; therefore, the distribution of seismic forces will be much different than that determined from a rigid diaphragm assumption. Columns assumed to be non-seismic-resisting have failed because of the displacements that they experienced.

- **Complete Load Paths.** The joints or seams between spanning members and the joints along the ends of such members are generally covered with thin concrete overlays and are often lightly reinforced. The structural response of the diaphragm may be strongly influenced by the action along these seams. Critical sections may require reinforcement.

- **Collector Design.** The chord forces and diaphragm collector forces should be designed to have limited yielding, or designed with confinement steel similar to ductile axial column members. Initial tension yielding causes a situation where subsequent cyclic compression forces may buckle the reinforcement. This type of failure was observed following the 1994 Northridge, California earthquake (Corley, 1996). Additionally, it was observed that shear wall/collector connections failed. These failures could be the primary collapse mechanism, or could be secondary to other factors. It is clear that collector-to-shear-wall connections are critical; they should be designed for ductility where possible, with strength commensurate with the ductility assumed. The effects of cyclic tension/compression actions should be recognized in the design of confinement steel. Also, it is important to recognize the effects of shear wall rocking and rotation on the collector connection. This action, along with the fracture potential of bulking bars, has not generally been recognized.

- **Vertical Acceleration.** Gravity-loaded long-span precast members may be vulnerable to vertical accelerations at sites close to fault systems. Corley states that “A combination of gravity load and vertical acceleration may have caused failure of some inverted tees." Other observers have noted this possibility with respect to different members.

C6.12.2 Analysis, Modeling, and Acceptance Criteria

No commentary is provided for this section.

C6.12.3 Rehabilitation Measures

Rehabilitation measures for precast concrete diaphragms are difficult and, in many cases, expensive. The installation of new shear walls or rigid braces can be very effective, in that demands on components, elements, and connections can be greatly reduced. Experience with other techniques is limited. In the case of untopped roof diaphragms, removal of the precast concrete deck should be considered. The installation of a modern seismic-resisting system may be economical in some cases.

C6.13 Concrete Foundation Elements

C6.13.1 Types of Concrete Foundations

This section provides guidelines primarily for seismic analysis, evaluation, and enhancement of concrete foundation elements that occur in buildings with structural frames, or concrete or masonry shear and bearing walls. Selected portions of these guidelines may also be applicable to other structural systems and to foundation elements of other structural materials (e.g., timber or steel piles).

C6.13.2 Analysis of Existing Foundations

The simplifying assumptions regarding the base conditions for the analytical model are similar to those required for gravity load analyses. The procedures described for more rigorous analyses are considered to provide more rational representation of the soil-structure and soil-pile interaction under lateral loading. These more rigorous procedures are therefore recommended to provide a higher confidence level for the more demanding Performance Levels. Since the net effect of these procedures is generally to reduce stresses in the building, but to increase displacements, these procedures may make it possible to accept an otherwise deficient stiff building if the resulting displacements are within allowable limits.

C6.13.3 Evaluation of Existing Condition

In the absence of dependable construction drawings, confirmation of the size and detailing of existing foundations may not be possible without resorting to invasive procedures. For larger or important buildings, limited demolition of selected foundations may be necessary where adequate construction documentation is not available. Drawings are more likely to be available for buildings with deep foundations. For most buildings with shallow foundations, if drawings are not available, selected exposure of representative footings may be required to establish size and depth. Conservative assumptions regarding reinforcement may be made, considering code requirements and local practice at the time of design. In case of doubt, it can be assumed that the foundation elements were designed adequately to resist the actual gravity loads to which the building has been subjected, although the actual factor of safety will still be in doubt.

Because of the difficulty associated with the exposure and repair of potential seismic damage to foundations, current preferred practice is to preclude damage by ensuring the yielding occurs in the columns or walls above the foundation. For this reason, it is stipulated that the existing foundation elements be evaluated with the smaller of the unreduced design forces or the forces based upon the capacity of the supported columns or walls.

C6.13.4 Rehabilitation Measures

The seismic rehabilitation or enhancement of foundation elements in existing buildings is generally an expensive and disruptive process. Limited accessibility, and the difficulty and risks associated with strengthening existing foundation elements that are supporting the building gravity loads, often lead the engineer to search for a more cost-effective solution. In many cases, when analysis indicates that existing foundation elements may be subjected to excessive seismic force, the deficiency may be reduced or mitigated by new vertical lateral-force-resisting elements (e.g., bracing or shear walls) that will divert the seismic forces to new foundation elements or to other lightly loaded existing elements. While the strengthening techniques described in this chapter are considered to be practical and feasible, the designer is encouraged to develop and evaluate alternative mitigation measures that may be more cost-effective for the building owner. Accepting performance that allows for permanent soil deformation below the footings will reduce the rehabilitation cost.

C6.13.4.1 Rehabilitation Measures for Shallow Foundations

Spread footings generally include individual column footings and continuous strip footings supporting wall loads. Existing small or lightly loaded column footings may be unreinforced; larger and heavier loading footings will have a horizontal curtain of steel near the bottom of the footing. Strip footings are generally composed of square or rectangular continuous footings designed so as to not exceed the allowable soil bearing pressures. A concrete stem wall may extend above the footing to support the wall above and may have a ledge to support the floor slab. The footing and the stem wall may be reinforced, or may have a few continuous horizontal bars at the bottom of the footing and one or two horizontal bars at the top of the stem wall. More recent or better-designed existing wall footings will have vertical reinforcement in one or both vertical faces of the stem wall.

A reinforced concrete shear wall or a concrete frame with an infilled concrete or masonry wall may have a combination footing, consisting of a strip footing under the wall and a monolithic spread footing at each end under the columns or boundary members of the shear wall.

Concrete mats are large footings that support a number of columns and walls and rely on the flexural stiffness of the mat to distribute the supported loads to the soil, or the piles or piers. Mats will usually have a horizontal curtain of reinforcement at the bottom and an additional curtain at the top of the mat; they may or may not have any distributed vertical reinforcement.

If the design seismic forces in a footing result in load combinations that exceed the deformation limits or the allowable soil pressure, the existing footing must be enlarged, or additional lateral-load-resisting elements may be added, to reduce the soil bearing pressure under the footing to allowable levels.

An existing column footing may be enlarged by a lateral addition if proper care is taken to resist the resulting shears and moments. The original footing will continue to support the load at the time of extension, and the extended footing will participate in the support of the subsequent loads. If the existing footing is founded on poor soil but more competent bearing strata occur at reasonable depth, it may be feasible to convert the spread footing into a pier-supported footing by drilling through the footing and providing cast-in-place reinforced concrete piers under the footing. If the existing footing has inadequate shear or moment capacity for the resulting forces from the new piers, the capacity may be enhanced by new concrete to increase the depth of the footing.

If the seismic rehabilitation criteria result in overturning moments that cause uplift in an existing spread footing, tension hold-downs can be provided. Because of their slenderness, the hold-downs may be assumed to resist tension only. Reversed movements from these tension ties may require the addition of horizontal reinforcement in new concrete fill at the top of the footing. The design engineer must consider whether uplift or rocking will cause unacceptable damage in the building.

A typical perimeter wall footing may also be strengthened by procedures similar to those described above for individual column footings. An alternative strengthening procedure commonly utilized for continuous footings is underpinning. Underpinning is generally accomplished by progressive incremental excavation under an existing footing, and replacement of the excavated material with new concrete to provide a larger footing. The lateral extension and the depth of the underpinning are generally selected so that the concrete may be assumed to be in compression and reinforcement of the underpinning is not required. Underpinning may also be used to provide tension hold-downs for an existing wall footing subject to uplift forces from seismic overturning moments. A pair of drilled and grouted tension ties is provided at each end of the wall footing and anchored into a new cap that is constructed by underpinning the end of the wall. If significant tensile forces are to be resisted, it may be necessary to provide concrete wing walls on either side of the wall, extending vertically from the new cap to a length adequate to transfer the tensile uplift force from the existing wall by dowels and shear friction.

Concrete mats are typically analyzed as isotropic plates with concentrated loads on an elastic foundation, and are sensitive to the assumed subgrade modulus for the soil. Because of the difficulty and cost associated with strengthening an existing mat foundation, it is recommended that, if any of the above deficiencies are identified, the assumed soil properties be reviewed and additional geotechnical investigations be made to determine if more favorable properties can be justified. Similarly, if the analysis indicates localized soil bearing pressures that exceed allowable values, the geotechnical consultants should be asked to review the allowable values with the actual conditions of loading and lateral confinement. The engineer and owner should also consider whether permanent soil deformation is acceptable.

If it is feasible to increase the depth of the mat with a reinforced concrete overlay, both the flexural reinforcement and vertical shear capacities may be enhanced. This may be the only retrofit procedure feasible for a deficient mat. A practical alternative to retrofitting would be to evaluate the consequences of allowing limited yielding of the reinforcement and/or cracking of the concrete under the design seismic loading. This evaluation can be performed with available nonlinear analysis computer codes, or can be approximated with linear elastic analyses by progressively “softening” the yielding elements.

If the soils under the mat are found to be compressible or otherwise unsuitable, pilings driven through drilled holes in the mat foundation to competent soil strata can be used. This is sometimes employed in new construction to offset an abrupt variation in the soil profile under the mat. In existing buildings, care must be exercised in design and construction so as not to damage the existing mat reinforcement, and deformation compatibility must be maintained under the design loadings without overstressing the mat.

C6.13.4.2 Rehabilitation Measures for Deep Foundations

Concrete piles or piers are generally surmounted by a concrete cap that supports the base of a column or wall. A concrete pedestal is sometimes utilized to raise the base of the column to a more convenient elevation and/or to achieve a better distribution of loads to the pile or pier cap.

Concrete piles may be precast, or precast and prestressed, and are driven with or without predrilling of the soil. The piles are considered to be point bearing if they are driven to “refusal” in rock or other hard material, and as friction piles if the loads are transferred to soil by cohesion or friction.

Concrete piers are generally designed as reinforced concrete columns, and constructed by placing the reinforcement and concrete in either open or cased drilled holes. Proprietary systems are in use that utilize thin metal shells driven with a steel mandrel in lieu of drilling.

Anchorage of the piles or piers into the cap may vary from simple embedment of several inches without dowels to complete development of the vertical reinforcement into the cap. Pile and pier caps are designed to resist the moments and shears from the pile or pier reactions. Typically, the caps are designed with sufficient depth to resist the shear without reinforcement, and a curtain of horizontal reinforcement near the bottom of the cap is designed to resist the flexural moments. For severe pile loads, or when the depth of the cap is limited, vertical shear reinforcement may be required, and a horizontal curtain of reinforcement may be provided near the top of the cap to anchor the shear reinforcement.

If the existing piles or piers are found to be deficient in vertical load capacity, the capacity can be increased by adding additional piles or piers. If the new elements are added with an extension of the existing cap, the existing cap may have to be strengthened to resist the moment and shear from the additional piles or piers. The new piles or piers will only participate in the resistance of vertical loads subsequent to their construction. In some cases, where the existing foundation is judged to be seriously deficient, it may be cost-effective to provide temporary shoring to permit removal and complete replacement of the foundation.

A common problem in the seismic rehabilitation of existing buildings is uplift on the existing foundation. If the existing piles or piers and/or their anchorages to the caps are inadequate for the design uplift forces, new elements can be provided to resist the tensile uplift forces. If new piles or piers are required to resist the vertical compressive forces, it may be feasible to design these new elements and to strengthen the cap to resist the uplift forces. If new elements are not required for the compressive forces, it may be possible to provide the necessary uplift capacity by means of hold-downs drilled through the existing caps. The hold-downs consist of high-tensile-strength steel rods or strands, anchored by grouting in firm material at the bottom and in the concrete cap at the top. The existing caps need to be investigated and strengthened, if necessary, for the reverse flexural moment resulting from the uplift forces.

Inadequate moment capacity of the existing cap reinforcement can be improved by adding additional concrete to increase the depth of the existing cap. This has the effect of increasing the effective depth of the cap, and thus reducing the tensile stress in the existing reinforcement. The top of the existing cap should be roughened and provided with shear keys or dowels to resist the horizontal shear at the interface. The additional depth that can be provided may be limited to functional restrictions (e.g., interference with the floor slab), or the additional weight that can be supported by the existing piles or piers. It should be noted that increasing the depth of the cap may decrease the effective length of the column above, and require a revision in the relative rigidity calculations for distribution of lateral loads. Additionally, it should be noted that this procedure may not be applicable to caps supporting columns that are assumed to be pinned at their base, since the additional cap depth may result in undesirable fixity of the column base.

Where the moments in the existing columns are large enough to cause uplift in the piles or piers, reversed moments will occur in the cap, requiring tensile reinforcement near the top surface. If this reinforcement is absent or deficient, the required reinforcement can be provided in a new concrete overlay to the existing cap. To improve the effectiveness of the new reinforcement, it may be necessary to drill and grout some of the bars through the existing column. If this is not feasible, the effective transfer of tensile forces to the new reinforcement must be investigated by the strut and tie method, or other rational procedures. Alternatively, temporary shoring of the column loads can be provided so that the existing column reinforcement can be exposed and the new horizontal reinforcement placed effectively. As discussed in the previous paragraph, if the additional depth of cap significantly reduces the effective length of the column, the distribution of the lateral load shears may have to be reevaluated.

Inadequate vertical shear capacity in the existing caps can also be improved by providing additional depth to the caps. Since it is not considered feasible to provide new vertical shear reinforcement in an existing cap, if the necessary capacity cannot be obtained by increasing the depth of the cap, the only available alternatives may be to remove and replace the existing cap with an appropriate new cap, or to provide new lateral-load-resisting elements (e.g., shear walls or braced frames) that will reduce the forces to be resisted by the existing cap to allowable levels.

If the vertical reinforcement in the existing piles or piers is adequately developed into the caps, then the pile or pier will provide lateral force resistance by flexural bending. The lateral load capacity of these elements can be approximated by assuming fixity at a depth below the cap equal to about ten diameters for very soft soils and five diameters for very firm soils. The moment or shear capacity can then be calculated assuming full or partial fixity at the cap. The pile or pier capacity is compared with the portion of the design lateral load to be resisted by the piles or piers, as determined by consideration of deformation compatibility with the portion resisted by passive pressure of the soil on the cap. If the total effective capacity of the piles or piers and the cap is inadequate, the practical alternatives are to enhance the passive pressure capacity of the cap; to remove and replace the existing cap with or without the addition of new piles or piers; or to reduce the lateral forces on the existing foundation elements by providing additional resisting elements.

Pile and pier foundations resist lateral forces by means of passive soil pressure on the caps or by bending of the piles or piers. If the anchorage of the existing piles or piers to the caps is inadequate or questionable in regard to development of moments in the piles or piers, passive soil pressure on the caps may constitute the principal lateral load resistance of the foundation. The total resisting capacity of the foundation system will include passive pressure on tie beams and perimeter walls extending below grade. In order to mobilize the total resisting capacity of the existing foundation system, it is important that all of the resisting elements be properly interconnected. This connection may be accomplished by a competent slab at or near the top of the caps, or by adequate tie beams to affect the distribution. If the existing total capacity is inadequate, the alternatives include enhancing the passive resistance of the soil; increasing the contact areas of the caps, tie beams, and perimeter walls; or a combination of these alternatives.

The passive resistance of the soil can be enhanced by a number of techniques, such as compaction and/or intrusion grouting with appropriate chemicals or soil/ cement mixtures, as described in Chapter 4.

C6.14 Definitions

No commentary is provided for this section.

C6.15 Symbols

No commentary is provided for this section.

C6.16 References

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