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STRUCTURAL CONCEPTS AND DETAILS FOR SEISMIC DESIGN

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STRUCTURAL CONCEPTS AND DETAILS FOR SEISMIC DESIGN

Prepared for

The Office of the Assistant Secretary for
Environment, Safety and Health
Office of Safety Appraisals
United States Department of Energy

Under Contract to

Lawrence Livermore National Laboratory
University of California
Livermore, California

September, 1991

EQE ENGINEERING AND DESIGN

MASTER

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FOREWORD

This document was prepared under the auspices of Lawrence Livermore National Laboratory (LLNL) under contract to the Assistant Secretary for Environment, Safety and Health, Office of Safety Appraisals of the U.S. Department of Energy (DOE). The LLNL contract with EQE Engineering and Design, a division of EQE International, was P.O. B089412. Preparation of the document was a team effort with a project team consisting of engineers from EQE Engineering and Design and an independent review team. Team members and their affiliations are listed below:

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This document is based primarily on the technical requirements of UCRL-15910 (1990), the Uniform Building Code (1991), and the SEAOC "Blue Book" design recommendations (1990).

This document has been issued for trial use and comment within the DOE community. Additionally, it will be sent to the Seismology Committee of the Structural Engineers Association of California (SEAOC) for their review and comment. Comments from all parties should be sent to Mr. Robert C. Murray by September 1, 1992. Please use the form at the end of this report.

Cover: Overturned cement silo at Copsa Cement Plant, Mexaco, 1985 (M8.1).

ABSTRACT

This manual discusses building and building component behavior during earthquakes, and provides suggested details for seismic resistance which have been shown by experience to provide adequate performance during earthquakes. Special design and construction practices are also described which, although they might be common in some high-seismic regions, may not be common in low and moderate seismic-hazard regions of the United States. Special attention is given to describing the level of detailing appropriate for each seismic region. The UBC seismic criteria for all seismic zones is carefully examined, and many examples of connection details are given. The general scope of discussion is limited to materials and construction types common to Department of Energy (DOE) sites. Although the manual is primarily written for professional engineers engaged in performing seismic-resistant design for DOE facilities, the first two chapters, plus the introductory sections of succeeding chapters, contain descriptions which are also directed toward project engineers who authorize, review, or supervise the design and construction of DOE facilities.

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The Art of Structural Engineering??

Structural Engineering is the art of molding materials, we do not wholly understand, into shapes we cannot precisely analyze, so as to withstand forces we cannot really assess, in such a way that the community-at-large has no reason to suspect the extent of our ignorance.

Anon, P.E.

INTRODUCTION

1.1 INTRODUCTION

Building codes specify minimum earthquake resistance requirements in terms of design lateral forces and allowable material stresses. However, achieving good seismic performance and safety is equally a function of the types of structural systems and the materials selected, the details of connections used throughout the structure, and the quality of construction. A building seismic system is an integrated system of resisting elements. Every element and connection along the seismic resisting load path must be adequate to maintain the integrity of the system.

Historically, unsatisfactory structural performance during earthquakes has been attributed less often to "underdesign," in terms of the code under which the structure was designed, and more often to the following: 1) poor building configuration, 2) the presence of "weak links" in the structure, 3) inability of the materials of construction to absorb repeated shock or overload, 4) unanticipated interactions between adjacent structures, and 5) poor construction (materials and/or practices).

All of these factors are within the control of the design and construction team. Irrespective of actual load levels and material stresses used in seismic design, a highly survivable structure can be achieved through application of the principles presented herein.

1.2 PURPOSE

The purpose of this document is to review fundamental aspects of seismic behavior, and to present good structural detailing practices based on current technology and recent earthquake experience data. The general scope of discussion is limited to materials and construction types common to low-rise office buildings, industrial buildings or production facilities typically found at Department of Energy (DOE) sites.

Although the manual is primarily written for professional engineers engaged in performing seismic-resistant design for DOE facilities, the first two chapters, plus the introductory sections of succeeding chapters, contain descriptions which are also directed toward project engineers who authorize, review, or supervise the design and construction of DOE facilities.

This manual was produced as a part of the DOE Natural Phenomena Hazards Project. It is intended to be used as a companion document to the other manuals produced by this project, as described below.

1.2.1 DOE NATURAL PHENOMENA HAZARDS PROJECT

The DOE natural phenomena hazards project is under the direction of Lawrence Livermore National Laboratory (LLNL), under contract to the Assistant Secretary for Environment, Safety and Health, Office of Safety Appraisals (OSA) of the U.S. Department of Energy (DOE). The overall goal of the project is to provide guidance and criteria for design of new facilities and for evaluation, modification, or upgrade of existing facilities such that DOE facilities can safely withstand natural events such as earthquakes, extreme winds, and flooding.

Figure 1-1 shows the general steps of the natural phenomena hazards project. A principal document of this project is a manual entitled UCRL-15910, "Design and Evaluation Guidelines for Department of Energy Facilities Subjected to Natural Phenomena Hazards." UCRL-15910 provides evaluation procedures, plus consistent-performance design criteria, for the various natural phenomena hazards affecting each DOE site. Other, supplementary manuals developed for the natural phenomena hazards project suggest design details and practices that represent current technology, and in many cases have been observed to perform well during past natural hazard events.

Basic criteria for the design and evaluation of DOE facilities are provided by the General Design Criteria Manual, DOE Order 6430.1A (Reference 1-1). Design Basis events due to natural phenomena hazards, as defined in 6430.1A, include earthquakes (DBE), winds (DBW), tornadoes (DBT), and floods (DBFL). DOE 6430.1A references UCRL-15910 as an acceptable approach to use for the design and evaluation of facilities for these hazards.

The manuals produced to describe and illustrate good design practice currently consist of the following documents:

UCRL 21131	"Structural Details for Wind Design"
UCRL 15714	"Suspended Ceiling System Survey and Seismic Bracing Recommendations"
UCRL 15815	"Practical Equipment Seismic Upgrade and Strengthening Guidelines"
UCRL CR-106554	"Structural Concepts and Details for Seismic Design" (this document)

FIGURE 1-1: GENERAL STEPS OF THE
NATURAL PHENOMENA HAZARDS PROGRAM

After Reference 1-7

1.3 SCOPE OF THIS MANUAL

1.3.1 WHAT IS INCLUDED

This manual discusses structure and component behavior during earthquakes, and provides suggested details for seismic resistance which have been shown by experience to provide adequate performance during earthquakes. Special design and construction practices are also described which, though common in some high-seismic regions, may not be common in low and moderate seismic-hazard regions of the United States.

Where possible, this manual has been organized to follow the same general organization, and use the same terminology, as the Uniform Building Code (UBC).

Chapter 1 is an introductory chapter which describes the function, scope and limitations of the manual. Chapter 2 describes basic seismic concepts and provides some reasons why certain types of buildings are inherently more seismic-resistant than others. Chapters 3 through 6 cover wall systems; moment frame and bracing systems; diaphragms; and foundations, respectively. Chapter 7 describes design aspects of partition systems and exterior nonstructural cladding. Finally, Chapter 8 gives guidance for building addition or retrofit projects and for unusual design situations. References listed at the end of each chapter contain further sources of information for each topic.

1.3.2 WHAT IS NOT INCLUDED

This manual is a guide; not a building code. Every engineer engaged in the design of structures is expected to have, understand, and use copies of all codes and criteria pertinent to each project. Also, the use of this manual does not provide a substitute for a seismic peer review of project designs. UCRL-15910 requires a seismic peer review for the design of most types of DOE facilities .

Details presented in this document should not be interpreted as "pre-designed." Each suggested practice or detail must be specifically designed in accordance with the particular criteria being used for overall design of the facility. Design involves determining loads, evaluating materials, and proportioning the members and connections in order to achieve satisfactory performance.

Seismic topics excluded from this manual include discussions of the methods for the determination of load criteria, response evaluation, and acceptance criteria (already provided in UCRL-15910), and any discussions of the geotechnical aspects of seismic design, including such effects as liquefaction, compaction, and slope failure.

Construction types which have been specifically excluded from this manual include wood design (not common at DOE facilities), post-tensioned concrete design, details appropriate for buildings over three stories in height, and details for unusual types of construction, such as base-isolation systems.

1.4 OTHER SOURCES OF INFORMATION

Seismic design practice is a topic which has sparked considerable interest among many professional engineers. Consequently, many excellent references exist which describe many aspects of seismic detailing in great detail. This manual is only a summary of key aspects of many of them, and is intended to highlight the types of details and code issues which DOE facility designers may commonly confront. Additional information for each topic is contained in the references at the end of each chapter. The interested reader is particularly directed towards the sources of information in the references listed below.

REFERENCES

- 1-1 U.S. Department of Energy, General Design Criteria, DOE Order 6430.1A, Washington, D.C., 1987 (draft).
- 1-2 UBC, 1991: Uniform Building Code, 1991 Edition, International Conference of Building Officials (ICBO).
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- 1-6 Stratta, J.L., 1987: Manual of Seismic Design, Prentice-Hall, Inc., Englewood Cliffs, NJ.
- 1-7 Kennedy et.al., 1990: Design and Evaluation Guidelines for Department of Energy Facilities Subjected to Natural Phenomena Hazards, Lawrence Livermore National Laboratory, Report UCRL-15910.

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SEISMIC CONCEPTS

This chapter introduces basic concepts in seismic ground motion, structure response and seismic design. Section 2.1 addresses the nature of seismic behavior, and how the building code approaches the seismic design problem. Section 2.2 describes basic types of structural systems and their components. The remaining sections of this chapter describe various techniques and concepts used in seismic design: Section 2.3 describes the "Load Path" concept; Section 2.4 presents various aspects of structure configuration; Section 2.5 discusses ductility; and Section 2.6 describes drift and deflection considerations.

2.1 EARTHQUAKES AND BUILDING CODES

This section presents an overview of seismic ground motion effects, basic structural dynamics, and building code philosophy. Section 2.1.1 is a general description of structure response to ground motion. Section 2.1.2 describes how seismic motions are characterized and used for design. Section 2.1.3 is a discussion of the vibration characteristics of structures. Sections 2.1.4 and 2.1.5 respectively describe the building code philosophy and its approach to specifying lateral force requirements. Finally, Section 2.1.6 discusses the parameters which relate to the capability of structures to resist excessive damage or loss of function.

2.1.1 STRUCTURE RESPONSE TO GROUND MOTIONS

During an earthquake, the ground beneath a structure or equipment item moves erratically as shown in Figure 2-1. As a result, the structure or equipment item is subjected to acceleration, which induces internal forces in proportion to its mass. Structural system stiffness and mass properties combine to resist this movement in the following general manner: stiff structures tend to follow the ground movement closely, while flexible structures do not. Flexible structures can experience amplified displacements or may remain relatively motionless as the ground moves beneath the structure.

Fig. 2-1: Seismic Motion

For most structures, seismic response will consist primarily of swaying motions, and accelerations within the structure will tend to increase with height. Also, very tall structures may experience especially high accelerations (called "whipping") at upper stories; and highly irregular structures may exhibit opposing motions within parts and appendages. Accelerations within structures may be either greater or less than those experienced at ground level, and are a function of the degree to which the structure is "in tune" with cyclic ground motions.

After the ground motion subsides, the structure will either return to its original position and shape, or it may be damaged and permanently displaced. If the elastic strengths of the primary structural and soil-foundation systems are not greatly exceeded during shaking, then the structure will return to pre-earthquake conditions. If major portions of the overall structural system undergo significant inelastic response, then observable structural damage and permanent displacement are likely. Figures 2-2 and 2-3 show examples of structural damage due to excessive inelastic response, and structure failure due to liquefaction (i.e. failure within the supporting soil system), respectively.

Within the structure, loose objects may fall, be damaged, or displaced, as shown in Figure 2-4. Structural components which are easily excited by shaking, or which are too stiff or brittle to comply with structure motions, may be bent, broken, or torn from their connections.

2.1.2 TIME HISTORIES AND RESPONSE SPECTRA

Earthquake motion records, or *time histories*, consist of timed recordings of displacements, velocities, or accelerations, as shown in Figure 2-5. Separate time histories are often recorded for each of two orthogonal horizontal directions (e.g. north-south and east-west) and the vertical direction. These records are used to study the characteristics of earthquake motion and structure response. However, time histories are not commonly used for design due to significant computational demands. Instead, response spectra are often used to reliably and cost-effectively assess dynamic response to earthquake motions.

A response spectrum, as shown in Figure 2-6 for the 1940 El Centro, California earthquake, is a graphical representation of maximum structural response to a particular earthquake. Structural response can be shown in terms of displacement, velocity and acceleration, or more commonly, acceleration alone. Figure 2-6 is a tripartite plot in which structural response in terms of displacement, velocity, and acceleration are simultaneously shown. Note that acceleration response spectra are generally most useful to structural engineers because the maximum seismic force experienced by a structural system is a product of the mass of the system and the acceleration response. Each point of the response spectrum represents the maximum response of a single degree-of-freedom (SDOF) system with a

Fig. 2-2, Top:
Structural Damage
Source:
EQE Files, Mexico, 1985

Fig. 2-3, Center:
Foundation Failure
Source:
EQE Files, Philippines, 1990

Fig. 2-4, Bottom:
Damage to Building
Contents
Source:
EQE Files, Armenia, 1988

specified "period of vibration" (discussed below). Note that, though each point on the response spectrum represents the peak response of a particular SDOF system, the actual time when the maximum response occurs may be different for each point.

Figure 2-7(a) shows a simple structural frame, which is commonly represented as a SDOF system for lateral motions. In its simplest form, a SDOF system consists of a simple spring/mass oscillator, as shown in Figure 2-7(b). If the SDOF system is pulled and released, the amount of time it takes (in seconds) for one complete oscillation is called the *fundamental period* of vibration (the inverse is the fundamental frequency, f). Expressed mathematically, the Period, T , is equal to:

$$T = \frac{1}{2\pi} \sqrt{\frac{M}{K}} = \frac{1}{f}$$

Where:

M = Mass = Weight/acceleration of gravity

K = Stiffness

f = circular frequency, cycles/second

Fig. 2-7: (a) Rigid Frame, (b) SDOF Representation

The dynamics of structural systems, including period, mode shape and damping, will be further discussed in the next section.

When response spectra for different earthquake time histories and varying site conditions (i.e. soil profiles) are superimposed, it becomes apparent that soil conditions are a dominant factor in distinguishing among spectra shapes and magnitudes. Figure 2-8 shows envelopes of response spectra from numerous earthquake records for four different soil types. Enveloping or averaging numerous spectra in this manner is common practice for establishing response spectra for design purposes. In fact, the response spectra shapes that are presented in the Uniform Building Code (UBC) are established in the same manner as Figure 2-8.

2.1.3 MODE SHAPES, PERIOD AND DAMPING

Seismic design procedures are based upon the mass, stiffness and damping properties within a structural system, in conjunction with the characteristic modes of vibration. All structural systems have modes of vibration, and each mode has a unique shape, period and damping. The essence of structural dynamics as applied to seismic analysis is the

determination of how these mode shapes combine to form the total response of the structure to ground motion input.

The period and mode of vibration can be illustrated by returning to the SDOF oscillator in Figure 2-7(b). As discussed earlier, the period is the time for one full oscillation after the system is set in motion. The "mode shape" (i.e. the configuration of the system in motion) is simply the one-dimensional translation of the spring alternating in tension and compression. The SDOF oscillator only has one mode as it has only one degree-of-freedom (translation). In addition to mass and stiffness, oscillating systems also possess damping properties. After the oscillator in Figure 2-7(b) is set in motion, the motion slowly diminishes with each cycle, and eventually it stops. This is due to energy being dissipated by the system.

Just as in the example of the simple oscillator, if the frame of Figure 2-7(a) is set to motion, its response will also diminish with each cycle due to damping effects. Damping for the idealized structure will primarily be due to elastic strain energy expended as the spring stretches and contracts. However, for the real structure, damping occurs as all structural elements (beams, columns, floors, etc.) undergo strain, and experience friction at interfaces such as connections. There are also damping contributions from non-structural items as well. For example, the friction and strains that occur as building cladding, partitions, and utilities (piping, HVAC, etc.) deform and sway also damp overall structural response. Significant damping also occurs if the structure experiences damage and associated energy dissipation in the form of yielding, buckling, cracking, spalling, and impact with adjacent elements.

The effect of damping is significant and must be reasonably approximated in order to reliably predict seismic response. Figure 2-6 shows response spectra plotted for several values of damping. As can be seen, the higher the damping value, the lower the seismic response. Damping is assigned in the form of a percentage of "critical damping," which is the amount of damping required to completely prevent oscillations. As an example, Figure 2-9 shows a typical pattern of displacement vs. time for both critically damped and 5% damped systems, when stretched to a distance " y_0 " and then released. Most seismic designs for structure performance in the elastic region are based on damping equal to about 5% of critical, depending on the materials of construction and the structural system. For most performance ranges, structure damping is rarely considered less than 2% or greater than 10%.

The elements of response discussed above apply to all structural systems. In real systems, modeled with numerous degrees-of-freedom, there exists one mode shape for each degree-of-freedom. As a simple example of a multiple-degree-of-freedom (MDOF) system, see Figure 2-10.

The building of Figure 2-10 is a four story moment frame structure. It is idealized for

dynamic analysis purposes as a "stick" with four masses corresponding to the weight and location of the floors and roof. The stick between masses is given beam stiffness properties equivalent to the lateral stiffness of the beams and columns at each floor level. This idealization has four degrees-of-freedom, i.e. one horizontal translational degree-of-freedom allowed at each mass location. Generally, vertical or rotational oscillatory motions are not considered as they are not important in the response of this type of structure when horizontal motions are applied to the base.

The periods and mode shapes for this idealized system are shown adjacent to the stick model. Note that the first mode is a "swaying" motion, which is typical of most building structures. This first mode of response has the longest period of vibration, and though other modes of shorter period and greater complexity may also participate to some degree, the first mode is typically the largest contributor to overall seismic response for all but the most unusual structures. For this reason, the magnitude and distribution of seismic design forces specified in the UBC are based upon estimates of the period and shape of the first mode of vibration.

2.1.4 THE BUILDING CODE PHILOSOPHY

In the 1990 Edition of the SEAOC Blue Book, the basic intent of the building code seismic design requirements is given as follows:

"The primary function of these Recommendations is to provide minimum standards for use in building design to maintain public safety during extreme earthquakes likely to occur at the building's site. These Recommendations primarily are intended to safeguard against major failures and loss of life, not to limit damage, maintain functions, or provide for easy repair. It is emphasized that the purpose of these recommended design procedures is to provide buildings that are *expected* to meet this life safety objective.

The specified design forces given herein are based on the assumption that a significant amount of inelastic behavior may take place in the structure due to a major level of earthquake ground motion. As a result, these design forces and the related elastic deformations are much lower than those that would occur if the structure were to remain elastic. For a given structural system, the design provisions are intended to provide for the necessary inelastic behavior, and representations of the element force levels and deformations in the fully responding inelastic structure and given as appropriate multiples of the values found by the linear elastic analysis of the structure under the specified design forces.

Structures designed in conformance with these Recommendations should, in general, be able to:

1. Resist a minor level of earthquake ground motion without damage;
2. Resist a moderate level of earthquake ground motion without structural damage, but possibly experience some nonstructural damage;
3. Resist a major level of earthquake ground motion having an intensity equal to the strongest either experienced or forecast for the building site, without collapse, but possibly with some structural as well as nonstructural damage.

It is expected that structural damage, even in a major earthquake, will be limited to a repairable level for structures that meet these provisions. The level of damage depends upon a number of factors, including the configuration, type of lateral force resisting system, materials selected for the structure, and care taken in construction."

While it is possible to design a structure to resist an earthquake by strength alone, i.e. by remaining fully or nearly elastic, building codes are based on the assumption that it is impractical to specify this level of resistance for all but very hazardous or extremely essential facilities, where continued function or a high assurance of safety is required. Building codes, therefore, specify design of ordinary structures for reduced force levels, but also require special detailing to insure that adequate inelastic capability is available. The level of force reduction and special detailing required varies with each type of construction, based on observed earthquake performance and test data.

For hazardous and essential facilities, where a reduced level of structural damage is desired in order to maintain a basic level of function, increased design force levels are required. This higher force level is accounted for by providing an *Importance Factor*, "I," in the basic force equation which increases the required design force for these facilities.

UCRL-15910 establishes design criteria for DOE facilities based upon the facility hazard classification. The building code approach is used for general use, important, and low hazard facilities. For moderate and high hazard facilities, UCRL-15910 specifies more conservative criteria. UCRL-15910 also contains special detailing and specific quality assurance requirements for DOE facilities of all categories.

2.1.5 THE UBC EQUIVALENT FORCE APPROACH

The equivalent force approach represents earthquake effects by approximating the total lateral inertia forces produced in the structure, rather than by specifying the ground motion itself. The equivalent force approach does not include other potential earthquake effects, such as surface fault rupture, landsliding, liquefaction, settlement, or heaving of soil.

The UBC describes two equivalent force approaches which may be used for design.

1. The *Static Force Procedure* (Figure 2-11), which provides a simplified distribution of the seismic forces calculated using the basic force equation. It is intended for simple, regular structural systems only.
2. The *Dynamic Force Procedure*, which is typically applied by computer using an earthquake response spectra. The response spectrum used may be either the general spectrum provided in the UBC, or it may be a unique spectrum which is developed for a specific site. In either case, the total design force, as represented by the structure base shear, cannot be less than 90 percent of that specified by the UBC basic force equation for regular structures, or 100 percent for irregular structures.

Dynamic analysis must be used for all structures greater than 240 feet in height, for structures meeting height and irregularity limits defined in Section 2.4, and for all structures having a period greater than 0.7 second which are located on soils consisting of more than 40 feet of soft clay or bay mud (having a shear wave velocity less than 500 feet per second).

The selection of either approach depends primarily upon the complexity and irregularity of the structure. Dynamic analysis is always an accepted approach, whereas use of the static method is limited to structures meeting certain criteria.

In the UBC, minimum required seismic design forces are specified by the following *Basic Force Equation*:

$$V = \frac{Z I C W}{R_w}$$

Where:

- V = Base Shear
- Z = Seismic Zone Factor
(for DOE facilities,
see UCRL-15910, Table 4-4)
- I = Importance Factor

- W = Seismic Dead Load (Weight)
 C = Soil/Structure Response Coefficient
 R_w = Seismic Force Modification Factor

The seismic dead load, W , includes structure weight, weights of contents, and some portion of live load, as defined in the UBC. Individual R_w factors are provided in the UBC for various standard types of structural systems and construction. The R_w factor is a force-modification factor, which is based on an estimation of the overall inelastic capability of the structural system and past earthquake experience. The following factors allow structures designed on a modified-force basis to perform adequately during major ground motion:

Total Elastic Strength Capacity. Building codes include margins of 1.33 to 1.7 between the allowable design stress and the yield or strength design basis for individual elements. Additional strength contributions may result from: design conservatism (intended or not), construction practices, and the contribution to lateral load resistance of non-structural elements, such as partitions or mechanical items, within the system. Also, conservatism will occur in structure elements where design considerations or load conditions other than for seismic control the final design.

Modification of Dynamic Response Characteristics. As buildings become overstressed, yield, and experience localized material and member failures, two things happen:

1. Lateral deflections increase significantly, causing an increase in the effective period of vibration of the building. For many structures and most soil conditions, this shift in period will shift the structure response away from the period range in which the greatest amplification of response occurs, thereby reducing inertial forces generated within the structure. However, for buildings or equipment which are relatively stiff, *which is common at many DOE sites*, this can cause a worsening effect by shifting the effective period of the item in the direction of increasing response amplification.
2. Through strain, heat, friction, and other mechanisms, energy is dissipated in the structure, reducing the response. This is represented as an increase in structure damping.

Redundancy, Well-Controlled Damage Modes and Inelastic Response. Seismic ground motions are of finite duration. Structures that contain redundant lateral force-resisting elements and well-controlled damage modes (i.e., contain multiple locations which may yield and become damaged before collapse occurs) will likely survive the duration of strong shaking in a major earthquake.

It is inherent in the force-reduction approach that a certain level of redundancy and capability for inelastic performance be present. In order to assure that these qualities will be present, the code imposes in several ways specific design requirements: by setting restrictions on the overall structural geometry (i.e., irregularity); by controlling damage modes; and by requiring minimum levels of strength and energy absorption capability within individual elements (as defined within the individual materials sections of the code).

2.1.6 PROVIDING STRUCTURES WHICH CAN RESIST OVERSTRESS

During strong shaking, structure materials which are overstressed begin to yield and experience significant changes in force-deflection relationships. Figure 2-12 presents typical force-deflection relationships for brittle and ductile materials. A structure can be designed to resist overstresses in a ductile manner, as shown in Figure 2-12, if it is designed to exhibit *inelastic behavior*, or inelasticity. Inelasticity can be attained by providing 1) ductility (as defined below) in members and connections, 2) by providing redundancy or backup systems, and 3) by controlling damage modes.

Ductility is a material property which exists only for non-brittle materials such as steel. It is a measure of the materials ability to undergo post-yield deformations without fracture (breaking). Inelastic behavior can be designed to occur in structures which use ductile materials (such as steel) as well as those which use brittle materials. For example, plain concrete is a brittle material, but properly reinforced concrete (and masonry) can be ductile and produce structures which exhibit satisfactory seismic performance. Conversely, while steel is an inherently ductile material, proper proportioning of members and design and detailing of connections is necessary to assure ductile behavior of the overall structural system.

Redundancy in a structural system allows for redistribution of internal loads around local areas of overstress, which effectively provides a backup system for protection against failure when substantial overstresses occur. As an example, a cantilever is a non-redundant structural system; failure is initiated when yield occurs at the base of the cantilever, as there are no backup or alternative load resisting elements.

Damage modes can be controlled by making undesirable damage modes stronger than desirable modes. For example, the

Fig. 2-12: Force-Deflection Curves
Brittle vs. Ductile Behavior

inelastic bending response of a properly reinforced concrete beam is inherently ductile, while the inelastic shear response tends to be brittle. Brittle shear failure is therefore prevented by proportioning bending and shear strength capacities such that inelastic response occurs first in (and is primarily limited to) bending.

Undesirable failure modes are:

1. Those resulting in total collapse of the structure, such as caused by progressive column failures.
2. Those involving sudden failure, such as a buckling or brittle (e.g. shear) failure.

2.2 BASIC STRUCTURAL SYSTEMS AND STRUCTURE COMPONENTS

In order to resist both vertical loads and wind or seismic effects, any building or structure which is stable requires two distinct structural systems; a vertical support system, and a lateral load-resisting system. Each system consists of a series of structural components (beams, columns, walls, etc.) which combine to provide resistance against the direction of loads. These components can consist of individual elements, such as columns, or they may consist of combinations of elements, which connect to produce sub-systems, such as shearwalls, moment frames, etc.

Basic structure components which combine to form the vertical support system include roof and floor framing systems, columns, bearing walls, and foundations. All structures must have complete and sound vertical load-supporting systems to avoid collapse under normal gravity and live load conditions. On the other hand, structures with missing or inadequate lateral load-resisting systems can stand indefinitely, unless an event occurs which results in lateral wind or seismic loads of sufficient magnitude to cause collapse.

Basic structure components which provide lateral force-resistance, shown in Figure 2-13, include shear walls, braced frames, and moment-resisting frames, as well as (not shown) diaphragms (such as roof decks, floor slabs, or horizontal bracing systems) and foundations. Depending upon the type, use, and geometry of the structure in each application, these components can be combined to form a *structural system*.

Standard structural systems are described in section 2.2.1. Basic lateral force-resisting components and their principal design elements are described in section 2.2.2.

(a) FRAME ACTION BY MOMENT-RESISTING BENTS

(b) SHEAR WALLS AS VERTICAL CANTILEVERS

(c) BRACED FRAMES OF STEEL

Fig. 2-13: Basic Lateral Force-Resisting Components

2.2.1 BASIC STRUCTURAL SYSTEMS

The UBC defines certain standard structural systems, called *Basic Structural Systems*, which have been classified according to relative seismic performance. Within each system, seismic " R_w " factors are assigned based upon the types of materials and lateral force-resisting components used.

The basic structural systems defined by the UBC are:

- ***Bearing Wall System:*** Any system which uses shear walls and/or braced frames in situations which must simultaneously support vertical loads and resist lateral loads.
- ***Building Frame System:*** Systems containing an essentially complete frame system which provides support for gravity loads. Resistance to lateral loads is provided by shear walls and/or braced frames.
- ***Moment-Resisting Frame System:*** Systems using moment frames to resist lateral loads and/or support vertical loads.
- ***Dual System:*** Systems which provide two complete lateral load systems, each capable of resisting all or part of the total seismic load. One system must always be a moment-frame system; the other can consist of either reinforced shear walls or braced frames which are added at selected locations.
- ***Undefined Systems:*** Building systems which do not fit the above descriptions, such as shell or guyed systems.

Frequently, there is little visible difference between the frames, bearing walls, etc., used to provide vertical support, and the similar frames, shear walls, etc., used to resist lateral forces. The basic distinction which exists between the two types of systems arises from the following factors:

- All loads applied to a structure tend to be resisted by the structure components which have the greatest rigidity (relative to other components) in the direction of load. Thus, vertical loads supported by an elevated floor slab tend to be borne by the adjacent columns. On the other hand, lateral loads applied against a floor diaphragm may span entirely past interior support columns (which provide little or no lateral rigidity) and be resisted by more distant (but very rigid) shear walls or braced frames.
- The connections used to fasten components must provide adequate strength to resist forces and deflections imposed from all directions. Where design assumptions require a rigid or inflexible link between lateral

load resisting components, the connection must impart rigidity; for non-lateral load resisting components, it must provide adequate flexibility to accommodate structure deformations without impact or binding.

Figure 2-14 shows examples of simple steel connections used to construct steel frames. The details shown at the left side of the figure are details which provide connections to the supporting beam or column webs only. These types of connections provide very little rigidity to resist rotations of the beams or columns, and are therefore usually idealized by engineers as being equivalent to hinges. When used to construct a steel frame, these details provide for vertical support, but provide very little resistance to lateral deflections. On the other hand, the details shown at the right side of Figure 2-14 provide substantial

Fig 2-14: Steel Frame Connections

restraint to prevent rotations of the attached beams or columns, and can be used to create a moment-resisting frame which resists lateral forces in the direction parallel to the frame.

Structures must be able to resist forces which are imposed from any direction. Generally, this means that two separate systems are provided, one for each of the two orthogonal axes of the building. When forces are applied about either orthogonal axis of the structure, some of the resisting elements will be aligned parallel to the forces. Wall and bracing elements have high relative rigidity in-plane, and thus provide substantial lateral support. However, walls or bracing systems have low relative strength and rigidity when aligned perpendicular to the direction of load, and thus must be properly attached to and supported by structural components which are parallel to the forces. Wall damage and collapses due to out-of-plane forces and deformations has frequently occurred in past earthquakes.

Figure 2-15 illustrates basic aspects of structural system behavior. When seismic forces are applied to the system in one direction, the distribution of forces and reactions follows a path, from the most flexible components to the most rigid, and from the top of the structure to its foundation, in the following manner:

- Components which are perpendicular to the ground motions are subjected to inertial lateral forces proportional to their self-weight. If the components are simply supported at their top and bottom sides, then the reactions from the lateral forces will be equal at the top and bottom edges. The exact nature of the top and bottom connections will determine the actual distribution of lateral forces to the supporting systems.

Fig. 2-15: Lateral System Load-Paths

- The diaphragm transfers loads from the supported perpendicular components, plus inertial loads from its own self-weight and the weight of any supported contents, to the components which transfer lateral forces vertically.
- The components which transfer lateral forces vertically (vertical components), whether shearwalls, braced frames, or moment-resisting frames, transfer loads from the diaphragm, plus loads from upper stories and inertial loads generated from self-weight, to the foundation system.
- The foundation system transfers lateral and overturning loads from the vertical components into the ground, and provides resistance against overall structure sliding or overturning.

2.2.2 BASIC STRUCTURAL COMPONENTS

Basic structural components which collect loads from roof and floor levels and transfer them to foundations, shown in Figure 2-13, include shear walls, braced frames, and moment-resisting frames.

Note: The UBC assumes that seismic stresses will govern in the design of the majority of structure elements and therefore ductile behavior is essential. The level of ductile detailing provided must always correspond to the value of R_w assumed. For many DOE facilities, seismic stresses do not govern (often by a wide margin). Thus, ductility in these components may not be a critical concern. In structures where very massive structural elements are used, it may be appropriate to perform analysis based upon a reduced value of R_w , in order to avoid unnecessary and elaborate detailing provisions.

Moment-Resisting Frames. Moment frames rely on bending forces developed in beams and columns to resist lateral forces. Because the rigidity of moment frames is much less than either shear walls or braced frames, lateral deflections of the structure will be much greater. The ratio between relative story deflection and story height is called the *story drift ratio*. Moment frames which permit too much drift during earthquakes may permit an excessive amount of damage to non-structural components, such as siding, partitions, and contents.

Critical design elements of moment-resisting frames include:

1. **Beams and Columns:** Frame elements must be designed to resist internal bending and shear forces. Optimum seismic performance is gained when the members selected have shear strengths which are greater than bending strengths, and when bending strengths of the beams are less than that of

the columns. This provides a controlled failure mode and, in multi-story frames, increases the total energy absorption capability of the frame.

2. *Beam-Column Joint:* The stability of the frame is dependent upon the beam-column joint remaining sound and relatively rigid under large frame deformations. Adequate strength and toughness must be provided in the joint to permit repeated cycling of stresses without a loss of joint integrity.

Shear Walls. The shear wall is designed as a vertical beam. Critical design elements include:

1. *In-Plane Shear:* The wall acts as a web, or shear membrane, to resist shear forces. Adequate anchorage must be provided along the top and bottom edges of the web to transfer the shearing forces into, and out of, the web.
2. *In-Plane Bending:* To resist bending stresses due to overturning moments, structural elements, called *chords* or *boundary elements*, are concentrated at the vertical edges of the wall. *Chords* are tension-only elements, which resist the tension component of bending forces. *Boundary elements* are tension-compression elements, which resist tension forces as well as the compression forces due to bending and vertical loads. Boundary elements are required by the UBC only in high-seismic zones.

Both chord and boundary elements must be anchored into a foundation which is capable of transferring the required vertical forces into the ground.

3. *Resistance Against Forces Perpendicular to the Wall:* Adequate bending strength and anchorage must be provided to resist inertia loads applied perpendicular to the plane of the wall, including those caused by the wall's self-weight.

Braced Frames. Braced systems rely on the axial rigidity of vertical and diagonal elements to resist lateral forces. The configuration of diagonal elements within the bracing can vary, and some configurations have better seismic performance than others. Types of bracing systems, shown in Figure 2-16, in order of relative seismic performance (best to worst), include:

1. *Eccentric-Braced Frame (EBF):* A special type of bracing systems which is designed to provide a high degree of seismic resistance by permitting controlled overstresses within a short, ductile beam segment.

2. ***Tension-Compression Bracing:*** Diagonal elements are provided either singly or paired in an "X" pattern to resist forces both in tension and compression.
3. ***Chevron Bracing:*** Arranged in either a "V" or inverted "V" configuration, the diagonal elements resist lateral loads by sharing the load between tension and compression elements. Bracing in this configuration also will resist vertical loads applied to the beam, although the UBC requires that the beam be designed to resist vertical loads as if the bracing did not exist. This configuration can experience problems when overstressed because buckling of the diagonal undergoing compression creates a force imbalance which increases bending forces on the beam. The UBC requires a 50% increase in the design forces of both horizontal and diagonal bracing elements in chevron systems which resist seismic forces.
4. ***Tension Bracing:*** Similar to tension-compression bracing, except that the diagonal elements, by being very slender, are assumed to buckle at very low compression forces. This configuration can experience a loss of strength under repeated tension-compression cycles. In *all* seismic zones, tension bracing is only permitted in structures of less than two stories in height, and all elements of the bracing system must be designed by increasing the applied design forces by a factor of $3R_w/8$.
5. ***K-Bracing:*** This system provides the worst seismic performance. Overstresses in the diagonal elements can lead to buckling of the diagonal undergoing compression. The resulting force imbalance creates a lateral force on the column, which could possibly lead to a general collapse of the system. In seismic zone 2, the UBC specifies that the diagonal elements must be designed by increasing applied design forces by a factor of 1.5. In seismic zones 3 and 4, K-bracing is only permitted in structures of less than two stories in height, and all elements of the bracing system must be designed by increasing the applied design forces by a factor of $3R_w/8$.

Critical design elements of bracing systems include:

1. *Diagonal elements:* Designed to resist lateral (story shear) forces. Because bracing connections often tend to have brittle modes of failure, the connections of bracing elements must always be designed to be stronger than the braces themselves.
2. *Vertical Elements:* Designed to resist overturning forces.
3. *Horizontal Elements:* Designed as axial struts to collect and transfer tension and compression forces into the diagonal elements. The strut system can be sloped, such as when pitched roof elements are used as struts.

Diaphragms and Horizontal Bracing Systems. Diaphragms are generally designed as flexible or rigid beams that span between vertical-resisting elements. Critical design elements of diaphragm systems include the following:

1. *Shear Membrane:* The floor or roof acts as a web, or shear membrane, to resist shear forces. The web is assumed to resist shear forces only (no tension or compression).
2. *Chords and Boundary Elements:* Located at the extreme edges of the diaphragm, the chords resist tension forces caused by bending in the diaphragm, in a manner similar to the flanges of an I-beam. When compressive stresses warrant, boundary elements may also be required to resist the compressive forces caused by bending.
3. *Collectors and Distributors:* Also called drag struts, collectors/distributors are used to resist axial tension and compression forces within the diaphragm in the following situations (see Figure 2-17):
 - a. Collect shear forces from the diaphragm and transfer to the vertical resisting elements (walls, braces, or frames)
 - b. Redistribute chord forces at diaphragm offsets, and gather loads and distribute to intersecting walls, braces, or frames.
 - c. Redistribute diaphragm shear forces and local chord forces around openings within the diaphragm.
 - d. Distribute points of concentrated force into the diaphragm, such as from heavy roof-top equipment.

Fig. 2-17: Collector Elements

Foundations. Foundation systems typically involve piles, caissons or spread footings. Critical design elements of foundation systems include:

1. *Overturning resistance.* Piles must have tensile capacity to resist uplift; spread footings must be massive enough and/or support adequate soil surcharge to counteract uplift.
2. *Sliding resistance.* Either passive soil pressure, friction, or a factored combination of both may be used, depending upon the soil type and foundation geometry.
3. *Bending and shearing strength.* The foundation system must have adequate strength to transmit loads without failure or excessive deflections.
4. *Settlement resistance.* Excessive soil bearing pressures caused by vertical loads and overturning moments could lead to some permanent settlement or tilting of the structure.

Because damage resulting from overstresses in the foundation can be difficult to detect and expensive to repair, it is prudent to provide additional margins in the foundation design where convenient and cost-effective.

2.3 LOAD-PATHS

The load-path concept involves the systematic analysis of loads throughout a structure, from points of origin or application, to the final points of resistance. This analysis can be done at a "macro" level; in which general forces carried by diaphragms, collectors, walls, etc. are determined; and at a "micro" level, in which forces through bolts, stiffeners and welds within a connection are traced.

While providing a continuous load path is an obvious design requirement for vertical loads, experience has shown that it is sometimes overlooked or not completely developed when designing for lateral loads. The failure to review *all* members and connections for *each* combination of load can lead to weak-links in the load path. The UBC requires a continuous load path as one of its basic requirements; thus detailed provisions for identifying and designing this load path are given in the appropriate design and material sections of the UBC.

Important aspects of developing and designing/detailing the lateral load path include:

- Consideration of all conditions of load. For example, a shear plate or clip angle connection which is adequate for connecting a steel beam supporting vertical loads may be inadequate to resist axial forces at points where the beam is used as a tension, or collector, element.
- Providing collector elements when transferring loads from diaphragms (roof or floors) into walls, braces, or frames.
- Stiffness compatibility of load-resisting elements acting in parallel (before and after yield). For example, welding cannot be used to "increase" the strength of a connection utilizing bearing or machine bolts; the weld must be designed to resist the all of the load because of slip within the bolted connection. The weld will resist the full load before the bolts begin to bear load.
- Consideration of eccentric conditions. *Eccentricity should always be accounted for when evaluating forces within connections.* For example, Figure 2-18 shows deformations in a detail of two bars which are connected by eccentric lapping.

Fig. 2-18: Eccentric Tension Splice

- Consideration of the transfer of seismic loads through the foundation system into the surrounding soil. It is not sufficient to assume that seismic forces are "resolved" when they reach the soil system. A rational means, using each element of the foundation system (e.g., bearing, friction, passive pressure), must be employed to resist seismic loads.

2.4 STRUCTURE CONFIGURATION

Although it is the structural engineer who determines the actual details of seismic resistance, the architect and other design team members also can greatly affect the ultimate seismic resistance of the structure. Simple, symmetrical structural geometries tend to distribute lateral load uniformly throughout the structural system. Structures with significant irregularities in geometry, mass or stiffness tend to have local areas of stress concentration. This is undesirable, as such areas can become early damage initiation points which can rapidly degrade structural integrity and result in poor seismic performance.

The static linear force distribution used in building codes and associated lateral force reduction factors are based primarily on the assumption of:

- relatively symmetrical geometry
- gradual variation of stiffness with height
- uniformly distributed stiffness in plan
- uniformly distributed mass in plan, and minimal variation of mass with height
- sound, complete and predictable path of resistance in the structural system
- structural modes of vibration are primarily translational; torsional components are small
- the fundamental shape of vibration can be represented approximately as a straight line.

It is the mutual responsibility of the architect and structural engineer to identify and resolve geometry problems early in the design process. The challenge of the design team is to create a building with a structural system that is reasonably regular and still retains the intended form and function. Poor geometry can result in poor performance unless special attention is given during the design process (at increased design and construction cost).

Guidelines have been provided in building codes to help determine how much irregularity is tolerable within the equivalent force approach. The following subsections describe the basic types of irregularities defined in the Uniform Building Code (UBC). Structures which do not meet these limits are required either to meet additional design conditions within the static force approach or must be designed using the dynamic analysis approach.

2.4.1 VERTICAL IRREGULARITIES

Basic vertical irregularity types due to structure geometry, or to stiffness or mass irregularities, are defined below. The description of these types, together with a summary of structure or analysis restrictions imposed, are:

TYPE A, Fig. 2-19: Stiffness Irregularity (Soft Story). A soft story is defined as having a lateral stiffness less than 70 percent of that in the story immediately above or less than 80 percent of the average stiffness of the three stories above, unless the story drift ratio is shown to be less than 1.3 times the calculated story drift ratio for the story above.

Experience has shown that structures which tend to concentrate lateral deflection in one or more stories also tend to concentrate stresses at these stories.

Structures greater than 5 stories or 65 feet in height of the type exhibiting soft story behavior must be designed using a dynamic analysis method. This criteria is waived if it can be shown that the story drift ratios remain essentially linear (that the deflected shape remains linear) despite the irregularity.

TYPE B, Fig. 2-20: Weight (Mass) Irregularity. Mass irregularity is considered to exist where the weight of any story is more than 150 percent of the weight of an adjacent story, unless the story drift ratio is shown to be less than 1.3 times the calculated story drift ratio for the story above. A roof which is lighter than the floor below need not be considered.

Structures greater than 5 stories or 65 feet in height with significant mass irregularities must be designed using a dynamic analysis method. This criteria is also waived if it can be shown that the story drift ratios remain essentially linear (that the first mode shape remains linear) despite the irregularity.

The weight of all building component, and equipment items should always be included into the weight of all analysis models. Structures supporting heavy (weight exceeds 25 percent of the story weight) components (such as equipment items, tanks, or process vessels) which are classified as non-rigid or flexibly supported (natural period of vibration of component or support greater than 0.6 seconds) should include both the stiffness and mass of the component as an appendage to the analysis model.

Fig. 2-19: Soft Story

Fig. 2-20: Mass Irregularity

TYPE C, Fig. 2-21: Vertical Geometric Irregularity. Vertical geometric irregularities are considered to exist when the horizontal dimension of the lateral force resisting system in any story is more than 130 percent of that in an adjacent story. One-story penthouses are waived.

Structures greater than 5 stories or 65 feet in height with significant vertical geometric irregularities must be designed using a dynamic analysis method.

TYPE D, Fig. 2-22: In-Plane Discontinuity in Vertical Lateral Force-resisting Element. In-plane discontinuity is considered to exist where there is an in-plane offset of the lateral load-resisting elements greater than the length of those elements.

Columns supporting offset shearwalls or other lateral force-resisting elements may experience substantial tension and compression forces due to overturning effects. The UBC specifies special provisions for these elements, extending continuously from the point of discontinuity into the foundation, as follows:

- In seismic zones 3 and 4, columns resisting overturning from discontinuous elements are required to be checked using special load combinations, which increase seismic forces by a factor of $3R_w/8$, and are required to be detailed as ductile-frame elements.
- In seismic zone 2, columns resisting overturning from discontinuous elements are required to be detailed as intermediate-frame elements, if concrete, or checked using plastic design provisions, if steel. Other materials are not addressed by the code, but should be treated similarly.

TYPE E, Fig. 2-23: Discontinuity in Capacity (Weak Story).

A weak story is one in which the story shear strength provided is less than 80 percent of that provided in the story above. *Note that a story may have adequate calculated strength to resist the specified seismic loads and yet still be classified as a weak story.* The story strength is the total strength of all seismic resisting elements sharing the story shear for the direction under consideration.

Structures with story strengths between 65 and 80 percent of that of upper stories are considered irregular and must always be designed for 100% of the forces from the Basic Force equation (versus 90% for regular structures if the dynamic method is used). Structures containing stories weaker than 65 percent of upper stories are limited to two stories, or 30 feet, in height, unless designed for $3R_w/8$ times code forces (corresponding to full elastic earthquake levels).

2.4.2 PLAN IRREGULARITIES

TYPE A, Fig. 2-24: Torsional Irregularity (Rigid and Semi-Rigid Diaphragms only). Torsional irregularity is considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts of the two ends of the structure (i.e. greater than 1.5 times the drift at the other end).

Fig. 2-24: Torsional Irregularity

The UBC requires torsional moments applied to torsionally irregular structures to be amplified by a factor " A_x ," as follows:

$$A_x = \left[\frac{\text{Max. Deflection}}{1.2 (\text{Avg. Deflection})} \right]^2 \leq 3$$

Torsionally irregular structures designed to seismic zone 3 or 4 criteria are not permitted a one-third increase in allowable stresses for 1) connections of diaphragms to collectors, or 2) connections of collectors or diaphragms to resisting elements (walls, braces, or frames).

In addition, for structures which exhibit torsional irregularity about both axes, the code requires that earthquake forces must be either evaluated using a dynamic analysis method or evaluated to include orthogonal (nonparallel) effects by combining the effects of the two orthogonal directions using one of the following methods:

- By designing for 100 percent of the forces in one direction plus 30 percent of the forces in the perpendicular direction.
- By combining the forces on a square-root-sum-of-squares basis, assigning positive and negative signs to give the most conservative results.

Type B, Fig. 2-25: Reentrant Corners (Plan Geometry Irregularity). Plan irregularities exist when plan configurations of a structure and its lateral force resisting system contain

reentrant corners where both projections of the structure beyond a reentrant corner are greater than 15 percent of the plan dimension of the structure in the given direction.

Geometrically irregular structures designed to seismic zone 3 or 4 criteria are not permitted a one-third increase in allowable stresses for 1) connections of diaphragms to collectors, or 2) connections of collectors or diaphragms to resisting elements (walls, braces, or frames). In addition, diaphragm chords and drag members must be designed considering movements of the projecting wings in both directions about the reentrant corner.

TYPE C, Fig. 2-26: Diaphragm Discontinuity. A significant diaphragm discontinuity exists if the diaphragms contains abrupt discontinuities or variations in stiffness, or has cutout or open areas greater than 50 percent of the gross enclosed area of the diaphragm.

Structures with discontinuous diaphragms designed to seismic zone 3 or 4 criteria are not permitted a one-third increase in allowable stresses for 1) connections of diaphragms to collectors, or 2) connections of collectors or diaphragms to resisting elements (walls, braces, or frames).

TYPE D, Fig. 2-27: Out-of-Plane Offsets. An out-of-plane offset consists of a discontinuity in the lateral force path, such as when a shear wall does not vertically align throughout the structure height because of a lateral offset of the wall which occurs at some point.

Structures with out-of-plane offsets designed to seismic zone 3 or 4 criteria are not permitted a one-third increase in allowable stresses for 1) connections of diaphragms to collectors, or 2) connections of collectors or diaphragms to resisting elements (walls, braces, or frames).

Columns supporting offset shearwalls or other lateral force-resisting elements may experience substantial tension and compression forces due to overturning effects. The UBC specifies special detailing provisions for these elements, extending continuously from the point of discontinuity into the foundation, which are the same as those described for Type D vertical irregularities (in-plane offsets).

Fig. 2-25: Reentrant Irregularity

AREA "AB" > 50% AREA "XY"

Fig. 2-26: Diaphragm Discontinuity

Fig. 2-27: Out-of-Plane Offset

Source: Reference 2-2

TYPE E, Fig. 2-28: Nonparallel Systems. A nonparallel system is present when the vertical lateral load resisting elements are not parallel to, nor symmetric about, the major orthogonal axes of the lateral force resisting system.

Structural geometries which are nonparallel do not tend to deflect in the direction of the assumed axis of the building, but instead exhibit coupled motion about two or more axis. Figure 2-28 is a plan view of a typical non-symmetric structural geometry.

For nonparallel systems, the code requires that earthquake forces must be evaluated to include orthogonal (nonparallel) effects. Preferably, a dynamic analysis which includes three-dimensional effects should be performed.

2.4.3 EXAMPLE OF DOE FACILITY IRREGULARITIES

DOE facilities often must contain irregularities in order to satisfy complex functional needs. Where radioactive materials are processed, wall thicknesses are often determined by shielding criteria rather than by structural analysis, frequently causing unusual distributions in floor and wall thicknesses.

Figure 2-29 shows a section through a hypothetical structure which contains several irregularities. Irregularity types, and actions which could be taken to resolve them, are as follows:

1. The left-side wall of the lower-center room increases in thickness with height. This condition is undesirable for vertical loads, since it induces eccentricity and bending into the structure. For seismic loads, this condition is an out-of-plane offset in the wall (Type D Plan Irregularity).

The irregularity shown could be supported by adding supporting beams into the floor beneath the wall. Alternatively, the lower wall could be thickened to equal the thickness of the upper wall, or pilasters could be added within the lower wall to support the eccentric weight.

If the wall eccentricity had to be retained, then special notes would need to be added to the construction documents to instruct the contractor to retain shoring beneath the eccentric portion until the entire floor and wall system necessary to support the eccentricity, including the floor located above the eccentric wall, was installed and cured.

2. There is an out-of-plane offset in the wall at the right side of the center room (Type D Plan Irregularity). This situation is common where a crane is used to remotely transfer material into adjacent rooms. If the upper wall continued to grade at some point beyond the offset, this would become an in-plane offset (Type D Vertical Irregularity).

If the condition were an out-of-plane offset, shear forces from the upper wall must transfer through the floor slab diaphragm into the adjacent lower walls. The large opening shown in the floor at the left of the upper wall could prevent shear transfer in this area. If the condition were an in-plane offset (a preferred alternative), then shear forces would be transferred through chord reinforcing into the in-plane walls beyond. Overturning forces from the upper wall must also be resisted, either by adding columns beneath each end of the wall (preferred), or by detailing special beam elements to support each end of the wall.

3. There is a large discontinuity (opening) in the first floor diaphragm at the location where the center room projects through. Depending upon the size of the opening, this may be classified as a Type C Plan Irregularity.

The design of diaphragms around openings of any size must consider local stress concentrations which occur around openings. This topic is described in detail in Chapter 5, Diaphragm Systems. Where openings are large enough, code requirements described for Type C Plan Irregularities must be included. In addition, the effect of openings on diaphragm rigidity must be considered where appropriate. This can generally be accomplished by a finite element analysis in which diaphragm flexibility is properly modeled.

4. The stiffness of the structural framing at the roof level may be substantially less than the lower structure. If so, this may be classified as a Type C Vertical (Geometric) Irregularity.

For the specific irregularity shown, the UBC provides specific guidelines (described in Chapter 8 of this document) regarding specific cases when the static force procedure may be used for a vertically irregular structure, otherwise the dynamic force procedure must be used. In this example, the dynamic force procedure would likely provide a more accurate force distribution than the static force method.

2.5 DRIFT AND DEFLECTION CONSIDERATIONS

2.5.1 CALCULATED VERSUS ACTUAL DEFLECTION

Structure drift is defined as the incremental deflection which occurs between building stories or structure levels, as shown in Figure 2-13(a). The accumulation of story deflections for all stories is thus the total deflection which occurs at the top of a structure. The drift ratio is defined as the ratio between interstory deflection and story height (i.e., drift/height).

Structure deflections and drift ratios which are calculated based upon a linear, elastic structural analysis are less than deflections which will occur when inelastic behavior is permitted in a structural system. For this reason, when seismic analysis is done using the equivalent static force procedure defined in the UBC, the actual drifts and deflections are a function of the R_w factor used in the analysis. Although the R_w factor is used as a reduction factor for structural forces, it must be used as an increase factor for deflections. It has been estimated that inelastic structural deflections are on the order of $3R_w/8$ times the calculated elastic deflections which result from code-based ($ZICW/R_w$) seismic forces.

In addition, the following factors, which are not directly addressed by building codes, can be significant:

- The presence of large quantities of non-structural elements such as cladding or partitions within a structure may tend to stiffen a structure, at least during the first few cycles of seismic motion, reducing deflections.
- Translation and rocking of the structure foundation system and surrounding soil mass may occur, increasing deflections.

In order to limit damage associated with interstory drift and structure displacement and to prevent instabilities due to vertical loads, drift control is accomplished in the UBC by:

1. Limiting calculated structure story drift due to code-prescribed seismic forces (i.e., no $3R_w/8$ factor) to $0.04/R_w$, or a maximum of 0.005, times the story height, for structures with fundamental periods of less than 0.7 seconds. For buildings having a fundamental period of 0.7 seconds or greater, this value is reduced to $0.03/R_w$, or a maximum of 0.004, times the story height.
2. Requiring the separation distance between adjacent buildings to be at least $3R_w/8$ times the elastic deflections calculated using code-prescribed seismic forces for each structure. Structure separations must be calculated considering the total deflections of adjacent structures, i.e. assuming that they move toward each other.

Failure to provide adequate separation between structures means that the structures may be subject to pounding against each other during strong motion. If the point of contact which the adjacent structure strikes is in an area of vulnerability, such as at mid-height of a column or bearing wall, a failure may result which could lead to collapse of the structure. Pounding also results in higher acceleration responses which may be destructive to both structural and non-structural elements in local building areas.

2.5.2 DEFLECTION COMPATIBILITY

Deflection incompatibilities exist when key elements of the lateral force resisting system are incompatible with the stiffness or deflection capacity of structural or nonstructural components of a structure. Examples of this condition are:

- Diaphragm systems with diaphragm deflections which exceed the out-of-plane deflection limits of attached walls or frames (Figure 2-30). This is of particular concern for structures with flexible metal

or plywood diaphragms, where diaphragm mid-span deflections can be several inches in magnitude (often more than the attached shearwalls or frames). Excessive out-of-plane deflections can cause problems (including P-delta effects) to attached walls.

- Flexible structure systems used to support stiff nonstructural or architectural elements, e.g. using sag rods or light steel bracing adjacent to a non-structural, but very stiff, wall or facade.

2.6 SUMMARY OF BASIC PRINCIPLES FOR SEISMIC DESIGN

Building codes specify minimum earthquake resistance requirements in terms of design lateral forces and allowable material stresses. However, achieving the intended level of seismic performance and safety is equally a function of structural systems and material selected, details of connectivity throughout the structure, and the quality of construction.

Historically, unsatisfactory structural performance during earthquakes has been attributed less often to "underdesign," in terms of code force levels, and more often to the following: 1) poor building configuration, 2) the presence of "weak links" in the structure, 3) inability of the materials used to absorb repeated shock or overload, 4) unanticipated interactions between adjacent structures, and 5) poor construction (materials and/or practices). All of these factors are within the control of the design and construction team. Irrespective of actual load levels and material stresses used in seismic design, a highly survivable structure can be achieved through careful application of the principles presented herein.

It is important in applying the principles set forth that the designer treat the building seismic system as an integrated system of diaphragms, walls, frames, collectors, etc. The designer must follow major inertial forces from the point of origin through the structure and into the ground, and make sure that every element and connection along the load path is adequate to maintain the integrity of the system. It is necessary to visualize the response of the complete structure and to keep in mind that the real forces in a major earthquake:

- are not static but dynamic, cyclic and repetitive;
- will cause stresses and deformations well beyond those determined from the elastic, code level design.

Seismic forces are assumed to come from any horizontal direction as well as in the vertical direction, and must be combined with gravity loads. For general use and low-hazard category facilities, UCRL-15910 specifies design in the same manner as the UBC, by generally applying seismic forces in horizontal directions only. For moderate- and high-hazard facilities, UCRL-15910 specifies design by combining horizontal and vertical seismic force components.

Basic guidelines to follow when designing for seismic resistance are to:

- Trace the loads through the structure to ensure a continuous load path.
- Adequately design connections to ensure that the connections are stronger than the elements connected (design of connections for 1.5 times element load levels is reasonable)
- Control failure modes by proportioning elements to yield in ductile modes.
- Build-in ductility through selection of materials, reinforcing, and design.
- Minimize irregularities when possible.

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DETAILS FOR WALL SYSTEMS

The following section discusses general shear wall construction and behavior. Detailed discussion for different types of wall construction is included in Sections 3.2 through 3.4 for masonry, cast-in-place concrete, and precast concrete tilt-up construction.

3.1 INTRODUCTION

3.1.1 SHEAR WALL BEHAVIOR

In general, individual shear walls (as shown in Figure 3-1) are designed to behave as cantilevers in-plane, fixed at the foundation level and free to rotate and displace at the roof or top of wall level. By resisting the in-plane shear and bending moments caused by lateral loads, shear walls impart lateral rigidity and stability to the structure. Walls often function as load-bearing elements as well, and are subject to vertical compressive stresses in combination with the stresses from lateral loads. There are usually advantages in the foundation design in having shear walls act as both vertical load bearing and lateral load resisting elements, however, the specified design seismic forces are greater than those for non-load bearing walls.

Shear wall reinforcing, shown in Figure 3-1, is designed by assuming that the wall behaves like a deep beam, with flanges at each end and a web. Horizontal reinforcing in the web is designed to resist the shear stresses. Uniformly spaced vertical reinforcing in the web helps to control diagonal cracking. Concentrated reinforcing located at each end acts as the flanges (or "chords") to resist overturning tensile forces. Where high compressive loads warrant, additional column reinforcing is also added at the flanges as "boundary elements" to resist the compressive forces and provide buckling restraint to vertical reinforcement.

Although the shear wall chord or boundary element reinforcing generally consists of a concentrated group of reinforcing bars near the extreme wall edge, it is sometimes alternatively designed using distributed vertical reinforcing across a portion of the wall length. Concentration of chord steel at the boundaries of a wall to resist overturning forces can be a more efficient use of the steel than distributing it uniformly, since the

Fig 3-1: Typical Shear Wall Reinforcing

effective depth to the center of reinforcing is increased. When boundary elements are included, it also allows the doweled anchorage splices and lap splices in the reinforcing steel to be enclosed and confined by transverse reinforcing ties, improving their performance during large strains. For tall walls (with overall height greater than length), chord reinforcing should be concentrated near the ends to the maximum practical extent.

Sometimes the reinforcing cannot be concentrated at the ends. For example, in masonry walls, chord reinforcing often is required to be uniformly distributed across a section of wall in order to prevent overcrowding of masonry cells at the end of the wall. When this occurs, the design of this reinforcing must reflect the reduced distance between the tension and compression forces which resist overturning. This can be done either by calculating the distance to an effective center of reinforcing or by performing a strain-compatibility analysis of the wall and reinforcing.

For short walls (walls with height less than length), a uniform distribution of vertical reinforcement is actually preferred. Although designed as beams, short walls behave much like concrete corbels, with the exception that the load is uniformly applied along the top of the wall and not generally concentrated at one end. After initial cracking, a truss-like mechanism of force develops, as shown in Figure 3-2. The primary load-resisting mechanism, shown in the diagonal wall segment marked "2," consists of a diagonal compression zone through the wall, which is restrained from uplift by the distributed vertical reinforcing. In the triangular corner segment (marked "1"), both vertical and horizontal reinforcing are required to resist external forces. Thus, for short walls in which lateral loads are uniformly distributed, the preferred distribution of vertical reinforcement is a uniform distribution of reinforcement over the full length of wall, with only nominally increased steel at the ends.

Short wall effects are included to some extent in code equations as increased concrete or masonry shear strengths for lower height/length ratios because the truss-like behavior mechanism is more efficient in resisting lateral loads than a cantilever mechanism in shear and flexure.

Fig 3-2: Shear Resistance of Short Walls

3.1.2 GEOMETRY

Typical shear wall geometries, shown in Figure 3-3, include rectangular, barbell, and flanged geometries (with the flanges extending laterally from the web in either one or both directions).

Rectangular sections cannot provide as much flexural

capacity as equivalent barbell or flanged sections. This is due to the limited amount of reinforcement that can be placed in the end regions of a rectangular wall. Special consideration, in the form of code height-to-thickness limits, must also be given to prevent lateral instability of rectangular walls, as this shape is more susceptible to buckling of the compression zone under severe load reversals.

The barbell-shaped section very often represents a wall between two columns. Column boundary elements have relatively large in- and out-of-plane stiffness. These elements also limit wall sliding shear at the foundation level by acting as large dowels. Since the embedded column boundary elements provide for vertical boundary reinforcement in the end regions of the cross-section, relatively high flexural capacities can be developed in this shape. In tall walls, where flexure frequently governs design, relatively high nominal shear stresses can be developed in barbell-shaped walls.

The flanged shape represents a section resulting from intersecting walls. As with the barbell section, the flanged shape can lead to a design with relatively high shear and flexural capacity. Unlike the barbell geometry, however, the flange shape does not always exhibit ductile behavior when web shear stresses are high. As web crushing occurs, the relatively thin flanges can separate from the web and buckle or shear, reducing their effectiveness and ultimate capacity.

Due to shear lag effects within the intersecting wall, only a portion of any intersecting wall length can be used for the flange width. Building codes provide little guidance for determining the effective flange width for concrete or masonry walls. References 3-13 and 3-14 provide guidelines for determining the effective flange width appropriate for modeling stiffness properties, however, these values are not necessarily appropriate for calculating the ultimate flexural strength. The desired failure mode is tension yielding of the flange (vs. a vertical shear failure where the web meets the flange), thus, the strength should be calculated using a flange width equal or somewhat less than the effective elastic width. An

Fig 3-3: Shear Wall Geometries
(Plan View)

appropriate design practice would be to assume a flange projection on either side of the "web" wall which is the lesser of the actual projection, one-third the wall height, or about four times the thickness of the projecting wall.

Rectangular and flanged geometries are generally adequate for short walls where the need for boundary elements is minor or absent. For tall walls or walls where substantial flexure occurs, barbell geometries are preferred, while flanged geometries may be used if adequate design capacity is given to the compression forces resisted by the flange sections, and the ability of the horizontal web reinforcing to transfer this force into the flange within the limited length available for development.

3.1.3 RIGIDITY

The rigidity of a shear wall is inversely proportional to its deflection under a unit horizontal force. Wall deflection is the sum of the deflections due to shear and flexure plus any additional displacement that may occur due to rotation at the base (due to foundation movements or other causes).

- Where shear walls are tied together by a rigid diaphragm or bracing so that all must deflect and rotate as a unit, the total translational lateral force in each wall is in direct proportion to its *relative rigidity* with respect to other walls, braces, or frames.
- Where shear walls are tied together by a flexible diaphragm or bracing, the total translational lateral force in each wall is in direct proportion to the *tributary area* of load which is transferred into each line of walls by the flexible diaphragm.

The impact on wall rigidity and stress distribution due to the number and sizes of any openings or penetrations must also be considered, as discussed below.

3.1.4 WALLS WITH OPENINGS

The presence of windows, doors, and service ducts in structures generally requires that exterior and interior shear walls be provided with openings or penetrations. It is important to good seismic performance that these openings interfere as little as possible with the moment and shear-carrying capacity of the walls.

The distribution and relative size of openings can greatly affect the distribution of force and relative rigidity of a shear wall. While small openings can be located randomly on the wall surface, large openings should be carefully located and properly evaluated. Where groups of large openings are randomly positioned, rather than aligned vertically or horizontally,

significant stress concentrations can occur, leading to serious weakness in the wall. As shown in Figure 3-4, poor seismic performance will result if insufficient attention is given to the size and placement of openings and to their effect on stress and rigidity.

Figure 3-5 shows typical deflection and crack patterns for a wall in which vertically arranged openings result in two shear wall sections which are coupled together by "spandrel" beam sections located above and below the openings. For walls where openings have been arranged to form spandrel panels, adequate horizontal and vertical reinforcing should be provided within the spandrel, or "coupling beam," zones to resist forces determined by an analysis which includes coupling effects.

The method of analysis used to determine the internal distribution of stresses around openings should reflect the probable deflection patterns and behavior of the wall. Approximate methods of analysis which are generally acceptable include:

- A relative rigidity analysis of piers and spandrels, based upon a relative rigidity analysis of the piers within the wall. This is commonly used for one or two story walls which contain a relatively small number of irregularly spaced openings.

Fig 3-4: Shear wall with Horizontal Openings

Source: Prof. Glen Berg, Alaska, 1964 (M8.4)

Fig 3-5: Deformation of Shear Wall with Openings

- A coupled wall-frame analysis, based upon an assumed pattern of deflection. This can be used only where openings have been arranged into vertical or horizontal rows.
- For very large openings, a frame analysis based on beam and column behavior may be appropriate. Considerations of both shear stiffness and joint rigidity should be included in the frame analysis.
- A detailed finite-element analysis of a wall with openings can provide acceptable results for unusual geometries. It should be recognized that the accuracy of the results are approximate, since assumptions regarding element type, mesh density, and foundation fixity and can cause results to vary considerably.

3.1.5 CRITICAL DESIGN ELEMENTS

Critical elements of shear walls subject to in-plane loading consist of:

- Web thickness and reinforcing, which resist shear forces.
- Tension and compression elements, located at each end of the wall and extending into the foundation or substructure, which resist overturning forces.
- Reinforcing dowels or bolts, located at floor and roof levels, which transfer shear forces from horizontal diaphragms into the wall.
- Reinforcing dowels, located at the bottom of the wall, which transfer loads from the wall into the foundation or substructure.

Critical elements of shear walls subject to out-of-plane loading consist of:

- Reinforcing dowels, bolts or straps located at both the top and bottom of the wall, which resist out-of-plane forces and prevent the wall from separating from the structure.
- Web reinforcing, which resists bending moments from out-of-plane forces.

3.2 REINFORCED MASONRY CONSTRUCTION

Masonry materials suitable for seismic-resistant construction include concrete masonry unit (CMU) and clay brick (CB) construction. Other material types, which are generally regarded

as unsuitable for seismic resistant construction due to their poor historic seismic performance, include adobe, glass block, stone, and hollow clay tile.

Since the strength of masonry construction can be greatly affected by the overall quality of construction, a major factor in the seismic design of masonry consists of understanding and specifying construction methods which will produce sound, high-quality construction.

3.2.1 BEARING vs. NON-BEARING CONSTRUCTION

Masonry walls can be constructed to support both vertical and lateral loads (bearing walls), to support lateral loads only (infill or screen walls), or to support neither (partitions).

Non-bearing construction usually requires an independent vertical load system of beams and columns. Types of non-bearing construction include:

- masonry infilled between columns of previously erected steel or concrete frames. Historically, masonry infill has performed poorly unless carefully designed and reinforced.
- masonry walls constructed with masonry pilaster elements at vertical load points.
- masonry walls with steel columns located adjacent to the wall at vertical load points.

Where masonry infill is used, a fully grouted joint with reinforcing steel is generally required around the entire perimeter of the infill panel to transfer shear forces into the masonry shear wall. Masonry infill panels in which flexible joint material is used around the perimeter are generally self-supporting and considered to be non-structural partitions. Details for partition systems are provided in Chapter 7.

3.2.2 SINGLE vs. MULTI-WYTHER

A wythe is a single row of masonry brick or block. Masonry can be constructed using a single wythe, or row, of masonry, or by using multiple wythes. Single wythe masonry typically consists of hollow-cell masonry units, shown in Figure 3-6. Walls are typically erected using the standard closed-end masonry unit shown at the top of the figure, although units which are open at one or both ends may also be used in order to facilitate masonry placement around pre-set vertical reinforcing, provide more clearance for grouting around reinforcing, or meet special requirements. At masonry rows where horizontal reinforcing is required, special bond-beam units are placed into which the reinforcing bars are positioned.

In multi-wythe construction, reinforcing bars can be either located within one or more wythes of hollow-cell masonry, or the reinforcing can be located within a minimum 2.5 inch (3.5 inch for high-lift construction) wide cavity constructed between adjacent masonry wythes. Horizontal reinforcing can consist of either rebar which is grouted together with vertical reinforcing into a cavity between wythes, or of mortar-joint reinforcing [Figure 3-8(a)] which is installed with the masonry. If the cavity between parallel wythes is used to contain reinforcing, as shown in Figure 3-7, it must be fully grouted. Otherwise, it can be left empty as an insulation and noise barrier. Multi-wythe walls with ungrouted cavities are referred to as "cavity walls." Adequate flashing and weep holes, as shown in Figure 3-10(a), must be provided wherever cavity spaces occur to control moisture condensation which will form on the cavity walls.

Multi-wythe walls with grouted cavities, and cavity walls in which all wythes contain reinforcing, should be treated as composite walls. In composite walls, loads are distributed to each wythe in proportion to its rigidity, and the rigidity of each wythe is calculated using both material and section properties.

For cavity walls which consist of one reinforced wythe plus one unreinforced wythe, the unreinforced wythe should be treated as anchored architectural veneer. All loads, both vertical and lateral, should be resisted by the reinforced portion of the wall.

3.2.3 UNREINFORCED WYTHES

Reinforcing is required for seismic resistance. Unreinforced wythes of masonry should not be used to support or resist structure loads, and is permitted only when treated as anchored architectural veneer. Seismic design considerations for anchored architectural

veneer are described in the following Section.

3.2.4 VENEER

There are two basic types of architectural veneer: anchored veneer [Figure 3-10(a)] and adhered thin veneer [Figure 3-10(b)].

Anchored veneer consists of a single wythe of standard masonry, which is secured to an adjacent structural wall by mortar-joint reinforcing [Figure 3-8(a)] or equivalent mechanical ties [Figure 3-8(b)] which connect adjacent wythes at appropriate spacing to laterally support the veneer against out-of-plane loads. Mortar-joint reinforcing consists of a continuous pre-welded wire lath which, when properly set into mortar, is able to function both to restrain out-of-plane forces and as a replacement to horizontal reinforcing bars. Wall ties are small wire anchors which only act to resist out-of-plane forces against masonry wythes. Ties should be of corrosion-resistant material, and adjustable ties (Figure 3-9) should not be used, as they bend easily and are prone to pull-apart failures.

In a veneer-clad wall, all required load carrying capacity, both vertical and lateral, is provided by the structural backup wall. Elastic joints or gaps should be provided at the top and along the two ends of the veneer panel to prevent transmission of vertical and seismic loads into the veneer. To ensure that the rigidity of the veneer does not significantly affect the overall structure behavior, the veneer wythe should be no more than 5 inches thick. On the other hand, to ensure that the veneer wythe is able to adequately span the distances between wall ties, it should be no less than 1.5 inches thick.

Adhered thin veneer consists of maximum 1 inch thick masonry units, which are adhered to a structural backing with mortar or thin-set grout. The structural backing can be of either metal stud or masonry construction. The thickness of adhered veneer should not be considered part of the effective wall section. The design of the structural backing should limit out-of-plane deflections to less than $1/480$ of the span when subjected to out-of-plane forces - wind or seismic. Any load capacity offered by the adhered veneer should be ignored in the design of the structure.

3.2.5 GROUTING

Grouting is required to anchor reinforcing to the masonry units and to produce a composite wall. Hollow-cell masonry units which contain reinforcing can be either grouted solid (fully grouted construction) or grouted only in the cells which contain reinforcing (partially grouted construction). Grout for all masonry types can be placed using either low-lift or high-lift techniques, as shown in Figure 3-11.

In seismic zones 1 and 2, hollow-masonry cells containing reinforcing are

located typically at about every 6th cell, for vertical bars, and at as much as 10 feet apart, for horizontal bars. For the quantity of grout required, it is typically worth the contractor's trouble to control the grout so that only the reinforced masonry cells are grouted.

However, in seismic zones 3 and 4, reinforcing is required at much closer spacings, particularly for horizontal reinforcing. Thus, contractors often find it less expensive to fully grout walls, even if drawings call for partial grouting. Since this increases the wall weight, and resulting out-of-plane inertia forces, it is recommended that fully grouted construction always be assumed when designing to these requirements.

Figure 3-12 shows typical details of a partially grouted low-lift wall. Low-lift grouting is the basic construction method defined in the UBC. It assumes that grout is placed in separate 4 to 5-foot lifts as the wall is erected. In multi-wythe walls, where reinforcing is located between the wythes, or in masonry constructed using open-end masonry units, the vertical reinforcing bars can be continuous (without lap splices), as shown in Figure 3-11, center. In reinforced closed-cell masonry, however, the vertical bars must be discontinuous (lapped), so that the height required to lift and place a masonry unit is reduced. This causes the vertical reinforcing to become highly discontinuous with lapped reinforcing, as shown in Figure 3-11, left side, which is undesirable for seismic resistance. Also, if the vertical

Fig 3-11: Grouting Methods

STRUCTURAL CONCEPTS AND DETAILS FOR SEISMIC DESIGN

REINFORCED HOLLOW MASONRY

PLAN VIEW

Fig. 3-12 : Typical Partial-Grouted Masonry with Low-Lift Grouting
After Reference 3-3

Fig. 3-13 : Typical Fully-Grouted Masonry with High-Lift Grouting

After Reference 3-3

reinforcing bar sizes exceed #6, the length of lap required becomes very inefficient or impossible to contain within the lift height unless mechanical reinforcing splices or open-end masonry units are used.

Although grout spaces are required to be cleaned out prior to grouting to remove any mortar droppings and debris, the use of cleanout openings at the base of the grout pour is not code-required for low-lift construction. Where inspection is not called for, and particularly where cleanouts are not used, the engineer should expect that the thoroughness of cleanout will be poor, which will affect the development of any lapped reinforcement in this area. Therefore, use of special lap lengths, inspection, or continuous vertical reinforcing with open-end masonry units should be considered.

The high-lift method of grouting, shown on the right side of Figure 3-11, assumes that both vertical reinforcing and grout are placed after an entire story-height of masonry is erected. Figure 3-13 shows typical details of a fully grouted high-lift wall. Due to the hydrostatic pressures involved when pouring high-lift grout, careful construction and grout pour timing is required, making it more difficult than low-lift construction. High-lift grouting is only suitable for fully-grouted walls and where openings, special reinforcing steel, or embedded steel details do not prevent the free flow of grout or inhibit the use of mechanical vibration to properly consolidate the grout. Cleanout openings, located in cells intended to contain vertical reinforcing, are required at the base of each grout lift in order to clean out mortar droppings and inspect reinforcement positioning prior to grouting. Cleanout openings are generally formed by breaking the face from a masonry unit, and must be patched or sealed prior to grouting. The grout mix is required to be a high-slump mix to permit pumping and to fill all voids. To counteract grout shrinkage, the use of a suitable shrinkage-compensating grout admixture is recommended.

Comparing low-lift with high-lift construction, low-lift grouting is often an easier and preferred construction method for the contractor, since there is less chance that problems will occur during grouting. For hollow-cell masonry, it is a less desirable method if the vertical reinforcing is not continuous and if mortar droppings are not properly removed prior to grouting. If high-lift grouting is specified, increased clearance must be provided around reinforcing to permit grout placement.

3.2.6 INSPECTION

For all seismic zones, the UBC requires special (continuous) inspection during construction for all masonry designed using the full allowed masonry strength values, but permits construction without special inspection (but with periodic inspection), if design is done using half of the normally allowed masonry strength (f'_m) values. The UBC also permits this alternative for hazardous or essential ($I = 1.25$) facilities. For DOE facilities, UCRL-15910

permits periodic inspection for general use ($I = 1.0$) facilities, but requires special inspection for important or low-hazard ($I = 1.25$), moderate-hazard, and high-hazard facilities.

When masonry is designed using the full f'_m value, the UBC requires special inspection plus either prism or unit testing of masonry material both before (typically by the supplier of materials) and during the masonry construction (i.e., field testing). When masonry is designed using half of the f'_m value, the UBC does not require either special inspection or field testing, but requires that the supplier of the materials provide a certification-of-conformance letter, together with recent test results, to the job site.

Special inspection of masonry consists of continuous, full time inspection by an independent, trained technician during:

- preparation of masonry wall prisms (both before and during construction),
- sampling and placing of all masonry units,
- placement of reinforcement,
- inspection of grout spaces,
- immediately prior to closing of cleanout openings, and
- during all grouting operations.

The UBC specifies that the special inspector must be employed by the owner (or the engineer or architect acting as the owner's agent), thus avoiding the possibility of conflict of interest if the inspector were employed by the contractor. The special inspector is intended to be a person specially qualified for the type of inspection required. The inspector must demonstrate competency to the satisfaction of the building official; generally this involves an examination which determines that the inspector not only knows the trade of work performed but also the code requirements for the work. In this regard, the International Conference of Building Officials (ICBO) maintains a program of testing and certification for special inspectors which is recognized by most municipalities.

During the performance of work, the special inspector acts as a deputy to the building official and must furnish inspection reports to the building official, engineer, and other designated parties. Discrepancies must be immediately called to the attention of the contractor for correction; if uncorrected, the design engineer and building official must be notified. When the inspection is complete, the UBC requires that the inspector submit a final report to the building official and engineer stating that to the best of his knowledge the work which was inspected complies with the plans and specifications and the code.

Periodic inspection consists of inspection visits which are made at specific milestones during the construction, as outlined in the project plans and specifications and approved by the building official. No specific periodic inspections are specified for masonry by the

however, a minimum level inspection is strongly recommended for all masonry construction which should include inspection:

- during the preparation of masonry prisms or taking of test specimens,
- during the course of masonry placement, if joint reinforcing or veneer wall anchors are used,
- after all masonry and reinforcing is placed and grout spaces are cleaned out, but immediately before any cleanouts are closed and grout is placed, and
- immediately after grouting.

Fig 3-14: UngROUTed Masonry Wall

Source: George P. Simonds, Kern County, 1952 (M7.7)

The importance of good workmanship and careful supervision of all construction in high-seismic regions cannot be overemphasized. Figure 3-14 shows an example of a portion of earthquake-damaged masonry wall in which the contractor had neglected to pour grout into the masonry after the reinforcing was installed (thus making the reinforcing completely ineffective). A similar wall caused the collapse of a two story penthouse on the roof of the AMFAC Hotel during the 1989 Loma Prieta Earthquake. Although building codes permit masonry designed on a half-strength basis to waive inspection, seismic performance frequently depends upon the performance of building details or on construction practices which are not highly dependent upon masonry strength.

Inspection is especially significant for masonry detailed in accordance with seismic zone criteria which does not match the general construction practice for the local region (such as might occur for the design and construction of Moderate or High Hazard DOE facilities in relatively low seismic regions in accordance with UCRL-15910 provisions). Quality problems may arise because of attempts to impose construction methods which differ from traditional regional construction practices. In these instances, pre-construction meetings, coupled with inspection requirements, are strongly recommended.

3.2.7 CONTROL JOINTS

Control joints, shown in Figure 3-15, are continuous vertical joints which are provided in walls to accommodate movements resulting from thermal and moisture changes in the

DETAILS FOR WALL SYSTEMS

Fig. 3-15 : Typical Control Joint Details

After Reference 3-3

masonry. Traditionally, concrete masonry walls, which are subject to some drying shrinkage with age, are constructed with control joints located about 20 to 40 feet apart. Clay brick walls, which tend to absorb moisture with age, and therefore expand, are constructed with expansion joints located about 30 to 60 feet apart. Because grouted multi-wythe walls also require full-height vertical cavity barriers of masonry at maximum spacings of about 30 feet in order to control the horizontal flow of grout, control joints are generally combined with this detail in multi-wythe construction.

The use of expansion and contraction control joints in masonry is sometimes viewed as an optional item by engineers, particularly in seismic zone 3 and 4 areas (where a large amount of horizontal reinforcing helps to control cracking). Walls constructed without these joints may be expected to experience some degree of cracking.

The UBC requires that the diaphragm chord reinforcement (the horizontal diaphragm boundary reinforcement required at floor and roof levels) must be made continuous, even across expansion and contraction control joints. This is required even if the joints are located at points where the diaphragm moment, and thus chord force, is calculated to be zero. Because other horizontal reinforcing, required for either shear forces or temperature and shrinkage control, is terminated at the control joint, each section of wall spanning between control joints should be designed and reinforced as an individual shear wall. The determination of shear forces within the wall segment should be based upon the relative rigidity of each wall segment with respect to the total length of wall.

Figure 3-16 shows recommended details for the detailing of reinforcing which is continuous through control joints. In order to provide some axial elasticity to the continuous reinforcing bars to enable joint movement without causing high stresses in the bars, it is recommended that a bond-break be provided along a short length of the reinforcing on either side of the joint, as shown.

3.2.8 DESIGN METHOD

Two separate methods of masonry design are defined in the UBC: the working stress design (WSD) method and the ultimate strength design (USD) method.

The WSD method can produce two separate types of designs, depending on shear stress levels calculated in the wall. For walls subject to only low shear stresses, all loads are assumed to be resisted by the masonry shear strength, and the code-specified minimum reinforcing is not assumed to resist any loads. For walls subject to high shear stresses (in excess of the masonry shear strength) the reinforcing is required to resist all loads.

Recent earthquake experience has led some researchers to suggest that code requirements for horizontal steel should be increased for masonry designs in which reinforcing is required to resist all loads. A 20% increase in the amount of horizontal steel above that required is considered prudent for these designs. For designs in which masonry shear strength is adequate to resist loads, earthquake experience has been relatively good for masonry of good quality construction. Care should be taken, however, to furnish code-required minimum reinforcing to provide a ductile backup system for the masonry.

In contrast to the WSD method, the USD method assumes that shear stresses are resisted by both masonry and reinforcing strength, although the fraction which masonry is permitted to resist varies depending on relative wall height, and can equal zero. Because the USD method is relatively new, little earthquake experience exists to judge performance.

3.2.9 SEISMIC ZONE 2 REINFORCING

The UBC specifies minimum seismic detailing requirements for masonry construction. Figure 3-17 and the following summarize minimum requirements for seismic zone 2 detailing:

- Vertical reinforcement - at least one #4 (0.20 in²) located: 1) at each end of the wall, 2) at corners, 3) at each side of openings, and 4) at maximum spacings of 4 feet apart. Detailed calculations may require increased reinforcement in the following areas:
 - a. Vertical reinforcing at ends of walls and adjacent to openings may increase due to shear wall chord forces.
 - b. The size or spacing of vertical reinforcing may increase due to bending moments caused by out-of-plane forces.
- Horizontal reinforcement - at least one #4 (0.20 in²) located: 1) at top and bottom of openings, 2) continuously at structurally connected roof levels, floor levels, and at the top of the wall, 3) at the bottom of the wall or in

Fig. 3-17 : Seismic Zone 2 - Minimum Recommended Reinforcing

the top of the foundation, and 4) at maximum spacings of 10 feet apart. Specific design requirements may increase reinforcement in the following areas:

- a. Continuous reinforcing is required at roof and floor levels due to diaphragm chord or drag strut forces. When additional reinforcing is provided in a local area, such as in an area of maximum diaphragm chord forces, at least 50% of this reinforcing should be continuous throughout the wall, including points where calculated chord forces are zero.
- b. Reinforcing required above openings may increase due to vertical loads (lintel beam reinforcing).
- Reinforcing located around openings must extend at least 24 inches or 40 bar diameters beyond the opening.
- When masonry is placed in a "stack bond" pattern (i.e., with no offsets in the vertical masonry joints), horizontal reinforcing at least equal to $0.0007bt$ is required (where: b = wall length and t = wall thickness), using either ladder- or truss-type joint reinforcing, or rebar.

In addition, the UBC specifies additional requirements for reinforced masonry walls, including the following:

- Vertical shear stresses should be considered in spandrel or lintel areas above openings and additional vertical reinforcing added in these areas if required.
- In areas where the calculated shear stress " f_v " exceeds the allowed masonry shear stress " F_v :"
 - a. Horizontal wall reinforcing must be provided to resist the total shear force. The maximum reinforcing spacing is specified as $b/2$, where " b " is the length of the wall segment.
 - b. Shear reinforcing must be anchored with a standard hook at each end.

In seismic zone 2, in masonry walls where masonry strength is assumed to resist all lateral loads, the code specified reinforcing spacing may exceed the width of tall, narrow wall segments. In order to control a possible fracture of the masonry in this area, additional horizontal ties are recommended at a spacing of about $1/2$ of the length of the wall pier (as is required when reinforcing steel resists shear stresses).

3.2.10 SEISMIC ZONE 3 AND 4 REINFORCING

For seismic zones 3 and 4, all applicable zone 2 requirements apply, plus the following:

- The minimum reinforcing area in each direction is $A_s = 0.0007bt$, spaced a maximum of 4 feet apart.
- The minimum combined horizontal and vertical reinforcing area must be at least $A_{s \text{ horiz}} + A_{s \text{ vert}} = 0.002bt$.
- The use of either rebar or ladder- or truss-type joint reinforcing is permitted as shear reinforcing, however, the engineer is cautioned that the use of joint reinforcing requires ongoing inspection during the masonry placement to ensure proper reinforcing placement (whereas the placement of rebar can often be inspected all at once, after the masonry is placed but before the grout is placed).
- If the WSD method is used, masonry shear wall design forces must be increased by 50%.
- Where concrete abuts masonry, concrete surfaces must be roughened to 1/16". Vertical joints must be treated similar to "stack bond" construction; either use open end grouted units or provide minimum 0.0015bt horizontal reinforcing.

3.2.11 INTERSECTION AND CORNER DETAILS

Figure 3-18 shows typical details suitable for seismic-resistant construction. Reinforcement should be located to provide for development of the horizontal reinforcing across the

intersection. This will prevent separation of the intersection, and develop the capacity of the horizontal reinforcing into the point where the wall chord reinforcing is located.

3.3 CAST-IN-PLACE CONCRETE CONSTRUCTION

Basic requirements for ordinary concrete shear walls are described in Sections 3.3.1 through 3.3.3. Special requirements pertaining to boundary elements, coupling beams, and wall piers are described in sections 3.3.4 through 3.3.6.

3.3.1 SEISMIC ZONE 1 AND 2 REQUIREMENTS

At least one #5 bar is required at the top, bottom, and at each side of walls, plus not less than 2 #5 bars around all openings, extending a minimum of 24" beyond the openings.

Two curtains of reinforcing are required if calculated shear stresses exceed $2\phi\sqrt{f'_c}$ or if the wall thickness is 10" or greater.

The maximum factored design shear force permitted for concrete shear walls is limited to:

$$\begin{aligned} V_u &\leq (8\phi\sqrt{f'_c}) dh && \text{for all walls sharing a common lateral force, or} \\ V_u &\leq (10\phi\sqrt{f'_c}) dh && \text{for any individual wall or pier.} \end{aligned}$$

The total shear strength of shear walls is calculated as:

$$V_u \leq \phi (V_c + V_s)$$

where,

$$V_s = (p f_y) d h$$

$$V_c = (2\sqrt{f'_c}) d h \quad \text{normally, although other values are permitted as described below}$$

$$\phi = 0.85$$

$$f'_c = \text{concrete compressive strength, psi}$$

$$f_y = \text{reinforcing yield strength, psi}$$

$$p = \text{the ratio of reinforcing steel area provided to gross concrete area}$$

$$d = 80\% L_w, \text{ unless a strain-compatibility analysis is performed to justify a higher value.}$$

$$L_w = \text{Wall length}$$

$$h = \text{thickness}$$

The UBC permits alternative, higher, V_c values if analysis is performed using more detailed equations provided in the code. Any such increases should be taken with caution, since little earthquake experience exists to support the equations provided for lower seismic zone

criteria. A preferable alternative is to use the V_c equations specified for seismic zone 3 and 4 areas.

The minimum area of horizontal reinforcing steel provided must be at least equal to 0.0025 times the wall height times thickness. The minimum reinforcing area can be split between reinforcing layers when two curtains of reinforcing are used. Actual reinforcing required may be greater depending on shear strength (V_s) requirements. The maximum spacing of horizontal web reinforcing is the lesser of $L_w/5$, $3h$, or 18".

The minimum area of vertical web reinforcing steel required varies, depending upon the height/length, H_w / L_w ratio of the wall and the quantity of horizontal reinforcing required, as follows:

- For tall walls ($H_w / L_w \geq 2.5$), the minimum amount of vertical reinforcing, p_v is 0.0025 of length times thickness.
- For short walls ($H_w / L_w \leq 0.5$), the minimum vertical reinforcing, p_v is equal to the actual horizontal reinforcing, p_h provided.
- For intermediate height/length values, the required vertical reinforcing, p_v varies linearly between these values, in accordance with the following formula:

$$p_v = 0.0025 + 0.5 [2.5 - (H_w / L_w)] (p_h - 0.0025)$$

The minimum reinforcing area can be split between reinforcing layers when two curtains of reinforcing are used. The maximum spacing of vertical web reinforcing is the lesser of $L_w/3$, $3h$, or 18".

3.3.2 SEISMIC ZONE 3 AND 4 REQUIREMENTS

At least one bar, the larger of #5 or two times the area of the required shear reinforcing per lineal foot of wall in size, is required at the top, bottom, and at each side of walls. In addition, not less than 2 #5 bars are required around all wall openings, extending a minimum of 24" beyond the openings.

Two curtains of reinforcing are required if calculated shear stresses exceed $2\phi\sqrt{f'_c}$ or if the wall thickness is 10" or greater.

The minimum specified area of reinforcing steel in each direction is equal to 0.0025 times thickness times length, with a maximum spacing between bars of 18". The minimum reinforcing area can be split between reinforcing layers when two curtains of reinforcing are used. Actual calculated stresses may require greater amounts of reinforcing.

The maximum factored design shear force permitted for concrete shear walls is limited to:

(Note that total wall length, L_w is used in the following equations instead of the distance to the center of the chord reinforcing, d .)

$$V_u \leq (8\phi\sqrt{f'_c}) L_w h \quad \text{for all walls sharing a common lateral force, or}$$

$$V_u \leq (10\phi\sqrt{f'_c}) L_w h \quad \text{for any individual wall or pier.}$$

The total shear strength of shear walls is calculated as:

$$V_u \leq \phi (V_c + V_s)$$

where:

$V_s = (p f_y) L_w h$ the strength contributed by the reinforcing, and

$V_c =$ the strength contributed by the concrete, as follows:

$V_c = (2\sqrt{f'_c}) L_w h$ where the ratio of height/length ≥ 2 ;

$V_c = (3\sqrt{f'_c}) L_w h$ where the ratio of height/length ≤ 1.5 ;

With linear interpolation between these values.

Other terms are:

$\phi = 0.60$, if the nominal shear strength is less than the shear corresponding to development of the nominal flexural strength of the wall (e.g. if shear governs the strength of the wall). Generally controls for short walls.

$\phi = 0.85$, if flexure governs the strength of the wall. May control for a tall wall.

$f'_c =$ concrete compressive strength, psi

$f_y =$ reinforcing yield strength, psi

$p =$ the ratio of reinforcing steel area provided to gross concrete area

$L_w =$ Wall length

$h =$ thickness

Although the seismic zone 1 and 2 equations are based on a reduced wall area (using d vs. L_w times thickness), the seismic zone 3 and 4 shear equations are based on the gross wall area (total wall length times thickness), excluding pilasters. This reduction in conservatism is usually offset by the reduced value of ϕ required.

3.3.3 CONSTRUCTION JOINTS AND WALL INTERSECTIONS

Adequate shear transfer must be provided at construction joint locations to transfer all loads through the joints. The design of construction joints is generally done using shear-friction concepts. Most joint types perform adequately when made following standard practices of roughening and cleaning the surface of previously cast concrete by bushhammering or sandblasting to remove laitance, loose particles, and construction debris.

Typical reinforcing details for wall intersections are shown in Figure 3-19. Reinforcement should be located to provide for as much development of the horizontal reinforcing into the intersection as possible in order to prevent separation of the walls, and to develop the capacity of the horizontal reinforcing into the point where the walls vertical chord or boundary reinforcing is located.

Seismic zone 3 and 4 requirements call for special detailing of horizontal reinforcing at wall ends when design shear stresses exceed $\sqrt{f'_c}$. At wall ends, horizontal reinforcing must either be provided with a standard hook (90°, or 135° to 180°) which encloses the vertical chord reinforcement, or the chord reinforcement must be enclosed by horizontal "U" stirrups of the same size and spacing as, and spliced to, the horizontal reinforcement.

Seismic zone 3 and 4 criteria also require that horizontal reinforcing splices in shear walls must be staggered so that no more than 50% of the horizontal reinforcing is spliced at any point. Although the minimum stagger distance between splices is not indicated, a clear distance of at least 24" is recommended between the spliced regions.

Where significant out-of-plane forces cause large bending moments at wall intersections (such as in liquid storage tank walls), additional diagonal reinforcing bars should be added across wall intersections, with size and spacing to match the wall horizontal reinforcing, as

shown in Figure 3-20. Due to the quantity of reinforcing in this type of joint, it is advisable to locate construction pour joints away from the intersection to a point at about 20 to 30 percent of the wall's lateral bending span.

Most wall corners are formed with 3/4 to 1 inch chamfers in the concrete surface in order to protect the concrete finish from broken edges. Where unusually large diameter reinforcing bars are used due to stress levels or massive wall thickness, the radius of bend of the reinforcing may cause the concrete cover in these corners to become excessive, as shown in Figure 3-21. At these locations, either increased chamfer sizes or additional smaller diameter corner reinforcing should be specified.

3.3.4 BOUNDARY ELEMENTS

For seismic zone 3 and 4 detailing, special boundary element reinforcing is required along edges of walls and around openings when wall vertical loads (for bearing walls) and overturning moments are sufficient to cause compression stresses in the wall to exceed $0.2 f'_c$, based on a $P/A \pm Mc/I$ analysis using gross section properties. These elements are not required for seismic zone 1 or 2 detailing, although adequate wall thickness and reinforcing must be provided to resist loads. Figure 3-22 shows an elevation of a shear wall with typical spandrel beam and boundary element reinforcing.

Boundary elements can be located within the thickness of the wall, or as "barbell" or pilaster, elements, as shown in Figure 3-23. When located within the wall, the hoop reinforcing provides protection for boundary reinforcing splices and confines a core of concrete against spalling to resist compressive loads. A barbell geometry provides greater compression capacity and is required if the compression force is sufficient to buckle a rectangular wall.

Boundary reinforcing consists of vertical column reinforcement, column hoop reinforcement, and cross ties. The ratio of vertical reinforcing (A_{st}) to gross concrete column area (A_g), must be between a minimum of 1 and a maximum of 6 percent.

The total compression capacity of the confined boundary element, treated as a short column, must be checked using the following equation to verify that the provided boundary elements are capable of resisting the total applied vertical load (i.e., the web area of the wall is not load bearing).

$$P_u \leq 0.80 \phi [0.85f'_c(A_g - A_{st}) + f_y A_{st}]$$

where:

P_u = the factored vertical load supported by the boundary element

A_g = the gross concrete area of the boundary element

A_{st} = the area of vertical reinforcing in the boundary element

ϕ = 0.70, if hoop reinforcing is used to confine vertical reinforcing

ϕ = 0.75, if spiral reinforcing is used to confine vertical reinforcing

Additional vertical reinforcing and/or concrete area (producing a barbell configuration) must be provided where compression loads are too high to meet this requirement.

Boundary element vertical reinforcing is normally required to meet low-alloy A706 steel requirements, but Billet Steel A615 Grades 40 or 60 may be used if special yield and tensile strength requirements are imposed.

A special type of boundary element, shown in Figure 3-24, is required when columns are used to support and resist overturning moments from discontinuous walls (such as caused by an "in-plane discontinuity" type of vertical irregularity or an "out-of-plane offset" type of horizontal irregularity), and if the axial force in the columns exceeds $0.1f'_c A_g$.

Special transverse reinforcing is required over the full height of the columns and continuing over the development length of the column's vertical reinforcing both above and below the column. This reinforcing is the same as that required for concrete columns in moment-resisting frames, and is described in Chapter 4, for both seismic zone 2 and seismic zones 3 and 4 requirements.

For seismic zones 3 and 4, in addition to all other applicable loading conditions, these columns must have the strength to resist the *axial force* resulting from the following load combinations:

$$1.0 \text{ DL} + 0.8 \text{ LL} + 3 (R_w / 8) E$$

$$0.85 \text{ DL} + 3 (R_w / 8) E$$

Fig 3-24: Columns at Discontinuous Walls

3.3.5 COUPLING BEAMS

In seismic zones 1 and 2, reinforcement for wall spandrel panels is required to be designed to resist forces as determined by a rational analysis. In seismic zones 3 or 4, the UBC requires that additional flexural and shear reinforcement, as shown in Figure 3-26, be provided for coupling beams where all of the following occurs:

- the ratio of coupling beam length-to-depth (L/d) is less than 4,
- factored shear stress V_u exceeds $4\sqrt{f'_c}$, and
- the spandrel panel is used to interconnect shear walls.

Coupling beam reinforcement consists of symmetrical diagonal shear reinforcement, extending across the full length of the member and located within the transverse beam hoop reinforcement of the spandrel beam. The area of each reinforcing diagonal, as shown in Figure 3-25, is determined as:

$$A_s = V_u / [2 f_y \sin \alpha]$$

where:

$$\tan \alpha = [h - 2d'] / L_s$$

Fig 3-25: Coupling Beam Geometry

The flexural strength contribution of the added diagonal reinforcing must be included into the flexural capacity calculation of the spandrel beam.

Horizontal and hoop reinforcing provided in the coupling beam is generally only nominal reinforcing equal to about 0.25% of the section area in each direction. Confining hoop reinforcing should also be provided around the diagonal reinforcing bars in order to confine a concrete section with thickness adequate to prevent buckling of the diagonal bars undergoing compression.

3.3.6 WALL PIERS

As the number and sizes of openings provided in shear walls increases, the structural behavior of the wall system approaches that of a frame system, for which special ductile reinforcing is required (as described in Chapter 4, Frame Systems). As a transition between walls and frames, the building code defines special reinforcement for short wall segments. These reinforcing requirements are defined by the ratio between clear pier height to horizontal length, and horizontal length to wall thickness.

For all seismic zones, wall segments with horizontal length-to-thickness ratios less than $2\frac{1}{2}$ must be designed and reinforced as columns.

Wall segments with horizontal length-to-thickness ratios between $2\frac{1}{2}$ and 6, and with clear height at least two times horizontal length, are defined as "wall piers." Special lateral reinforcing is required for seismic zone 3 and 4 criteria, as follows:

- The amount of lateral reinforcing must be adequate to resist shear force corresponding to the probable flexural strength of the wall pier. Probable flexural strength is calculated based on vertical loads and the upper-bound yield strength of the vertical reinforcing specified.
- When the axial compressive force in the wall pier, including earthquake effects, is less than $A_g f'_c / 20$, the transverse reinforcement may have standard hooks at each end in lieu of hoops. The spacing of transverse reinforcement must not exceed 6 inches.
- The transverse reinforcement must extend beyond the clear height of the wall pier for at least the development length of the largest vertical wall pier reinforcing size used.

The requirement for special wall pier reinforcing is waived when the wall pier is a not part of the lateral-force resisting system, or other shear wall segments along the same wall line provide lateral support to the wall piers and have a combined stiffness at least six times that of the sum of all wall piers along that wall line (e.g., the wall piers resist only a small fraction of lateral loads).

3.4 CONCRETE TILT-UP CONSTRUCTION

3.4.1 INTRODUCTION

Tilt-up construction is a type of precast construction in which wall panels are site-cast on a level surface and then lifted or tilted onto prepared foundations to form building walls. Figure 3-27 depicts a typical tilt-up construction sequence. Because the wall panels are reinforced concrete, all of the design and detailing rules described in Section 3.3 for the in-plane (shear wall) design of tilt-up walls apply. However, because the construction and behavior of tilt-up walls is unique and different from cast-in-place concrete construction, many construction details are different.

Typically, the thickness of tilt-up walls ranges between 6 and 9 inches, and normally only a single curtain of reinforcing is used. This reinforcing curtain is generally located near the center of the wall, however, the actual distance of the reinforcing from the casting surface (normally the exterior wall face) is generally required to be a standard increment for an available reinforcing support chair (since the wall is poured horizontally).

In early tilt-up construction, panels were connected together with reinforced pour strips to form a continuous structural wall. In most current construction, this step is avoided in order to improve thermal expansion and concrete contraction behavior, and to reduce the expense of the connections. Instead, wall panels are typically connected only at the roof and floor levels, where continuous reinforcing is required, and then sealed with flexible caulking, as shown in Figure 3-28.

Where a building wall is formed by a row of tilt-up panels which are connected only at chord points, as shown in Figure 3-29, the structural behavior of each panel is that of an independent shear wall. Forces are distributed to each panel in proportion to each panel's rigidity relative to total rigidity of all panels in that row. The rigidity of each panel should consider both the effects of any openings and the foundation type and its effect on wall rigidity.

Earthquake experience has shown that the connections between wall panels and to other portions of the structure are critical to the performance of tilt-up structures. These connections have historically been shown as weak points in the structural system, which are capable of resulting in separations between panels and in separations from the roof and floor diaphragms.

Fig 3-27: Tilt-Up Construction Sequence

Fig 3-28: Tilt-Up Joint Details (Plan View)

The design of the panels should not only consider forces which are applied parallel to the wall, but also forces which are applied out-of-plane. Slender wall panels (walls with unsupported height greater than 25 times the thickness) should be designed for out-of-plane forces using the slender wall criteria provided in the UBC, and adequate anchorage, suitable to the type of construction, should be provided at roof and floor levels to resist the resulting reactions. The use of diagonal braces extending from roof beams to the wall (as a measure to reduce the unbraced wall length) is *not* recommended, particularly for roofs with flexible diaphragm systems. Lateral deformations of the roof diaphragm could cause a diagonal brace to induce lateral bending forces into the wall which it might not be able to withstand.

Connection details which are important to seismic performance include:

- details for shear transfer and out-of-plane restraint at roof and floor levels, described in Chapter 5 (Diaphragm Systems),
- the chord connection detail, described in Section 3.4.2,
- details for shear transfer at the base of the wall, described in Section 3.4.3, and
- foundation uplift anchorage (required only where design forces cause uplift to occur), described in Section 3.4.3.

Fig 3-30: Typical Wall Chord Connection Details

3.4.2 CHORD CONNECTIONS

Structural connections between panels are usually made using welded chord bar splices at the roof and floor levels. Typical connection details for chords are shown in Figure 3-30. Because these connections typically involve welding of Billet A615 reinforcing steel, the "Carbon-Equivalent" (CE) must be determined, based upon a chemical analysis of the reinforcing steel, in order to determine the suitability of welding and preheat requirements.

Chord connections should be as direct as possible and should minimize eccentricity. Figure 3-31 shows a section through a chord detail which utilizes a series of lapped and welded plates to transfer forces. In a moderate earthquake, lateral bending of the eccentric connection caused the lateral bursting around connections shown in Figure 3-32.

3.4.3 WALL BASE CONNECTIONS

Generally, tilt-up wall panels are cast with lateral reinforcing near the base which is then tied into the floor slab system via a narrow slab "pour strip." Commonly, reinforcing dowels are *not* provided between the floor slab and the wall foundation system, as shown in Figure 3-33. This assumes that the floor slab is able to function as a rigid diaphragm which is able to transfer lateral shear forces from the wall into the subgrade beneath the floor slab.

The detail shown in Figure 3-33 is *not* recommended for situations where construction joints or openings in the floor system may prevent proper diaphragm behavior. In this condition, additional reinforcing dowels should be provided to carry the force from the pour strip into the foundation system, where sliding friction and/or passive soil pressure provides resistance. A typical connection detail for this alternative is shown in Figure 3-34. Because it provides a more direct force-transfer mechanism, the detail shown in Figure 3-34 is preferred in Seismic Zones 3 and 4.

These details are suitable for the transfer of shear force only; where uplift occurs additional details are required.

Where wall uplift occurs due to overturning moments, uplift connections are provided at the ends of the panels, as shown in Figure 3-35. When checking for uplift using working stress procedures, vertical dead loads which act to reduce uplift

Fig 3-35: Wall Uplift Connection

forces must be taken as 0.85 of dead load. If ultimate strength procedures are used, uplift forces should be based on a factored load combination of $U = 0.9D - 1.4E$.

Wall uplift connections are relatively expensive in comparison to other aspects of tilt-up construction. While alternate details can be designed which are less complex than the detail shown in Figure 3-35, these details generally introduce either eccentric loading or requirements for field welding to chord reinforcing bars.

Uplift connections are usually required at very few wall panels, if at all. This is because tilt-up buildings typically have a large number of relatively solid wall panels on each wall line and, therefore, do not experience significant uplift. Where specific design conditions indicate that an extensive number of uplift connections are required, it is an indication that the overall building shear wall system is relatively highly stressed. In this situation the design engineer should re-think whether an adequate number of resisting elements have been provided for the seismic system.

3.4.4 WALL PIERS

The geometry of the individual wall panels must be carefully laid out to provide strong, stable and liftable sections. Excessive openings within the panels can cause panel behavior to resemble a moment-frame system, rather than a shear wall system, which may give poor seismic performance. The UBC restricts this behavior by defining short wall segments as "wall piers." Design requirements for concrete wall piers are described in Section 3.3.6.

Figure 3-36 is an elevation of a wall with substantial frame behavior. The short wall segments located between openings would probably require design as wall piers. In addition, if the vertical load stresses in the piers exceeded $0.2f'_c$, these elements would require boundary element reinforcing.

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DETAILS FOR MOMENT-FRAME AND BRACED-FRAME SYSTEMS

This chapter discusses seismic detailing requirements for moment-frame and braced frame systems. Section 4.1 provides an introductory description of the general performance characteristics of moment- and braced-frame systems. Section 4.2 presents requirements for concrete moment-frames. Sections 4.3 to 4.7 address general steel design, steel moment-frames, steel braced-frames (both concentric and eccentric), and pre-engineered metal building systems, respectively.

4.1 MOMENT AND BRACED-FRAME SYSTEMS

4.1.1 MOMENT FRAME BEHAVIOR

Lateral forces in moment frames are resisted by bending and shearing of columns and beams, which are connected by rigid joints. Joints can become highly stressed, and the details of their construction are important. Energy absorption through substantial deformation of the frame in the inelastic range is an important aspect of seismic resistance. For this reason, moment-resisting frames often utilize steel with rigid, welded joints in order to take advantage of the natural ductility of the material. Properly reinforced concrete frames have also been accepted as ductile frames.

The basic philosophy of seismic design is that sudden or brittle failure modes should be prevented by concentrating inelastic deformations in regions having a capacity for ductile behavior. The general failure modes of moment-frames can be categorized as either weak-beam (Figure 4-1) or weak-column (Figure 4-2). The weak-beam mechanism provides a

much greater energy-absorption capability and a greater margin against collapse than the weak-column failure mode for several reasons. First, the total number of plastic hinges required to develop a complete weak-beam collapse mechanism is always equal to or greater than the equivalent number of weak-column hinges, even for one-story structures. Second, weak-column hinges tend to form in all members nearly simultaneously, whereas weak-beam hinges tend to initiate in one area (usually the base) and then propagate. Finally, because weak-beam hinges form progressively, the structure experiences a gradual lengthening in the structure period (due to the increased sidesway), which generally acts to decrease the seismic forces input to the frame. Weak-column systems do not experience a significant decrease in seismic forces due to period-shift, because hinges tend to form simultaneously.

In order to ensure weak-beam behavior, the columns and other non-ductile elements must be designed by proportioning their strength to the calculated beam strength, instead of by the specified design forces. Hence, the design engineer must design the beams first. Any unnecessary conservatism applied by the engineer when selecting beam sizes has a proportional (and possibly adverse) effect throughout the remaining design, since the size and strength of many elements are dependent upon the beam selection. Beams which contain mechanical openings may become weakest at the cutouts, and so must be carefully reinforced.

Moment-resisting frame systems tend to be much more flexible than shear wall or braced frame systems. This flexibility must be accounted for in the design of architectural elements such as curtain walls, partitions, and ceilings.

Because moment-frames are very flexible, deflection limitation criteria frequently govern the design, and deflection calculations must be reasonably accurate. Calculations for deflections must include all significant components of deflection. The most significant components are:

1. Column bending due to flexure and shear.
2. Joint rotation due to girder and joint stresses.
3. Distortion of the frame due to column axial deformation.

In addition, P-Delta effects can be a significant design factor, potentially leading to instability. The P-Delta effect is the secondary effect on shears, axial forces and moments on frame members induced by the vertical loads acting on the laterally displaced frame. The UBC requires that P-Delta effects must be included if they affect results by more than 10%. However, in Seismic Zones 3 and 4, P-Delta effects need not be considered where the story drift ratio does not exceed $0.02/R_w$.

Fig. 4-3,
Buckled Bracing
Source: Reference 4-7

Fig. 4-4, Failed Connection,
San Fernando, 1971
Source: Reference 4-11

4.1.2 BRACED-FRAME BEHAVIOR

Braced-frames resist vertical loads as trusses cantilevered from the foundations or lower stories. Braced frame columns act as chords, and floor beams and diagonal braces act as web members of the truss. For single-story structures, X-bracing made of slender rods or light rolled sections as tension-only elements are often used. For multi-story construction, heavier diagonal braces of rolled steel sections or tubes resist both tension and compression.

Braced-frames typically are (or should be) designed to minimize eccentricities at connection points to maintain axial load transfer and avoid excessive bending stresses. This type of framing system is known as the "Concentric Braced Frame" (CBF). The CBF system can exhibit poor inelastic performance (as shown in Figures 4-3 and 4-4) unless significant safety margins are imposed on members and connections within such systems. Recent UBC and other steel design specifications contain provisions intended to assure safe performance of CBF structures.

When CBF framing is proposed for structures greater than two stories in height, it is recommended that a dual system be considered. A dual system has moment-resisting connections at beam-column joints to create a "back-up" moment-frame within the CBF.

An alternative braced-frame system is the "Eccentric-Braced Frame" (EBF). An EBF is similar to a CBF except that the working points of the bracing are offset from the beam-column joint along the beam to create regions of shear hinging in the webs of beams. These beams, called "link-beams," provide a yielding, ductile failure mechanism. Remaining framing elements are then designed by properly proportioning their strength relative to the link-beam strength. Figure 2-16 shows a sketch of one EBF and four types of CBF configurations.

4.2 REINFORCED CONCRETE MOMENT-FRAME SYSTEMS

This section discusses the main provisions of the three different current code-based levels of seismic design for reinforced concrete moment-frames. These frame types are defined as follows:

- Seismic Zone 0 and 1: "Ordinary Moment Frame" (OMF), described in Section 4.2.1
- Seismic Zone 2: "Intermediate Moment-Resisting Frame" (IMRF), described in Section 4.2.2
- Seismic Zones 3 and 4: "Special Moment-Resisting Frame" (SMRF), described in Section 4.2.3

As an introduction to these detailed discussions, the following paragraphs present brief discussions of the basic aspects of ductile concrete frame behavior, the limitations of ordinary ultimate strength design of beams and columns in providing for ductility, and the basic terminology and strength relationships of ductile concrete frame design.

Ductile behavior in concrete moment-frames is achieved through controlled yielding of tensile reinforcing in conjunction with proper confinement of concrete zones subjected to high shear and compression stresses. Non-ductile failures occur through failures of over- or under-reinforced sections in bending, bursting of unconfined concrete sections in compression and/or shear, or through failures due to inadequate lap length or development of reinforcing.

The ordinary design provisions for reinforced concrete in shear, flexure and compression provide controls to prevent over- or under-reinforced concrete bending failures and rebar development failures. The ordinary provisions do not, however, provide controls to prevent either shear failure modes of beams and columns or weak-column failure modes. For these modes, member proportioning decisions are made based on the section size and reinforcement required for member loads, and frequently include other factors such as cost, constructibility and esthetics. On this basis, frame designs frequently result which have a uniform factor of safety against normal loading conditions, but which exhibit non-ductile behavior when extraordinary loading conditions occur. To counter this, the special frame design provisions intended for seismic regions (as described in this section) include consideration of proportioning elements of the frame system as a whole.

The basic purpose of special seismic design criteria is to provide increased protection: 1) to highly compressed concrete, by requiring special confining hoops, and 2) to anchored and spliced reinforcing, by increasing safety margins and relocating splice points. In addition, the minimum shear strength of beams and columns within frames is based on the bending

capacity provided, not the design shear forces. Thus, for a beam which is uniformly loaded and fixed at both ends (Figure 4-5), the corresponding minimum shear strength required for both ends to reach their full bending strength is:

$$V_{\max} = \frac{(M_a + M_b)}{L} + \frac{w_{gr} L}{2} \quad (\text{all terms positive})$$

where:

M_a, M_b = Bending strength at ends "a" and "b" (either M_n or M_{pr} described below)

L = Beam span

w_{gr} = Uniform load due to factored gravity loads (see Figure 4-5)

Several different terms are used by the UBC (and this chapter) to represent the bending strength of a reinforced concrete section. These terms are:

M_u = *Design Moment Strength*. The design moment strength, M_u , is the bending strength calculated by using reinforcing strength = f_y , and $\phi = 0.9$. It is used to design the required flexural reinforcing to resist the design moment calculated from the applied forces.

M_n = *Nominal Moment Strength*. The nominal moment strength, M_n is the bending strength calculated by using reinforcing strength = f_y , and $\phi = 1.0$. M_n is used to calculate M_a, M_b for IMRF systems.

M_{pr} = *Probable Moment Strength*. The probable moment strength, M_{pr} is the bending strength calculated by using reinforcing strength = $1.25 f_y$, and $\phi = 1.0$. It represents the approximate *maximum* bending strength, including material overstrength and strain hardening effects. M_{pr} is used to calculate M_a, M_b for SMRF systems.

4.2.1 ORDINARY MOMENT-RESISTING FRAMES (OMF); SEISMIC ZONE 0 AND 1 REQUIREMENTS

For structures in very low-hazard seismic zones, special design or detailing for seismic events is not required in the building code. Requirements are instead based upon the general design recommendations of the ACI (Reference 4-1). Frames designed and constructed in accordance with these provisions are defined as Ordinary Moment Frames (OMFs).

OMF systems are generally lightly reinforced systems which exhibit limited inelastic capabilities. Reinforcing typical of many of these structures is shown in Figure 4-6. The following are details characteristic of OMF construction:

1. Positive (bottom) beam flexural reinforcement with a typical embedment length of 6 inches at the beam-column joint;
2. Little or no transverse reinforcement in the beam-column joint; and,
3. Column lapped splices are confined by widely spaced ties and located in the maximum moment region just above floor level.

Fig. 4-6: Elevation of OMF Column

Anchorage of the bottom reinforcing is regarded as most critical because failure and withdrawal of the bottom reinforcing from the joint generally occurs at relatively low stress levels, before severe damage in the other areas occurs.

Many code-development groups which have studied issues such as progressive collapse, construction failures, and wind-induced damage have pointed to the need to provide greater ductility and redundancy in all structures. In a well-proportioned structure, the sizes, strengths, and damage mechanisms of all elements are balanced relative to each other, so that the structure, if overloaded, will respond in a ductile and predictable manner. Code requirements for ordinary frames provide adequate margins of strength to resist most types of vertical and lateral loads, but do not contain provisions that assure a well-proportioned structure. For this reason, design engineers in all seismic zones should consider including at least some key aspects of ductile-frame design criteria (e.g., proportioning shear and bending strengths), even if seismic forces "do not govern."

4.2.2 INTERMEDIATE MOMENT-RESISTING FRAMES (IMRF); SEISMIC ZONE 2 REQUIREMENTS

Moment frames designed and constructed in moderate-hazard seismic zones are defined as Intermediate Moment-Resisting Frames (IMRFs). The requirements for IMRFs include the requirements for ordinary frames, as well as several additional specific seismic provisions. These seismic provisions include requirements for shear strength, beam stirrups, column ties, and, under some circumstances, two-way slab systems. A significant element of ductile design which is *not* included is the requirement to prevent weak-column failure modes.

Minimum Shear Strength. For beams, columns, or two-ways slabs resisting earthquake effects, the magnitude of the design shear force must be the lesser of:

1. The sum of the shear associated with development of the nominal moment strength at each restrained end and that due to factored gravity loads, as shown in Figure 4-5. Nominal moment strength, M_n is calculated by assuming that reinforcing stress = f_y , and $\phi = 1.0$.
2. The maximum shear corresponding to the application of design gravity and earthquake forces, but with the earthquake effect taken at twice the normal code prescribed value, as follows:

$$\begin{aligned} U &= 0.75 [1.4 D + 1.7 L + (1.1 \times 1.7 \times 2 E)] \\ &= 1.05 D + 1.28 L + 2.8 E \end{aligned}$$

IMRF Requirements for Beams. Beams are defined as elements in which the factored compressive stress, f_a , is less than or equal to $0.1 A_g f'_c$ (if greater, the member is designed as a column).

The positive moment strength at the face of the joint must be not less than 1/3 the negative moment capacity at the same section. The moment strength (either positive or negative) at any section in the span must not be less than 1/5 the maximum moment strength at either end of the beam.

The minimum spacing of transverse reinforcing required for beams, as shown in Figure 4-7, must be reduced within a short distance of each beam end in order to provide confinement within the zone of possible plastic hinging. This requirement is similar to the stirrup spacing requirements described for Seismic Zones 3 and 4, except that closed stirrups are not required by code within the confinement zone (however, the engineer may wish to consider using the two-piece stirrup shown in Figure 4-15).

Beyond the confinement zone, shear reinforcing stirrups are required at a maximum spacing of $d/2$, and must be continued for the full span.

IMRF Requirements for Columns. Unless code-conforming spiral reinforcing is used, closely spaced column ties must be provided within a short distance " L_o " from the tops and bottoms of columns, as shown in Figure 4-8.

Closely spaced ties provide greater confinement within the region of possible plastic hinging, where spalling of the outer "shell" of concrete which surrounds the reinforcing may occur.

The reduced column tie spacing, s , must not exceed 8 diameters of the smallest longitudinal enclosed bar, 24 diameters of the tie bar, one-half the smallest cross-sectional column dimension, or 12 inches. Length L_o must be equal to or greater than one-sixth of the clear span (height) of the column, the maximum cross-sectional dimension of the member, or 18 inches. The first tie must be located at not more than $s/2$ from the joint face.

Where foundations for grade-level columns are located below grade, the columns may experience unanticipated moment-fixity due to the restraint offered by relatively

rigid floor slabs and paving, as shown in Figure 4-9. In this case, extending confinement ties for only a length L_o above the footing may be inadequate to protect the base of the column. It is recommended that confinement ties at the base of grade-level columns be provided for a distance of at least L_o above grade. Thus the adjusted confined length is $L_o' = L_o + d_o$ (shown in Figure 4-9).

Minimum beam-column joint reinforcing requirements are as follows:

- *Joints restrained on all four sides by beams or slabs of approximately equal depth.* Nominal ties must be provided, at a maximum spacing which does not exceed twice the column confinement tie spacing " s ."
- *Other joint configurations.* Nominal ties must be provided, at a maximum spacing which does not exceed twice the column confinement tie spacing " s ." The quantity of joint reinforcement provided must be at least $A_s = 50 bs/f_y$.

Fig. 4-9: Columns with Paving Restraint

IMRF Requirements for Two-Way Slabs. For short span systems, one of the most economical types of construction is the flat slab system. Not only does it simplify concrete forming, but it also reduces the interstory height and the overall dead weight of the structure, both important considerations in earthquake design.

However, if the slab system is constructed without beams or walls, it must act as a part of a frame system to resist lateral loads. Lateral deflections of the resulting frame system can induce large bending moments and shear forces into the slab system in the vicinities of columns.

Two-way slabs are not permitted to be used as frame elements in Seismic Zones 3 or 4 due to the marginal-to-poor seismic performance which these systems have exhibited during past earthquakes. Of particular concern is the potential punching-failure mode caused by unbalanced moments and shears at the slab-column joint.

Slab-Column Joint. At the slab-column joint, bending moments due to Dead- and Live-Loads are generally balanced about the columns, and moments transferred to the column are small.

Seismic and unbalanced span moments produce a net unbalanced moment about the slab-column joint, however, which must be resisted by moments in the columns. All reinforcement provided to resist M_s , the factored unbalanced slab design moment which is resisted by the column moments, must be placed within the column strip (the column strip is defined as the lesser of 25% of the slab span in either direction).

The critical location for the transfer of unbalanced slab moment is assumed to occur at a distance $d/2$ away from the face of the column, where d is the depth to reinforcing in the slab. At the critical section, two mechanisms of moment transfer are assumed to occur, as shown in Figure 4-10; by the eccentricity of the shear forces about the critical section, and by direct flexure about the section.

The fraction of the unbalanced slab moment, $T_f M_s$ which is transferred at the joint by flexure must be resisted by reinforcement placed within a reduced *effective slab width*, defined as the column width plus $1.5h$ at each side, as shown in Figure 4-11 (where h is the slab thickness). The fraction, T_f is calculated as:

$$T_f = 1 / [1 + (2/3) \sqrt{b_1/b_2}]$$

Where b_1 is the depth and b_2 is the width of the critical shear perimeter for moment transfer (see Figure 4-10), which is assumed to extend beyond the column face for a distance $d/2$ on all sides of the column (and d is the depth to reinforcing for the slab). The remainder of the unbalanced moment, given by $T_v M_s$ (where $T_v = 1 - T_f$) is transferred by eccentricity of shear about the centroid of the critical section.

Not less than one-half of the reinforcement in the column strip at the support must be placed within the effective slab width. Both top and bottom reinforcing are required at the

joint, since moments are reversible.

Slab Flexural Reinforcing. As shown in Figure 4-12, a minimum of 1/4 of the column strip top reinforcement must be continuous throughout the span.

Continuous column strip bottom reinforcement must be not less than 1/3 of the total column strip top reinforcement at the support. A minimum of 1/2 of all bottom reinforcement at midspan must be continuous and developed at faces of supports.

All top and bottom reinforcement must be developed at discontinuous edges.

Fig. 4-12: Arrangement of Reinforcement in Slabs

Source: Reference 4-1

4.2.3 SPECIAL MOMENT-RESISTING FRAMES (SMRF); SEISMIC ZONE 3 AND 4 REQUIREMENTS

Moment frames designed and constructed in high-hazard seismic zones are required to be detailed as Special Moment-Resisting Frames (SMRFs). All ordinary frame (OMF) provisions apply, in addition to specific seismic provisions. Since most SMRF provisions are somewhat different than IMRF criteria, all provisions are presented in this section.

Seismic provisions include specific requirements for design load combinations, section and material limitations, and beam, column, and beam-column joint design and reinforcing requirements. These requirements are discussed in the remainder of this section.

Load Combinations. For Seismic Zones 3 and 4, the standard factored load combinations which include earthquake loads are replaced by the following:

$$U = 1.4 (D + L + E)$$

$$U = 0.9 D \pm 1.4 E$$

Material Limitations. A minimum specified concrete strength f'_c of 3,000 psi and a maximum specified reinforcement yield strength, f_y , of 60,000 psi are mandated. A maximum concrete strength of 6,000 psi for lightweight concrete is also mandated. Caution should be used if lightweight concrete greater than 4,000 psi compressive strength is used.

These limits are imposed as reasonable bounds on the variation of material properties, with respect to their effects on the ductility of members in which they are used. A decrease in the concrete strength and an increase in the yield strength of the tensile reinforcement tend to decrease the ultimate curvature (and hence the ductility) of a member subjected to flexure. Also, an increase in the yield strength of reinforcement is generally accompanied by a decrease in the ductility - as measured by the maximum deformation - of the reinforcing steel.

Reinforcing steel which resist flexure and axial forces in frame members (primarily longitudinal bars) is required to be A-706, Grade 60 steel. Grade A-706 reinforcing bars are low-alloy, deformed bars intended for applications where welding and/or bending properties are of importance. However, conventional A-615 billet steel bars of Grade 40 or 60 may be used in these members if the following two conditions are satisfied:

$$\text{Actual } f_y \leq \text{Specified } f_y + 18,000 \text{ psi}$$

$$\text{Actual ultimate tensile stress} \geq 1.25 f_y$$

The first requirement helps to prevent the magnitude of the actual shears that can develop in a flexural member from exceeding the calculated probable moment strength, M_{pr} when hinges form at the ends of a beam. The second requirement is intended to ensure steel with a sufficiently long yield plateau.

Note: A potential hazard can result if Grade 60 reinforcing is substituted for concrete elements that are designed on the basis of Grade 40 reinforcing. The shear strength of beams and columns may become inadequate if the moment capacity of member ends should be greater than intended when then frame elements were proportioned.

Section Limitations (Figure 4-13). Concrete beams used in SMRF systems to resist seismic forces must conform to the following limitations:

- The factored compressive stress, f_a on the section is less than or equal to $0.1 A_g f'_c$ (otherwise, column limitations apply).
- The clear span of the element is greater than or equal to 4 times its effective depth (otherwise, deep beam or coupling beam criteria must be used).

- The beam width must be no less than 30% of the beam depth, or 10 inches (this requirement is intended to provide compact cross-sections that have a reasonably low risk of lateral instability and are able to contain the required reinforcement).
- The beam width must be no greater than the width of the supporting column plus 1.5 times the depth of the beam (this requirement is intended to prohibit the use of a flat-slab strip for a beam element in a moment-resisting frame).

Also, concrete columns used in SMRF systems to resist seismic forces must conform to the following:

- The shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, must be at least 12 inches.
- The shortest cross-sectional dimension must be at least 40% of the perpendicular dimension.
- Columns which contain beam longitudinal reinforcing which extend through the column (to develop continuity for members attached at both side) must have a column depth parallel to the reinforcing of at least 20 times the diameter of the largest longitudinal bar.

SMRF Requirements for Beams (Figure 4-14). In the following section of text, and in Figure 4-14, the term " l_d " refers to the basic reinforcing development length defined in the code. The term " l_{dh} " refers to the basic development length of reinforcing bars with standard hooks. Special Seismic Zone 3 and 4 requirements for l_{dh} are defined in the code and are discussed near the end of this section.

Longitudinal Reinforcing. Top and bottom flexural reinforcing are each required to be within the following reinforcing ratio, ρ limits:

$$\rho_{min} = 200 / f_y \quad \text{with at least two continuous bars at both top and bottom of member.}$$
$$\rho_{max} = 0.025$$

The ordinary maximum reinforcing ratio limitation of $\rho_{balance}$ is insufficient to ensure ductility under strong and repeated reversals of load. Instead, a reduced maximum reinforcing ratio of 0.025 is imposed to ensure that the member has adequate rotational capacity, resulting from yielding of the reinforcement, prior to crushing of the concrete. The limiting ratio also includes considerations of steel congestion and limitations on shear stresses in beams of typical proportions.

From a practical standpoint, lower steel ratios should be used whenever possible, in order to limit congestions of bars and ease constructibility. At beam-column joints, construction difficulties can arise as a result of reinforcement congestion. To prevent this, it is common to produce large-scale drawings of joint areas to ensure that adequate access exists for steel placement and for proper concrete placement and consolidation.

At beam ends, the design moment capacity, M_u , provided by positive (bottom) bars at the joint face must be at least half of the moment capacity provided by negative (top) bars.

Within the span, the moment capacity at any point must be equal to at least 25% of the maximum moment capacity, M_u , provided at the face of either joint.

Reinforcing lap splices are not permitted:

- within the joint,
- within a distance of $2h$ from the joint, where h is the total beam depth, or
- at locations of potential hinging.

In addition, lap splices must be confined by hoops or spiral reinforcement with a maximum spacing or pitch of $d/4$ or 4 inches.

As an alternative to lap splices, welded splices or mechanical connectors may be used, if:

- they are used only on alternate bars in each layer at any section, and
- the center-to-center distance between splices of adjacent bars is at least 24 inches.

Transverse Reinforcing. Special concrete confinement reinforcing in the form of hoops is required:

- over a distance $2h$ from the faces of support (where h is the total depth of the beam), and
- over distances $2h$ on both sides of sections where flexural yielding may occur due to earthquake loading.

Confinement hoop spacing must satisfy the following requirements:

- first hoop located at 2 inches from face of support
- Maximum hoop spacing, s_{max} , of:

$$s_{max} \leq d/4$$

$$\leq 8 \text{ times diameter of smallest longitudinal bar}$$

$$\leq 24 \text{ times diameter of hoop bar}$$

$$\leq 12 \text{ inches}$$

Beyond the zone where confinement hoops are required, standard shear reinforcing stirrups are required. The maximum permitted spacing of stirrups is $d/2$, where d is the effective depth of the beam.

Shear reinforcing is determined based upon the shear required to develop the maximum probable moment, M_{pr} , at the beam ends. The shear strength contribution of concrete, V_c , is neglected if the earthquake-induced shear is greater than $1/2$ of the total design shear and the factored axial compressive force including earthquake effects is less than $0.05 A_g f'_c$.

Special detailing of hoops and stirrups is required for seismic design. These "seismic stirrups" can be either in the form of continuous closed hoops or in the form of a two-piece stirrup plus cross-tie, as shown in Figure 4-15. Due to its greater ease of assembly, two-piece hoops are currently the most common type used for transverse beam reinforcing in high-seismic regions.

SMRF Requirements for Columns (Figure 4-16)

Column Strength Requirements. Engineers are strongly encouraged to configure concrete moment frames to develop strong-column, weak-beam geometries. Sometimes this goal is not possible, for example in long-span, short-column frames such as parking structures. Seismic design criteria therefore permits both geometries, but with severe restrictions on the latter.

Strong-column systems are defined by the following equation:

$$\sum M_c \geq (6/5) \sum M_g$$

where:

$\sum M_c$ = The sum of the design flexural strengths, M_u , of the columns framing into the joint. The contribution of factored axial loads to flexural strengths, consistent with the direction of forces considered, must be included.

$\sum M_g$ = The sum of the design flexural strengths, M_u , of the beams framing into the joint.

Weak-column systems are permitted with the following restrictions:

- Columns supporting reactions from that joint must be provided with transverse hoop reinforcement with spacing to meet confinement criteria throughout the full column height (this can cause construction problems, since the column mid-height area where tie spacing requirements relax somewhat is often the area where concrete pump hoses and vibration probes are introduced).
- The columns must be ignored in calculating the strength and stiffness of the structure (thus, only *some* vertical structure elements are permitted to be weaker than the attached beams). The strength of the columns should not be ignored if it may result in an unconservative condition; for example, by inducing torsion in the structure.

Column Longitudinal Reinforcing. The column reinforcement ratio, $\rho_g = A_s/A_g$, is required to be within the following limits:

$$0.01 \leq \rho_g \leq 0.06$$

Although the OMF maximum reinforcing ratio limit of 8% is reduced to 6%, practical construction limitations will in most cases place the practical upper limit on the reinforcing ratio nearer to 4%. Convenience in detailing and placing reinforcement in beam-column connections makes it desirable to keep the column reinforcement low.

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

Reinforcing lap splices are permitted within the center half of the column height and must be designed as tension splices. Alternatively, mechanical splices can be used at any section if:

- they are used only on alternate bars at a section, and
- the center-to-center distance between splices is staggered by at least 24 inches along the length of the column.

Code requirements for both longitudinal and transverse column reinforcing are summarized in Figure 4-16.

Column Transverse Reinforcing. As in beams, transverse reinforcement in columns must provide confinement of the concrete core as well as shear resistance. In columns, however, the transverse reinforcement must all be in the form of closed hoops or continuous spiral reinforcement - two-piece hoops are not acceptable. Sufficient reinforcement should be provided to satisfy the requirement for confinement or shear, whichever is larger.

Column confinement reinforcing is required over a distance L_o from each joint face or over distances of L_o on both sides of any section where flexural yielding may occur due to inelastic lateral displacements of the frame, where:

$$L_o \geq \text{depth "h" of the member}$$

$$L_o \geq 1/6 \text{ of the clear span (height) of the member}$$

$$L_o \geq 18 \text{ inches.}$$

If spiral or circular hoop reinforcing is used, the volumetric ratio, ρ_s , of the hoops must be at least:

$$\rho_s \geq 0.12 (f'_c / f_{yh})$$

$$\rho_s \geq 0.45 (f'_c / f_{yh}) [(A_g / A_{ch}) - 1]$$

where:

f_{yh} = the specified yield strength of the transverse reinforcement, psi

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

If rectangular hoop reinforcement is used, with total reinforcing area, A_{sh} , and spacing, s_h :

$$A_{sh} \geq 0.09 s_h h_c (f'_c / f_{yh})$$

$$A_{sh} \geq 0.30 s_h h_c (f'_c / f_{yh}) [(A_g / A_{ch}) - 1]$$

where:

h_c = the cross-sectional dimension of the column core, measured center-to-center of the confining reinforcement, inches.

s_h = the spacing of transverse reinforcement measured along the longitudinal axis of the column, inches.

$s_{max} \leq 1/4$ of the smallest cross-sectional dimension of the member, or
 ≤ 4 inches.

Fig. 4-16 : SMRF Column Reinforcing Requirements

Source: Reference 4-2

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

Rectangular hoops, when used in place of spirals, are considered less effective with respect to confinement of the concrete core. Their effectiveness is increased, however, by the use of supplementary crossties, each end of which must engage a peripheral longitudinal bar. Crossties of the same bar size and spacing as the hoops may be used. Consecutive crossties must be alternated end-for-end along the longitudinal reinforcement (Figure 4-17). Crossties or legs of overlapping hoops must be spaced no farther apart than 14 inches in a direction perpendicular to the longitudinal axis of the member.

SMRF Requirements for Beam-Column Joints

In many areas of the country, the beam-column connections normally are not designed by the structural engineer. Detailing of bars within the joints is usually delegated to a draftsman or detailer. In earthquake-resistant frames, however, the design of beam-column connections requires as much attention as the design of the members themselves, since the integrity of the structure may well depend on the proper functioning of such connections. Beam-column joints represent regions of geometric and stiffness discontinuities in a frame and as such tend to be subjected to relatively high stress concentrations. A substantial portion of damage in frame structures subjected to strong earthquakes has been observed to occur at these connections. This has been particularly evident where inadequate attention has apparently been given to their proper design and detailing.

Because of the congestion of reinforcement that may occur as a result of too many bars converging within the limited space of the joint, the proportioning of the frame columns and beams should be undertaken with due regard to the design of the beam-column connection. Little difficulty is usually encountered if the amount of longitudinal reinforcement used in the frame members is kept low and if clear drawings showing bar arrangements are provided.

In order to prevent a brittle shear failure, the joint must have adequate shear strength to resist the forces imposed on it by the connecting members. Figure 4-18 shows an example of a column joint with connecting members on each side. When only gravity loads are imposed on the system, the bending moments in the beams act in opposite directions

concurrently, which therefore transfer relatively little shear and bending in to the columns. Due to seismic, however, the moments in the beams act in the same direction, and must be resisted by the columns. As a result, the joint is subjected to a substantial shear force, shown in the column shear diagram as V' .

Fig. 4-18: Forces in Joints

For design purposes, the joint must be able to resist the most critical combination of horizontal shears transmitted by the framing beams and columns. In order to ensure that the joint is stronger than the attached beams, the joint is required to be able to resist the probable bending strength of the attached beams, M_{pr} . Therefore, the shear force in the joint is calculated by assuming the stress in tensile reinforcement of framing beams equal to $1.25 f_y$.

Test results have indicated that the shear strength of joints is not very sensitive to the amount of transverse (shear) reinforcement present in the joint. Therefore, current thinking is that shear forces are transferred within the joint primarily through a diagonal compression strut, shown in Figure 4-18. For normal weight concrete, the shear strength of the joint is computed as:

$$\phi V_c = 20 A_j \sqrt{f'_c} \quad \text{for joints confined on all four faces}$$

$$\phi V_c = 15 A_j \sqrt{f'_c} \quad \text{for joints confined on three faces or on two opposite faces.}$$

$$\phi V_c = 12 A_j \sqrt{f'_c} \quad \text{for other joints}$$

Where:

$$\phi = 0.85$$

A_j = The effective cross sectional area within a joint, in a plane parallel to the plane of reinforcement generating shear in a joint. The joint depth is the overall depth of the column. Where a beam frames into a support of larger width, the effective width of the joint may not exceed the smaller of the following:

- The beam width plus the joint depth.

- Twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side. See Figure 4-19.
- In no case is A_j greater than the column cross-sectional area.

For lightweight concrete, the nominal shear strength of the joint may be taken as no more than 3/4 of the values given above for regular weight concrete.

When the factored shear in the joint exceeds the shear strength of the joint, the designer may either increase the column size or increase the depth of the beam. The former will increase the shear capacity of the joint section, while the latter will tend to reduce the required amount of flexural reinforcement in the beams, with an accompanying decrease in the shear transmitted to the joint.

Minimum confinement reinforcement, equal to the amount required in the columns above and below the joint, must be provided through the joint around the column reinforcement.

For joints confined by beams on all four faces, a 50% reduction in the amount of confinement reinforcement is allowed, and the minimum spacing may be increased to 6 inches. A member is considered to provide confinement to the joint if at least 3/4 of the joint face is covered by the framing member. The 50% reduction recognizes the beneficial

effect provided by these members in resisting the bursting pressures generated within the joint.

The required amount of confinement reinforcing is not a function of the calculated shear stress in the joint, even though the confinement reinforcing helps to resist the joint shear stresses transmitted by framing members. This is because the reinforcing also is intended to provide confinement to the column concrete to maintain its vertical load-carrying capacity even after spalling of the outer shell of concrete surrounding the hoops.

Except in very large columns, it is usually not possible to develop the yield strength of a reinforcing bar from a framing beam within the width of a column. Where beam reinforcing can extend through a column, its capacity is developed by embedment in the column and within the compression zone of the beam on the far side of the connection. Where no beam is present on the opposite side of a column, such as in exterior columns, the flexural reinforcement in a framing beam must be developed within the confined region of the column. This is usually done by means of a standard 90-degree hook plus whatever extension is necessary to develop the bar, with the development length measured from the near face of the column, as shown in Figure 4-20. The tail of the hook should always be turned into the confined joint, so that the pull of the hook can be balanced by forces from the compression strut zone within the joint, as shown in Figure 4-21.

For flexural reinforcement which is embedded within ductile-frame joints, the UBC provides a separate criteria for determining the required bar development length, as follows:

$$\begin{aligned}
 l_{dh}, \text{ development length} \\
 &\geq f_y d_b / (65 \sqrt{f'_c}) \\
 &\geq 8 d_b \text{ (} d_b \text{ is the bar diameter)} \\
 &\geq 6 \text{ inches}
 \end{aligned}$$

For the same bar sizes with 90-degree hooks embedded in lightweight concrete, l_{dh} must be increased by a factor of 1.25 from the above.

Comparing the above expression for l_{dh} with standard concrete development criteria, the above expression already includes the coefficients 0.70 (for concrete cover)

and 0.80 (for enclosure by ties) that are normally applied to the basic development length, l_{hb} . This is because hooks in seismic frame joints are required to be embedded in the confined core of the column. The above expression for l_{dh} also includes a factor of about 1.4 to increase the required development length to provide for the effect of load reversals.

Frame Elements Not Part of the Seismic-Resisting System

Frame members that are not relied on to provide lateral resistance to earthquake-induced forces need not generally satisfy the stringent ductility requirements governing lateral load-resisting elements. However, some special requirements are imposed, in the form of special design checks and increased factors of safety. These requirements include the following:

1. The gravity-load system must be able to maintain its vertical load-carrying capacity when lateral forces are imposed. Of particular concern are interior support columns and bearing walls. When lateral deflections due to unfactored seismic forces (i.e., corresponding to code level base shear, with no load factors or $3(R_w/8)$ factor) are imposed:
 - Columns which are pinned at top and bottom (typically steel columns) will require slightly greater shear restraint at top and bottom to resist P-delta forces. However, this force is generally quite minor.
 - Columns with moment-resisting connections at top and/or bottom (typically concrete columns) will either: 1) develop elastic fixed-end moments, or 2) develop plastic hinges, if the flexural capacity is less than the elastic fixed-end moment.

The UBC requires that supporting elements must be adequate to resist a lateral displacement corresponding to the *unfactored* code forces and a vertical load corresponding to the full *factored* axial load, including P-Delta effects.

Elements of the gravity-load system need not be designed to resist the full elastic fixed-end moments when the above displacements are imposed. However, if plastic hinges are likely to occur (because the flexural capacity is less than the calculated elastic moment), minimum flexural and confinement reinforcement (as defined by the minimum reinforcing requirements described above for SMRF elements) must be provided in the regions where hinging could occur.

2. Where the shear strength for structural members is less than the shear corresponding to development of the nominal flexural strength, M_n , the shear strength reduction factor, ϕ , must be taken as 0.60.

3. Where the transverse reinforcing for columns does not conform to the transverse reinforcement requirements for SMRF columns, the strength reduction factor, ϕ , for axial compression and flexure must be taken as 0.50.

4.3 STEEL CONSTRUCTION

4.3.1 SEISMIC DESIGN CRITERIA

For load combinations which do not include seismic effects, the UBC recognizes two different design methodologies based on the recommendations of the American Institute of Steel Construction (AISC): 1) an Allowable Stress Design (ASD) criteria (Reference 4-5), which is based upon the "maximum elastic strength" of elements, and 2), a Load and Resistance-Factor Design (LRFD) criteria (Reference 4-6), which is based upon the "limit strength" of elements.

The ASD criteria does not by itself provide specific requirements for seismic design. The seismic design requirements of the UBC (described in this section) are instead based on the recommendations from the Seismology Committee of the Structural Engineers Association of California (SEAOC) (Reference 4-19). Seismic design criteria have recently been developed for the LRFD criteria, and are very similar to the ASD criteria described herein.

The basic approach for seismic design requires creating ductile performance by controlling non-ductile failure modes and creating "weak links" in the structural system. The seismic design criteria is not intended to be compared to normal ASD allowable stress limits, but rather against simplified Plastic Design equations of "strength" which are provided.

"Strength," as defined by the UBC equations, represents the approximate minimum total strength and stability of the element, minus material overstrength and strain hardening effects. Where design criteria require determination of the maximum total strength, the minimum total strength is typically increased by a factor of 1.25 to add in the effects of potential material overstrength and strain hardening.

In both this manual and the UBC, wherever the term "strength" is used, it is in reference to the plastic strength (defined as the force resisted when a section is at the full yield, F_y , stress), and not to either the ASD "elastic strength" or the LRFD "limit strength" terms.

4.3.2 DESIGN PHILOSOPHY

The intent of code criteria based upon strength principles is to base the design of non-ductile elements on the maximum load which adjacent elements can transfer into the

element. By providing a control on which elements yield or fail first, failure can be made to occur in a desirable manner.

In general, the strength and ductility of steel structures depends on:

- the control of local and general structure buckling modes,
- the control of locations of yielding,
- and the strength and toughness of connections used.

Steel structures, particularly moment-frame structures, are in general more prone to lateral deflection and stability problems than other types of construction. The design is therefore often determined based on deflection limits and "P-delta" effects, rather than on minimum strength requirements.

However, even in structures where the design is controlled by deflection criteria, the structure configuration should provide control over the locations where yielding occurs, by forcing overstresses to occur in specific "weak-links" which have known ductile performance.

Because the failure of connections does not normally provide ductile performance, connections should not normally be designed as "weak links." In order to prevent failures, the UBC therefore typically requires that steel connections must be capable of resisting the lesser of; 1) the ultimate capacities of the members connected; or 2) seismic loads corresponding to the total "gravity load," plus $3(R_w/8)$ times the code seismic forces.

When the term "gravity load" is used, both in this manual and in the UBC, it refers to the full design vertical load. As defined in the UBC, gravity load consists of: dead load, plus live load including any reductions for tributary area, plus snow load where applicable.

4.3.3 SELECTION OF MATERIALS

Because some grades of steel with specific ductility and weldability characteristics are better suited to seismic-resistant construction, the UBC sets limitations on the types of steel which may be used for seismic-resisting elements. The following common steel grades are permitted to be used in lateral force-resisting systems of all types and sizes of structures:

- A-36 - all purpose, carbon grade steel
- A-441 - high strength, manganese-vanadium steel
- A-500 & A-501 - hot- and cold-formed carbon steel tubing
(both round and rectangular shapes)
- A-572, Gr. 42 & 50 - high-strength, low-alloy steel
- A-588 - atmospheric corrosion-resistant high-strength,
low-alloy structural steel

- A-283, Gr. D - intermediate tensile-strength carbon steel plate
(permitted for base plates and anchor bolts only)

The following common steel grades do not generally provide adequate ductility for seismic-resisting elements, based on existing test data:

- A-53, Gr. B - seamless & welded carbon steel pipe
A-242 - atmospheric corrosion-resistant high-strength,
low-alloy structural steel
A-514 - quenched and tempered alloy structural steel plate
A-529 - structural carbon steel, 1/2" max. thickness
(commonly used for pre-engineered metal buildings)

The use of these materials for steel framing in lateral-force resisting systems is only permitted for one-story structures.

4.3.4 REQUIREMENTS FOR STEEL COLUMNS (SEISMIC ZONES 3 AND 4 ONLY)

The integrity of a structural system in which overstress and plastic hinges are allowed to occur is strongly dependent on the axial capacity of its columns. Special requirements, applicable to both moment- and braced-frame systems, are therefore made for columns in order to emphasize the importance of columns to the seismic resistance and prevent potential buckling modes.

In addition to designing to resist all normally applicable load conditions in accordance with normal code stress limits, columns in moment or braced frames (with the exception of columns in moment-resisting frames with average compressive axial stress, $f_a \leq 0.3 F_y$) must have the strength to resist the axial force resulting from the following two load combinations:

$$\text{Axial compressive strength} = 1.7 F_y A \leq 1.0 \text{ DL} + 0.8 \text{ LL} \pm 3 (R_w / 8) E$$

$$\text{Axial tensile strength} = F_y A \leq 0.85 \text{ DL} \pm 3 (R_w / 8) E$$

However, the force used need not exceed the maximum force that can be transferred to the columns by adjacent elements of the structure (for example, due to plastic hinges forming in the attached beams), or the limit as determined by the uplift which an attached foundation is capable of resisting. When using this requirement with pile foundations, remember that the actual pile uplift capacity may be much greater than the values employed for the foundation design (do not assume that the pile capacity given by the geotechnical consultant is a limiting value for the purposes of this requirement).

As a part of the column, column splices must also be adequate to resist these forces, and base plates and anchor bolts should have adequate hold-down strength to resist the tension force resulting from the second load combination.

4.4 STEEL MOMENT-FRAME SYSTEMS

4.4.1 INTRODUCTION

Moment-resisting frames remain the dominant type of steel framing in seismic design. Their versatility and ease of erection is favored by engineers and fabricators alike. The details for the joints in such frames are rather standardized. Such joints are also employed in dual systems in conjunction with braced frames.

Moment frames which resist seismic forces are categorized into two types; Ordinary Moment Frames (OMF), which exhibit moderately ductile behavior, and Special Moment Frames (SMF), which exhibit a very high degree of ductile behavior. The type of framing used for each type is very similar, although the individual sections and the types of connections used can be quite different.

Typical steel framing for moment-frames consists of wide-flange columns which extend continuously from the foundation to the top of the structure. Column splices, where required, are located at least 3 feet above girder flanges, which is away from the zone where plastic moments may form and at a location convenient for field welding. Beams or girders consist of wide-flange members which connect between the faces of columns. The beam-to-column connection consists of tab plates which are shop welded to the columns and to which the beam or girder web is bolted. Where a moment-resisting joint is required between the beam and column, the girder flanges are then field-welded to the column with full-penetration welds.

Moment-frames resist the effects of vertical loads and seismic forces by resisting rotations between the girder and column at the girder-to-column joint. As a result, shear and bending forces are developed in the frame, which during seismic motions are generally most critical in the vicinity of the girder-to-column joint.

Figure 4-22 shows an elevation of a small portion of moment frame, extending about the column joint between the mid-heights of the column and between the mid-spans of the girder (the approximate location of the inflection points of the seismic moment diagrams). Also shown are the girder gravity and seismic load moment diagrams and column free-body, shear, and moment diagrams. The girder bending moments due to gravity loads act in opposite directions concurrently, which therefore transfers relatively little net shear and bending into the columns. Due to seismic forces, however, the moments in the girders act

in the same direction, and must be resisted by the columns. As a result, the joint "panel zone" is subjected to a substantial shear force, shown in the column shear diagram as V_a .

The shear force resisted by the panel zone can be shown to be approximately:

$$V_a = (M_a / 0.95 d_1) + (M_b / 0.95 d_2) - V_b$$

where:

- M_a = girder moment on left side of column
- M_b = girder moment on right side of column
- d_1 = girder depth on left side of column
- d_2 = girder depth on right side of column
- V_b = the shear in the column outside the panel zone

This equation can be easily derived by considering that the terms $M / 0.95 d$ represent the axial force in the girder flanges due to the bending moment. The shear in the column outside the panel zone, V_b can be obtained by taking moments about the points of inflection of the column, resulting in:

$$V_b = (M_a + M_b) / H$$

Where: H = the average of the column heights above and below the connection.

Unless designed with very large degrees of overstrength, some amount of yielding, leading to the development of plastic moments, is intended to occur to columns and girders during a seismic event. It is therefore important that the column and girder sections be able to develop their full plastic moments without developing buckling or instability problems.

AISC design specifications divide steel sections into the three categories of compact, non-compact, and slender, depending upon the governing width/thickness ratios of section elements which are under compression. Figure 4-23 illustrates how width/thickness ratios affect the relative bending strengths of each of these categories (numbered as 2 through 4).

Figure 4-24 then shows the relative rotational ductility which can be achieved by each category, as well as that which can be achieved by a more restrictive width/thickness ratio (shown as number 1).

Compact sections are capable of developing a fully plastic stress distribution with a rotation capacity, R , of approximately 3 times the rotation at yield before the onset of local buckling. Non-compact sections can develop the yield stress in compression elements before local buckling occurs, but will not resist inelastic local buckling at the strain levels required for a fully plastic stress distribution. Slender compression elements buckle before the yield stress is achieved.

For OMF systems in all seismic zones, and for SMF systems in seismic zones 1 and 2, the rotation capacity provided by the standard compact section criteria

Fig. 4-23: Generalized Bending Response for Local Flange, Local Web, and Lateral Buckling

Fig. 4-24: Moment-Rotation Behavior of Beams

is adequate to provide acceptable seismic performance. For SMF systems in seismic zones 3 and 4, however, a greater inelastic rotation capacity, on the order of $R = 7$ to 9, is required to provide sufficient structure ductility to resist the number and magnitude of inelastic cycles which could occur. Therefore, the UBC requires that all girders in SMF systems must meet a slenderness ratio requirement of:

$$b_f/2t_f \leq 52/\sqrt{F_y}. \text{ (Figure 4-24, Number 1)}$$

4.4.2 ORDINARY MOMENT-RESISTING STEEL FRAMES (OMF)

Ordinary Moment Frames of structural steel are moment-resisting frames which do not meet the detailing requirements for ductile behavior defined for Special Moment Frames (SMFs). Due to the reduced provisions for ductile behavior, OMFs are required to be designed using an R_w factor which corresponds to twice the seismic force required for SMF systems.

In seismic zones 2, 3, and 4, OMF systems must conform to one of the following two alternatives:

- Design all frame girder-to-column connections in accordance with the criteria required for SMF systems.
- Design all frames for a combination of "gravity loads" plus $3(R_w/8)$ times seismic forces.

In seismic zone 1, OMF systems must conform to one of the following two alternatives:

- If design seismic forces are determined using the standard R_w factor of 6, girder-to-column connection detailing requirements do not apply.
- If design seismic forces are determined using an R_w factor of 12, girder-to-column connections must be designed in accordance with seismic zone 2, 3, and 4 OMF requirements.

4.4.3 SPECIAL MOMENT-RESISTING STEEL FRAMES (SMF); SEISMIC ZONE 2 CRITERIA

In seismic zone 1, SMF construction is not required. In seismic zone 2, the following UBC requirements must be adhered to for SMF systems.

Determining Failure Modes and Selecting Member Sizes. The basic philosophy of SMFs is simply that non-ductile failure modes should be prevented, and that inelastic deformations should be concentrated in regions having a capacity for ductile behavior. Thus, column

buckling and connection failures are to be avoided. At the same time, frame stability and therefore frame stiffness must be maintained.

Ductile behavior and inelastic energy absorption is allowed to occur in any of three elements of the moment frame: 1), by forming flexural hinges in the beams; 2), by allowing shear yielding to occur in the joint panel zone; or 3), by forming flexural hinges in the columns. Optimum frame behavior occurs when controlled yielding in *both* the girder and the panel zone regions occurs. The strength of each of these elements is defined as follows:

1. $M_c = \sum Z_c (F_{yc} - f_a)$, = the flexural strength of columns, where $f_a > 0$.
2. $M_s = \sum Z_b F_{yb}$, = the flexural strength of the girders
3. $M_{pz} = 0.95 d_b V_{pz}$, = the flexural strength of the panel zone for a joint in which all connecting girders have equal depth, d_b (including joints in which a girder connects from only one side).

where:

V_{pz} = the shear strength of the panel zone

$$V_{pz} = 0.55 F_y d_c t \left[1 + (3 b_c t_{cf}^2 / d_b d_c t) \right]$$

d_c = the depth of the column web

d_b = the depth of the girder web

t = the girder flange thickness

t_{cf} = the column flange thickness

Z_c, Z_b = the Plastic Section Modulus of the column and girder, respectively

F_{yc}, F_{yb} = the material yield strength of the column and girder, respectively

If the flexural strength of the panel zone, $1.25 M_{pz} \geq M_s$, then the panel zone is adequate to resist the flexural strength of the attached girders. Likewise, if $M_c \geq M_s$, the flexural strength of the columns is adequate to resist the flexural strength of the attached girders. The 1.25 factor is an empirical factor which is intended to account for observed overstrengths in the flexural capacity of joint zones, based on test data.

For joint conditions in which girders of unequal depths connect, the maximum panel zone shear which occurs when the flexural strength of the girders is reached can be calculated by substituting M_a, M_b terms in the relationship between panel shear, V_a and external moments, M_a, M_b defined in Section 4.4.1 with appropriate $Z_b F_{yb}$ terms. If the resulting shear is less than V_{pz} then the flexural strength of the joint zone is adequate to resist the girder flexural strength.

Whichever of these element has the least strength, becomes the "weak link" in the frame. Depending on which of the above three elements is chosen to have the lowest elastic

threshold, different constraints must then be observed by the Engineer during the design, in addition to certain general requirements.

Weak-Girder Configurations. Because allowing yield in columns may be undesirable, and frame stiffness and drift concerns may prevent allowing joint yielding, yielding of girders, often in conjunction with yielding of the panel zone, is the preferred design alternative.

Note: A common design practice for moderate-to-tall structures in high-seismic regions is to specify the use of A-572 Grade 50 material for columns and A-36 material for girders and miscellaneous framing. The slight increase in cost per pound for A-572 material can be more than offset if the resulting column 1) creates a weak-girder, strong-panel/column configuration, and 2) the need to weld web doubler plates within the joint panel zone is avoided.

If yielding in the girder is set as the inelastic mode, the design of the girder-to-column connection and the provision of adequate lateral bracing for beam stability are usually the greatest design concerns. Beam and column lateral bracing requirements are described in a later part of this section. Girder-to-column connection criteria for weak-girder configurations are as follows.

Figures 4-25 and 4-26 show various common moment-resisting girder-to-column connection details, all of which are acceptable for use in SMF systems if properly detailed. Of the three types of details shown,

1. The connections at the top use high-strength, slip-critical (A-325-SC) bolts to connect the web and full-penetration field welding to connect flanges. It is the most common type of connection in western states and is the basic "alternate connection" defined by the UBC. It has relatively good ductile performance, however, the bolted-only web connection sometimes cannot transfer the full plastic moment capacity for some wide-flange shapes (the UBC has supplemental requirements for these shapes which are described below).
2. The connections in the center use field-welding to connect both the girder web and flanges (the bolts are for erection purposes only). This connection offers the most ductile performance, but is the most expensive to build.

3. The connections at the bottom use high-strength, slip-critical bolts to connect both the web and flanges. Unless carefully detailed, this connection offers the least ductile performance. The number and size of bolt holes in girder flanges is restricted by net-to-gross flange area

restrictions in the UBC. All flange and web bolting shown must be high-strength (Slip-Critical) bolting.

The girder-to-column connection used must be adequate to develop the smaller of the following two shear forces:

1. The shear force resulting from the "gravity load" plus the shear forces resulting from the girder flexural strength, $M_s = Z_b F_{yb}$, occurring at both ends of the girder.
2. The shear forces resulting from the "gravity load" plus $3(R_w/8)$ times the girder shear resulting from the prescribed seismic forces.

While any type of connection may be used, it must first be shown by either test or calculation to have the required total strength. As an alternative, the UBC prescribes a specific connection detail which may be used.

Figure 4-27 summarizes basic detailing requirements for the alternate girder-to-column connection, when connected to column flanges. Girder flanges are connected with full penetration butt welds to the columns. For the bolted-web connection shown, the method of web connection required is dependent on the yield characteristics which result from the properties of the girder section used, as follows:

1. Where the flexural strength of the girder flanges contribute $\geq 70\%$ of the flexural strength of the entire girder section, the web connection is assumed to provide shear strength only.
2. Otherwise, the connection must be able to provide shear strength, plus a flexural strength equal to at least 20% of the flexural strength of the girder web. In order to provide the required flexural strength, the web is required to be welded either directly or through shear tabs to the column. This can be done either by replacing the bolted connection with a fully-welded web connection, or by adding small supplemental welds to the ends of the web tabs, as shown in Figure 4-27.

Figure 4-28 presents similar detailing requirements for girder-to-column web connections. In the plan view of the figure, note that, because the width of girder does not match the column depth, a severe re-entrant corner occurs in the girder flange. Recent research has indicated that this area is prone to fracturing when cycled inelastically. A recommended detail modification is shown in Figure 4-29, where a tapered continuity plate is used to transition stresses into the column flanges.

Fig. 4-27: SMF Girder-to-Column Flange Detail

Source: Reference 4-19

Fig. 4-28: SMF Girder-to-Column Web Detail

Source: Reference 4-19

Weak-Panel Zone Configurations. If yielding of the panel zones is set as the "weak link," adjacent girder-to-column connections must be adequate to develop the moment corresponding to development of 125% of the panel zone shear strength and beam and/or column lateral bracing may be required.

The UBC does not include the same frame drift (deflection) calculation requirements for Seismic Zone 2 that are described for Seismic Zones 3 and 4. However, the designer should be aware that shear distortions of the panel joint will tend to increase lateral deflections of the frame.

Weak-Column Configurations. Although yielding in columns is, in general, an undesirable mode of failure, the UBC does not restrict the use of weak-column configurations in seismic zone 2, with one exception (frame systems using trusses). In weak-column configurations, the column lateral bracing requirements described below must be observed. Girder-to-column connection design requirements are the same as that previously described for weak-panel joint configurations.

A specific type of strong-girder, weak-column system occurs when trusses are used as girder elements in SMF systems. An example of this type of framing may be employed in one story industrial buildings having long spans between columns. Since trusses are made up of axially loaded members without provisions for flexural ductility, it is the columns which need to yield in order to protect the truss. Therefore, these columns need the following special provisions to insure their ductility, joint stability, and resistance to tension:

1. The trusses must be able to develop adequate flexural strength to be able to resist 125% of the total flexural strength of columns framing into the truss from above and below.
 - The flexural strength of the truss which is available to resist seismic forces is the axial compression or tension "strengths" of the truss members, minus dead load forces.
 - The connections of the truss members to the column must be able to develop the maximum possible force resulting from the "strengths" of the members.
2. In buildings of more than one story, the column axial stress must not exceed $0.40 F_y$ and the ratio of the unbraced column height to the least radius of gyration must not exceed 60.

3. In addition, the columns of truss frames must be laterally braced at each truss chord with adequate capacity to resist a lateral force equal to 1% of the compression yield strength of the chord.

Lateral Bracing of Columns. Adequate out-of-plane bracing is required to ensure that the columns are capable of reaching their seismic design capacity. Columns which are bent into reverse curvature by seismic forces (the usual case), are generally able to develop their full capacity if laterally braced at both ends in the regions of greatest moment. Beam-to-column connections are usually restrained laterally by roof or floor framing, which attach to the column in the direction out-of-plane to the moment frame.

The UBC defines lateral bracing criteria for three general restraint conditions:

1. If the column can be shown to remain elastic when subject to seismic forces, then the column is considered to be adequately supported out-of-plane if the flanges of the column are laterally supported only at the level of the girder top flange. The column is considered to remain elastic if one of the following conditions is met:
 - The flexural strength of the column is at least 1.25 times either the flexural strength of the girders or the shear strength of the panel zone.
 - The stresses resulting from the lesser of the girder flexural strength or panel zone strength will limit column stress ($f_a + f_{bx} + f_{by}$) to $\leq F_y$ of the column.
 - The column will remain elastic under gravity loads plus $3(R_w/8)$ times the prescribed seismic forces.
2. Where the column flanges cannot be assumed elastic (don't meet any of the above criteria), then the column flanges must be laterally braced at both the girder top and bottom flanges. The bracing provided must be able to resist a minimum force of 1% of the girder flange capacity, while limiting displacement to less than 0.2 inch.

Bracing at the column flanges can usually be economically achieved by using a beam framing into the weak axis of the column, if a positive connection is provided to the column flanges through stiffener or continuity plates, as shown in Figure 4-30. Flange bracing forces are resisted via stiffener plates and a full depth gusset by a moment couple in the bolts connecting the beam to the gusset.

3. Special cases can occur in which roof or floor framing members are not attached to the column-girder joint out-of-plane, producing an unrestrained or laterally unsupported joint, as shown in Figure 4-31.

It is extremely important that the columns do not buckle out of the plane of the seismic frame. To assure this, four design requirements are imposed on columns without lateral support transverse to the joint; 1) increased seismic forces, 2) reduced allowable axial stress, 3) a reduced column slenderness ratio, and 4) the inclusion of the effects of bracing and P-Delta forces when figuring bending stresses about the unbraced axis of the column.

Lateral Bracing of Beams/Girders. UBC Seismic Zone 2 criteria does not include lateral bracing requirements for girders, however, the addition of lateral bracing as described below for Seismic Zone 3 and 4 criteria is strongly recommended, particularly for weak-girder configurations. Without adequate lateral bracing, the full plastic flexural strength cannot be fully developed in many instances.

Girder Flange Limitations. For steel with specified ultimate strength $F_u < 1.5 F_y$ (which includes most of the high-strength grades of steel), bolt holes or other reductions to the beam flange area are not permitted at locations where plastic hinges may form. For these steel grades, plastic hinges may be unable to form in areas where the flange area has been reduced.

Fig. 4-30: Bracing of Column Flanges

Source: Reference 4-9

Fig. 4-31: Laterally Unrestrained Joint

For the remaining grades of steel, a limited number of bolt holes are permitted. Bolted connections of flange plates of beam-column joints must have a net-to-gross area ratio of:

$$A_{\text{eff}}/A_{\text{gross}} \geq 1.2 F_y/F_u$$

Where:

$$\begin{aligned} A_{\text{eff}} &= U A_{\text{net}} \\ &\leq 0.85 A_{\text{gross}} \end{aligned}$$

A_{gross} = the gross area of the section

A_{net} = the net area of metal at a section, minus metal area lost due to bolt holes. Bolt hole widths are taken as the actual diameter of the hole, plus a distance of 1/16 inch for each hole. The value of A_{net} used should be the minimum value obtained from sections taken through any set of bolt holes. Procedures for this calculation are given in References 4-5 and 4-6.

U = a reduction coefficient, which is defined as follows for bolted connections:

1. Any member connected with only two fasteners per line in the direction of stress (the minimum permitted), $U = 0.75$.
2. Members connected with three or more fasteners per line in the direction of stress:
 - a. W, M or S shapes with flange widths not less than 2/3 the depth, and Tee shapes cut from these shapes, if connected through the flanges, $U = 0.90$.
 - b. Other shapes, including built-up shapes and members which are connected through the webs, $U = 0.85$.

In addition, abrupt changes in beam flange area are not permitted within the possible plastic hinge regions. Thus, the bending strength of beams may not be reduced over a short distance from the column face (by trimming the beam flanges or other means) in an effort to reduce the cost of connections.

Stiffener, or Continuity Plates. Stiffener, or girder continuity plates, may be required at joints, depending upon the conditions listed below. Because testing has shown that continuity plates are critical to developing the flexural strength of attached girders, continuity plates *should* always be provided wherever there is a chance that they may be required.

The following three conditions determine whether stiffener plates are required at either the tension or compression flanges of the girder:

1. Stiffeners are required at both tension and compression flanges whenever:

$$0 \leq P_{bf} - F_{yc} t (t_b + 5k)$$

2. Stiffeners are required opposite the compression flange if:

$$d_c > 4100 t^3 \sqrt{F_{yc}}$$

3. Stiffeners are required opposite the tension flange if:

$$t_f < 0.4 \sqrt{(P_{bf}/F_{yc})}$$

where:

P_{bf} = $A_f F_{yb}$, for SMF systems in Seismic Zone 2, 3 or 4.

Note: this is a special UBC requirement which replaces the standard AISC definition for P_{bf} .

F_{yc} = Column Yield Stress

t = Thickness of column web

t_b = Thickness of girder flange

t_f = Thickness of column flange

k = Distance from outer face of column flange to toe of web fillet

Since seismic moments are reversible, stiffening plates should always be provided at both flanges in the joint if they are required at all. Normally, the size of stiffening plates provided do not extend beyond the edge of the flanges which are stiffened, however, wider stiffeners may be required if the stiffeners also provide column lateral bracing functions, as described above.

Web Doubler Plates. Ordinary frame design requires columns to be reinforced in the panel zone with a doubler plate if the column web is not thick enough to resist the shear through the web in the plane below the top beam flange. For shapes with $h/t_w \leq (380/\sqrt{F_{yc}})$ subject to non-seismic loading conditions, web reinforcement is required in the shear in the panel zone exceeds:

$$V = 0.40 F_{yc} d_c t_w$$

where:

d_c = the column depth

t_w = the panel web thickness

h = the clear distance between flanges

Since this criteria is more restrictive than the seismic zones 3 and 4 maximum stress criteria for panel zones, it is not appropriate to require it for load combinations which include seismic forces in seismic zone 2. A reasonable approach is to provide web doubler plates if required for dead, live, and wind forces, but to ignore this criteria (and thereby implicitly allow yielding) for zone 2 seismic forces.

Doubler plates are expensive and, in most cases, it is more economical to choose a column section with a web thickness sufficient to avoid them. In addition, the addition of thick doubler plates can sometimes cause more harm than good, due to cracking and stresses which can occur from shrinkage caused by large areas of welding.

4.4.4 SPECIAL MOMENT-RESISTING STEEL FRAMES (SMF); SEISMIC ZONES 3 AND 4 CRITERIA

For seismic zones 3 and 4, all applicable zone 2 requirements apply, with certain additional requirements and exceptions, as described in this section.

Weak-Girder Configurations. If yielding in the girder is the first inelastic mode, then the design of the girder-to-column connection and the provision of adequate lateral bracing for beam stability are usually the greatest design concerns.

- Girder-to-column connection requirements are the same as those described above for seismic zone 2.
- Seismic Zone 3 and 4 requirements for lateral bracing of girders are described below.

Weak-Panel Zone Configurations. If yielding of the panel zones is the first inelastic mode, additional requirements for Seismic Zones 3 and 4 include limitations on both minimum panel zone shear strength and thickness, as well as specific calculation requirements (described below) which must be included in the structure drift calculations.

The minimum joint panel zone strength must be capable of resisting the shear induced by beam bending moments due to "gravity loads" plus 1.85 x the code seismic forces, or:

$$M_{pv} \leq M_D + M_L + 1.85 M_E$$

To control shear buckling, the minimum joint panel thickness t_z , must conform to following:

$$t_z \geq (d_z + w_z)/90$$

where:

- d_z = the panel zone depth between continuity plates
- w_z = the panel zone width between column flanges

The thickness t_z , should not include double plate thickness unless the plate is connected to the column web with plug welds adequate to prevent local buckling of the plate, as shown in Figure 4-27.

Doubler plates provided to increase panel zone shear strength must be placed not more than 1/16 inch from the column web and welded to the column flanges with either butt or fillet welds to develop the shear strength of the doubler plate, as shown in Figure 4-32. The top and bottom of the plate must be welded across the plate width at with at least a 3/16 inch fillet weld. A common practice is to continue the doubler plate beyond the girder flanges for a distance of at least 2.5 times k_{col} , as shown in Figure 4-27. Otherwise, the

Fig. 4-32: Web Doubler Plate (Plan View)

Source: Reference 4-9

top and bottom welds of the doubler plate will intersect with the welds for the flange stiffener plates, and welding details such as shown in Figure 4-33 must be used. This detail is not recommended for applications with web doubler thickness greater than about 1/2 inch, due to the quantity of welding involved and its resulting restraint against thermal contraction.

Fig. 4-33: Web Doubler Plate (Elevation and Sections)

Source: Reference 4-16

Weak-Column Configurations. Because yielding in moment-frame columns is, in general, an undesirable mode of failure, seismic zones 3 and 4 criteria permits the column to be the weaker element only if at least one of the following conditions is met:

1. For columns with axial stress, $f_a < 0.4 F_y$ for all normal design load combinations, and meeting at least one of following additional requirements:
 - a. Top story column of a structure with period, $T > 0.7$ seconds.

- b. The column is part of a dual system, where the SMF is not resisting a significant percentage of the total story shear (less than 20% of the net story shear *and* less than 33% of the shear resisted by that column line).
 - c. If the column design is done without the normally permitted 1/3 stress increase.
2. For columns in any story having a lateral shear strength at least 50% greater than that of story above.
 3. Columns which are not part of the lateral-force resisting system.
 4. Where trusses are used as girder elements in SMF systems, a strong-girder, weak-column system occurs. In addition to the limitations and criteria previously described for Seismic Zone 2, normal ASD allowable stresses (axial, shear, bending, etc.) of the columns are reduced 25% if one end frames into a truss, and 50% if both ends frame into trusses.

Fig. 4-34: Lateral Girder Bracing w/ Gusset

Source: Reference 4-9

Fig. 4-35: Lateral Girder Bracing w/ Braces

Source: Reference 4-9

Lateral Bracing of Beams/Girders. Both the top and bottom flanges of beams which are part of the SMF must be braced, either directly or indirectly. Spacing of beam lateral bracing elements between column centerlines must not exceed a distance equal to $96r_y$, where r_y is the radius of gyration of the girder about its weak axis. In addition, braces are required at any points of concentrated load where a plastic hinge may form.

Bracing is typically provided either at the girder 1/3 span points or at midspan. An intersecting beam with a full depth gusset plate, as shown in Figure 4-34, may be adequate to brace both girder flanges if the beam has adequate depth to resist the bottom flange brace force (1% of the flange axial capacity) while limiting the flange deflection to less than 0.2 inch. Alternatively, braces can be added, extending from the girder bottom flange to the top flange of an adjacent beam, as shown in Figure 4-35.

Stiffener, or Continuity Plates. Basic recommendations for stiffener, or continuity, plates are similar to the Seismic Zone 2 criteria described in Section 4.4.3. However, Seismic Zone 3 and 4 criteria requires that the calculated flange force P_{bf} be replaced by the following terms:

$P_{bf} = A_f F_{yb}$ when calculating the required stiffener area, A_{st} , or when checking compression flange stiffener requirements

$P_{bf} = 1.8 A_f F_{yb}$ when checking tension flange stiffener requirements

Girder Flange Limitations. The restrictions on bolt holes or reductions in area applicable in Seismic Zone 2 also must be observed in Seismic Zones 3 and 4. However, as an even stronger statement, the UBC states that abrupt changes of the beam flange area are not permitted within possible plastic hinge regions of SMFs.

Deflection/Drift Calculations. Historically, most stress analysis and lateral deflection calculations have been based on "stick models" which calculate forces based on member centerline locations. These procedures assume that beam and column spans are large with respect to the depths of the members, so that the effects of shearing deformations and joint size can be neglected. Axial shortening of columns is also ignored in many cases (although with modern methods of computer analysis its effects are easy to include).

For SMF systems in Seismic Zones 3 and 4, the drift analysis should be based upon calculations which include the effects of the following:

1. Bending and shear deformations which are calculated based on actual clear spans between girder and column flanges (rather than on member centerline distances), unless it can be demonstrated that the drift calculated using member centerline distances is typically within 15% of the deflections calculated using clear span lengths.
2. Column axial deformations, unless it is shown that they contribute less than 10% to the total drift.
3. Deflections due to rotation and deformation of the joint panel zone, as shown in Figure 4-36. Panel zone deformations may be neglected if it is shown that the flexural strength of the joint panel zone is greater than 0.8 times the girder flexural strength,

$$M_{pz} > 0.8 \sum M_s, \text{ (and thus the joint panel zone remains elastic).}$$

For relatively small (less than 5 stories) structures, a simplified analysis model is an acceptable and reasonably accurate analysis approach. For larger structures, since several of the conditions noted above require performing the more complex analysis in order to determine the degree of error caused by the simplified analysis, it is generally preferred to develop a stiffness model which includes the required effects.

4.5 STEEL CONCENTRIC-BRACED SYSTEMS

This section discusses concerns for Concentric Braced Frame (CBF) systems which relate to overall system configuration, required member and connection design strengths, and connection configuration and detailing practices.

The UBC defines a CBF as any type of braced frame except an EBF (EBFs are very specifically defined systems and are described in Section 4.6). CBF systems include bracing configurations in which the neutral axis of the various intersecting members (beams, column, and diagonal members) might not meet at a common point at each joint (and thus are non-concentric).

While it is commonly assumed for the purposes of analysis that the neutral axis of the various intersecting members meet at a common point at each joint, in actuality, some eccentricity often occurs due to the physical limitations of the connections. This eccentricity is usually ignored in the design of the members if reasonably small. However, any eccentricities must be considered in the design of bolts or weldments within the connection.

4.5.1 BRACING CONFIGURATION REQUIREMENTS

In Section 2.2.2, the various basic types of concentric bracing systems were defined as: 1) tension-compression, 2) tension only, 3) chevron, and 4) K-bracing (Figure 2-16). The following paragraphs summarize code requirements and design recommendations particular to each configuration.

Tension-Compression Bracing Systems. Tension-compression bracing can consist of either single diagonal elements or of pairs of braces which are arranged in an "X" pattern. Since X-bracing always has at least one element in tension, it provides somewhat better strength and seismic resistance than the single element configuration.

An assumption sometimes made by engineers when designing X-bracing is that the brace in tension is able to provide out-of-plane support to the brace in compression. This assumption causes a reduction of the effective unbraced length to about half of the total

brace length. For static loads, the tension brace provides a strengthening effect if it has both adequate stiffness and strength to restrain lateral displacements of the compression member. For strong cyclic forces such as earthquakes produce, this assumption is suspect, since elongation of the tension elements when repeatedly overstressed could prevent them from providing lateral support to the compression elements. Since research using typical earthquake displacement histories does not at present exist to support this assumption, its use is not recommended in members which resist seismic forces.

Instead, the compression capacity of the diagonal elements should be calculated by using the full length of the diagonal member, either from center-to-center of work-points, or from face-to-face of the horizontal and vertical members. Where forces require heavier bracing, an economical practice is to use wide-flange members as diagonals in an X-bracing configuration. By turning the wide-flange members sideways, the weak axis of the members can be braced in-plane by the intersection point of the bracing (which is a valid way to reduce the unbraced length).

Experimental studies performed at the University of Michigan indicate that the seismic performance of concentric bracing can be considerably improved if highly compact section properties are utilized. As a result, UBC code-writing groups are considering possible future provisions to limit width-thickness ratios of unstiffened elements of compression bracing members to $30/\sqrt{F_y}$, in conjunction with possibly less restrictive requirements of R_w and/or β -factors.

Chevron Bracing Systems. Due to the relative ease with which they can be fitted around doorways and windows, chevron and inverted-chevron bracing systems are often used in steel buildings.

Chevron bracing elements typically intersect near the midspan of the connecting beam element. Because of this, the bracing not only resists lateral loads, but also acts as a vertical support for the beam to which it is framed. Tributary gravity loads must therefore also be considered in the design of the bracing members, in addition to lateral forces. However, the beam itself must be capable of supporting all tributary gravity loads as if the supporting bracing were not present. This is because compression buckling of one side of the supporting braces when overstressed can 1) cause a loss of the support to the beam, and 2) cause the remaining tension brace to pull against the beam, as shown in Figure 4-3.

UBC requirements for chevron bracing systems in seismic zones 2, 3, or 4 are as follows:

1. The beam intersected by the braces must be continuous between the columns.
2. The beam itself must be capable of supporting all tributary gravity loads as if the bracing did not exist.

3. The bracing members (columns, beams, and diagonals) and their connections must be designed for 1.5 times the otherwise prescribed seismic forces in addition to any " β " or $3R_w/8$ factors (as defined in Section 4.5.2). For Seismic Zones 3 and 4, however, column designs may be governed by the load combinations defined in Section 4.3.4.

Tension-Only Bracing Systems. Tension-only bracing systems use very slender bracing elements in X-bracing configurations to resist lateral loads. They are identical to similar tension-compression bracing, except that the braces are assumed to buckle elastically at very low loads, so that only the tension elements resist lateral loads.

Tension-bracing systems have fair seismic performance when used in very light structures, but are prone to severe deformation and loss of strength if forced to resist severe cyclic loads. For seismic zones 2, 3, and 4, the use of tension-only bracing systems is only permitted for one- and two-story structures. The UBC also requires that the bracing members (and their connections) be designed for $3R_w/8$ times the otherwise prescribed seismic forces.

K-Bracing Systems. Just as overstresses in the compression diagonal of chevron-bracing systems can lead to lateral forces against the intersecting beam, similar overstresses in K-bracing systems can lead to lateral forces against, and possible failure of, the intersecting column. For seismic zone 2, the UBC specifies that the bracing elements must be designed to resist a force which is 1.5 times greater than the applied design forces. For seismic zones 3 and 4, K-bracing is only permitted for one- and two-story structures, and bracing members (and their connections) must be designed for $3R_w/8$ times the otherwise prescribed seismic forces.

4.5.2 MEMBER STRENGTH REQUIREMENTS

In a concentric bracing system, the total strength is controlled by the axial-force capacity of the connecting elements or connections thereof.

The UBC requires bracing which resists seismic forces to be designed on the basis of a "stress-reduction factor," β , which is intended to account for the rapid reduction in member strength which occurs when members are cyclically loaded into the inelastic range.

The definition of "bracing members" includes all of the components of the vertical truss system which resists lateral loads: diagonal elements, vertical (column) elements, and horizontal strut elements (but not to any horizontal collectors attached to the bracing). Since all of these elements experience substantial cyclic forces as a part of the bracing system, it is appropriate that each of these components be designed using a "stress-reduction factor," β , which is calculated separately for each member. It is inappropriate to

design a bracing system using a β -factor for the diagonal element only, since this could result in a configuration in which the "weak-link" could occur in the column or strut, rather than the diagonal member.

As an exception, columns of bracing systems in seismic zone 3 or 4, which are designed using the special load combinations described in Section 4.3.4, should not be designed using β -factors, since this would result in the layering of two separate factors of safety.

The allowable compression stress for bracing members which resist seismic forces is determined from the following formula:

$$F_{as} = \beta F_a$$

Where:

$$\beta = \frac{1}{1 + \frac{KL/r}{2C_c}}$$

KL/r = Slenderness ratio of the bracing member

$$\begin{aligned} C_c &= \sqrt{(2\pi^2 E / F_y)} \\ &= 126.1 \text{ for } F_y = 36 \text{ ksi.} \end{aligned}$$

For seismic zone 2, β need not be less than 0.8. For seismic zones 3 and 4, the minimum value is restricted by minimum L/r requirements.

For seismic zones 2, 3, and 4, other UBC requirements relating to the required member strength are as follows:

1. Where built-up members such as double-angles are used as bracing members, the L/r of the individual parts of the bracing members between stitch-plates, when computed about a line perpendicular to the axis through the parts, cannot be greater than 75 percent of the L/r of the member as a whole.
2. Braced frames not meeting the other design requirements described in this section are permitted in buildings not over two stories in height, but the systems must be designed for $3(R_w/8)$ times the normal code seismic forces.

For seismic zones 3 and 4 only, the UBC also requires the following:

3. The requirements for materials and special requirements and load combinations for columns discussed in Section 4.2, above, apply for bracing systems and the columns which resist bracing forces.

4. The L/r for bracing members may not exceed $720/\sqrt{F_y}$, except as permitted by item 2, above (thus, for A-36 steel, $L/r_{\max} = 120$).
5. The lateral force distribution between members in tension or compression in any line of bracing (column line) is restricted. A maximum of 70% of the total force resisted by each bracing line may be resisted by members in either tension or compression.

This requirement is exempted if it can be shown that the compression bracing alone has the strength, neglecting the " β " factor, to resist $3(R_w/8)$ times the seismic force tributary to that line of bracing

What this requirement means is the following:

- Each bracing line must have at least two braces (one in tension, one in compression), such as a single bay of X-bracing.
- Bracing lines requiring many braces must have a roughly equal distribution of members in tension and compression.
- This requirement also restricts the practice used by some engineers of calculating the net bracing line capacity by saying that bracing line capacity = " x " members times permitted compression plus " y " members times permitted tension (as opposed to using principles of stiffness to distribute lateral forces between elements in tension and compression and then comparing these forces with member capacities).

4.5.3 CONNECTION STRENGTH REQUIREMENTS

The required strength of bracing connections should be adequate to prevent failure modes by out-of-plane buckling of gussets or brittle failures of the connections. Brittle connection failures, such as shown in Figure 4-4, prevent inelastic behavior to develop in the brace as intended.

For seismic zones 2, 3, and 4, UBC requirements for the required strength of connections are as follows:

1. Bracing connections must be designed with adequate plastic strength to resist the lesser of:
 - The tensile strength of the bracing.
 - $3(R_w/8)$ times the code seismic force in the bracing.

- The maximum force that can be transferred to the brace by adjacent elements of the structure (for example, due to the plastic strength of attached collector beams), or the limit as determined by the overturning uplift which an attached foundation is capable of resisting. (when using this requirement with pile foundations, remember that the actual pile uplift capacity may be much greater than the values employed for the foundation design).

This requirement applies to the connections of all of the bracing system parts which frame together at a brace joint - diagonal members, beams, and columns. It also applies to the column baseplate connections which transfer shear, compression, and uplift forces from the bracing system into the foundation or supporting structure.

2. In order to prevent bolted brace connection failures such as shown in Figure 4-4, the ratio of effective net area to gross section area of the connected elements must meet the following net-to-gross area ratio requirement:

$$A_{eff}/A_{gross} \geq 1.2 F'/F_u$$

Where:

F' = The force, including increases from item one, above, being transferred at the section where A_{eff} is being considered.

A_{gross} = the gross area of the section

For connections using bolted elements, $A_{eff} = U A_{net}$, but may not exceed $0.85 A_{gross}$. Values of U are listed in Section 4.4.3, as well as References 4-5 and 4-6.

For connections using welded elements, $A_{eff} = U A_{gross}$, but may not exceed $0.85 A_{gross}$. Values of U for welded connections are provided in References 4-5 and 4-6.

4.5.4 CONNECTION DETAILING PRACTICE

The size and shape of a gusseted connection is generally dictated by fastener requirements. The thickness of the gusset plate is selected on the basis of one or more of the following:

- buckling of the gusset plate, based on the maximum stress calculated on the Whitmore Section, or
- block shear failure of the gusset due to tensile forces.

The Whitmore Section stress procedure (described in Reference 4-13) estimates the maximum stress in the gusset plate and the plate buckling capacity. The block shear concept (References 4-5 and 4-6) predicts the ultimate strength of a bolted gusset plate in tension.

When detailing gusset connections, two options are available to the designer. One is to detail the connection so that the axes of all members framing into the connection pass through a common point (Figure 4-37a). This alternative insures that the force in the diagonal brace does not produce a moment in the connection and is termed a concentric connection. The loads along the gusset plate edges produced by this arrangement (Figure 4-37b) are difficult to assess, although some assessment is required in order to evaluate the required plate thickness and attachment weld sizes. In addition, for very steep or flat bracing angles, this arrangement can produce a long gusset plate which requires additional thickness or stiffening to prevent buckling before the brace reaches its full capacity.

Fig. 4-37: Axes of All Framing Members Pass Through a Single Point

Source: Reference 4-12

Fig. 4-38: Axes of Diagonal Bracing Members Pass Through the Corners of the Gusset Plates

Source: Reference 4-12

A second option is to select a "working point" which permits a simplified approach to determining the loads on the edges of the gusset plate. For example, if the line of force in the diagonal brace passes through the corner of the gusset plate, a common and simple procedure is to assume that the vertical component of the brace force is carried by the gusset-to-column connection and the horizontal component is carried by the gusset-to-beam connection (Figure 4-38b). In addition to simplifying the design of the gusset attachments, this approach generally produces a compact and less expensive gusset plate. However, if the line of action of the force in the diagonal member does not pass through the intersection of the beam and column centroidal axes, the force in the diagonal member will produce a moment in the connection (Figure 4-38a). Due to the moment produced by this

arrangement, such a connection is termed eccentric (or non-concentric). The moment, if significant, must be accounted for in the selection of the beam and column members.

Connection eccentricities can induce three types of behavior in structural systems:

1. Very small eccentricities can affect the distribution of stresses in components within the body of the connection.
2. Connection eccentricities can affect stresses within, and the resulting capacities of, connecting members.
3. Connection eccentricities can affect the stiffnesses of the connecting members, thereby affecting the distribution of forces within the general structure.

As an example, eccentricities which exceed the following values can cause more than a 10% increase in lateral deflection (which presumably is significant to effect stiffness distribution)

- When measured along the longitudinal axis of the member resisting the eccentricity, the eccentricity exceeds 3 times the member's radius of gyration.
- When measured perpendicular to the longitudinal axis of the member resisting the eccentricity, the eccentricity exceeds 1.5 times its radius of gyration.

Although these eccentricities may cause only about a 10% change in the stiffness of bracing systems, the stresses within the connected members (excluding the $3R_w/8$ factor described below) may be increased by anywhere from 100% to 400%, depending upon the geometry of the shape subject to bending.

Members and connections of CBF systems that are subjected to shear and flexure caused by eccentricity within joints must be designed and detailed to have adequate strength to resist $3R_w/8$ times the design seismic forces in combination with all other applicable loads.

Connection Details. Actual bracing connection details used vary considerably according to the magnitudes of forces. Figure 4-39 shows an example of a detail appropriate for relatively light force levels, while Figures 4-40, 4-41, and 4-42 are details which might be appropriate for moderate force levels. Significant aspects of these details are as follows:

1. Typically the details use shop-welded connections. Field connections can use either high-strength bolts or welds (bolts are shown).

2. The connection of the gusset/beam assembly to the column can use either a long bolted tab plate (Figure 4-41) or a pair of long framing angles (Figure 4-42). Where large vertical shear forces must transfer through the gusset into the column, the greater number of bolts provided by the double angles provides greater vertical shear strength. On the other hand, where large horizontal forces must be transferred through the column (such as from a collector beam on the far side of the column), the tab plate provides greater rigidity and tensile strength than the angles.
3. The beam flanges are chipped (one or both sides of the flange are cut and ground flush) to permit a single, continuous tab plate or pair of framing angles to be used. This allows easier alignment of parts and punching of holes in the shop, less fit-up problems during erection, and more even bearing of the members.
4. The wide-flange bracing is connected through the flanges instead of the web. This offers the following advantages:
 - The force path is more direct.
 - The connections are more compact.
 - The bearing and tearout strength of the flanges is greater than that of the relatively thin web.

Fig. 4-40: Moderate Brace-to-Girder Detail

Source: Reference 4-9

Fig. 4-41: Moderate Brace and Girder-to-Column Flange Detail

Source: Reference 4-9

- The flange plates provide a convenient mechanism to stiffen the gusset plate to prevent buckling.

For heavier forces, the connection details shown in Figures 4-43 and 4-44 are appropriate. These connections differ from the previous details in the following respects:

1. A double gusset plate is used rather than single gussets.
2. The web of the girders lie in the horizontal plane rather than in a vertical plane, which reduces the total gravity load carrying capacity of the girders.
3. The connections possess greater torsional and out-of-plane rigidity due to the double gusset plate system and are therefore less prone to buckling failures.

Fig. 4-43: Heavy Brace and Girder-to-Column Connection

Source: Reference 4-9

Fig. 4-44: Heavy Brace to Girder Connection
(Note: The circled numbers identify the Whitmore Section used for checking compression in the brace gusset.)

4.5.5 THE BASE PLATE CONNECTION

The base plate can be viewed as not a direct part of the bracing system, but rather, a separate element of the lateral-load resisting system which connects the bracing and foundation systems. As such, the load multiplication factors required for bracing connections should not necessarily be applied to forces resisted by the base plate. Rather, separate factors of safety may be appropriate, which are derived on the basis of the ductility of the base plate system, when subjected to lateral shear and vertical tension/compression forces.

- For lateral shear forces, the base plate is a necessary element to resist the horizontal component of forces imposed by attached diagonal bracing elements. The performance of most base plates when subject to large shear forces is relatively non-ductile. Hence, it is appropriate to require that the base plate be designed with a greater margin of safety than the bracing system, by increasing lateral forces by a factor of 1.5, the same factor required for bracing connections.
- For axial tension/compression forces, the base plate is not a direct part of the bracing system, since the vertical component of forces from bracing elements is resisted by the column. The performance of most column base connections is relatively ductile in tension and compression. However, some details can be non-ductile and therefore require additional margins of safety. Specific recommendations for uplift design forces are given in Chapter 6, "Foundation Systems."

Figure 4-45 shows an example of a base plate detail, in which a "shear lug" is provided to transfer shear forces into the foundation. The use of these lugs may be required where high shear forces occur, such as at locations where large diagonal bracing elements connect to the foundation.

4.6 STEEL ECCENTRIC-BRACED SYSTEMS

4.6.1 FRAME GEOMETRY AND LINK LENGTH CONSIDERATIONS

Eccentrically braced frames (EBFs) can be considered a hybrid structural system, combining the stiffness of conventional concentrically braced frames with the ductility and energy dissipation capacity of conventional moment resisting frames. Typically, the installed cost of EBF framing is somewhat higher than either comparable CBF or SMF construction. EBF construction, therefore, most commonly occurs: 1) in structure heights of 160 to 240 feet, where CBF construction is not permitted, and 2) in situations where drift is important, and

where the reduced magnitude of shear and overturning forces (as compared to CBF construction) permits a reduction in the type or size of foundation required. Another potential application for EBF systems is in structures with significant irregularities in vertical mass distribution. By adjusting member sizes and link lengths, EBF systems can adjust frame stiffness within a wide range to roughly equalize the stiffness/mass ratio at each story of a structure.

The distinguishing characteristic of an EBF is that at least one end of every brace is connected so as to isolate a segment of a beam called a "link." Figure 4-46 illustrates common EBF arrangements, with the link lengths identified by the letter "e." In a well designed EBF, inelastic activity under severe cyclic loading is restricted primarily to the links, which are designed and detailed to sustain large inelastic deformations without loss of strength. Remaining bracing elements (brace, column, and beam) are sized to provide a minimum strength which exceeds the governing shear or bending strength of the link beam.

Single diagonal bracing, Figure 4-46(a), is useful for narrow bays. If possible, this EBF type should be used in symmetrically opposing pairs (one tension, one compression) to maintain overall symmetry in the inelastic lateral response of the frame. For wider bays, the EBF types shown in Figs. 4-46(b) and (c) are preferred. The EBF type shown in Figure 4-46(b) is particularly advantageous because of its symmetric configuration and because the links are not connected to the columns. Thus, simpler and less expensive beam-to-column connections can be used.

For all configurations, each brace is typically arranged with only one end attached to a link beam. This; 1), provides a more predictable inelastic response and 2), improves the economy of construction by permitting a more standard nominally concentric CBF connection to be used at one end of the brace.

Fig. 4-46: Typical EBF Geometries

Source: Reference 4-15

An important factor in the geometry of the EBF is the length of the link beam. For very short links, shear dominates inelastic link behavior, whereas for long links, flexure dominates. A link is considered short if its length "e" satisfies the following:

$$e \leq 1.6 M_s / V_s$$

where:

$$\begin{aligned} M_s &= \text{Fully Plastic Moment Capacity,} \\ &= Z F_y \quad \text{for } f_a \leq 0.15 F_y \\ &= Z (F_y - F_a) \quad \text{for } f_a > 0.15 F_y, \text{ and} \end{aligned}$$

$$V_s = 0.55 d t_w F_y = \text{Fully Plastic Shear Capacity}$$

Links satisfying this criteria yield primarily in shear. Because the shear force (and hence the distribution of yielding) is nearly constant across the link length, short links are capable of very large plastic rotations and have excellent energy dissipation capacity. Longer link lengths therefore require somewhat heavier sections to achieve the same ductility provided by short links.

In the length range of $1.6 M_s/V_s$ to $2.6 M_s/V_s$, links typically experience significant amounts of both shear and flexural yielding. In this range, links are termed *long links*. Greater than $2.6 M_s/V_s$, flexure dominates, and the total energy absorption capacity is reduced. Although permitted by code, link lengths greater than about $2.2 M_s/V_s$ are not recommended, due to the reduced ductility provided.

The design of EBF systems is generally controlled by stresses and not deflections. However, lateral deflections and link rotations must be checked. Frame drift is calculated as $3(R_w/8)$ times the elastic drift due to the basic code seismic force. The link rotation angle, Θ , is defined as the plastic angle between the link and the beam segment outside the link which occurs at the maximum frame drift. This angle is commonly calculated by the slightly

conservative practice of assuming that all of the lateral frame drift is caused by plastic deformations of the link mechanism, as shown in Figure 4-47.

Link beam inelastic rotations are limited to $\theta \leq 0.06$ radians for short link beams. At this level of short link rotation, some local damage to floor and nonstructural contents damage may occur in the vicinity of the link beam. For long link beams, the link beam inelastic rotation limit is decreased linearly to a more conservative $\theta \leq 0.015$ radians at $2.6 M_s/V_s$.

4.6.2 BUILDING CODE REQUIREMENTS

UBC code requirements for seismic zones 2, 3, and 4 are identical.

Selecting Member Sizes. Separate selection requirements are provided for link beams, braces and columns, as follows:

1. The link beams in EBFs must comply with the "compact" requirements of the AISC, except that the flange width/thickness ratio is limited to:

$$b_f/2t_f \leq 52/\sqrt{F_y}.$$

For most bracing geometries, the horizontal component of the brace force at yielding in the link beam will cause the axial stress in the beam to be greater than $0.16 F_y$, thus limiting the d/t of the beam webs to:

$$d/t \leq 257/\sqrt{F_y}$$

Since the addition of web thickener plates to the link beam is not permitted, the designer is therefore effectively limited to a reduced choice of rolled shapes. This requirement is quite restrictive, particularly since

most EBF research has been done using WF sections which did not comply with this requirement.

As an alternative, the designer might consider using the web slenderness criteria provided by the AISC LRFD specification for high-seismic areas, which includes an allowance for axial stresses:

- For $P_n/\phi P_y \leq 0.125$, $d/t_w \leq (1 - [1.54 P/\phi P_y]) 520/\sqrt{F_y}$
 - For $P_n/\phi P_y > 0.125$, $d/t_w \leq (2.89 - [P/\phi P_y]) 152/\sqrt{F_y}$
2. Each brace must have a compressive strength of at least 1.5 times the axial force necessary to develop the governing link beam strength (bending or shear). The design of the brace and its connections should also include relevant CBF code requirements, including the use of "stress-reduction factors," β .
 3. Columns must be designed to remain elastic at 1.25 times the governing strength of the EBF bracing. In seismic zones 3 and 4 the column requirements described in Section 4.3.4 must also be observed and will generally govern over this requirement.

Link Beam Detailing. Basic requirements for link beam detailing are summarized on Figures 4-48 and 4-49. Specific considerations include:

1. The use of web doubler plates within link beams is not permitted. No openings may be placed within the web of a link beam.
2. The web shear may not exceed $0.8 V_s$ under the basic code seismic forces (this requirement scales stresses into the range of working stress design, where the maximum permitted shear stress is $F_v = 0.40 dt$).

Note: A common starting point for the design of link beams is to first estimate the required web area to resist seismic forces using $A_w = V/(0.8 \times 0.55 dt)$, and then estimate the required Section Modulus from the web shear strength.

3. Web stiffeners, located on each side of the web, are required at each end of the link beam (where the brace connects). In addition, for all long-link configurations (link length greater than $1.6 M_s/V_s$), additional pairs of stiffeners are required at a distance of b_f from each end of the link to preclude the possibility of flange buckling.

Fig. 4-48: Link Beam and Intermediate Stiffeners

After Reference 4-19

Fig. 4-49: Link Details

After Reference 4-14

4. Intermediate web stiffeners should be provided:

- for all short-link configurations (link length less than $1.6 M_s/V_s$), or
- for configurations where a combination of link length and axial stresses ($f_a > 0.15 F_y$) cause flexure to govern (by $M_s = Z [F_y - F_a]$), and the resulting shear stress at M_s exceeds $0.45 F_y dt$.

These stiffeners are required to assure development of the shear strength of the web by preventing web buckling. For link beams which are less than 24 inches in depth, intermediate stiffeners may be placed on one side only; otherwise, they must be placed on both sides of the beam web.

Code requirements for the size and spacing of intermediate stiffeners are empirically based on the results of testing. Stiffeners must be at least 3/8 inch thick, and must extend to the edge of the link beams flange (i.e., width of each side = $(b_f/2) - t_w$). Stiffener spacing is a function of the calculated link beam rotation angle, Θ , as follows:

- For link beam rotation $\Theta = 0.06$ radians, the spacing may not exceed $38 t_w - d/5$.
- For link beam rotation $\Theta \leq 0.03$ radians, the spacing may not exceed $56 t_w - d/5$.
- For rotation angles between 0.03 and 0.06 radians, interpolation is used.

5. Where, due to the configuration of the EBF, a link beam must connect directly to the column, the connection must be able to develop the full plastic capacity of the beam, both in bending and in shear. The conventional moment-type flange connection, which uses full-penetration welds of the flanges to the column and continuity or stiffener plates as required, is adequate to develop the bending capacity. To develop the shear capacity, conventional bolted-web connections are not acceptable, since excessive bolt slippage occurs. Either a welded shear tab, as shown in Figure 4-49, or a full penetration weld directly between the link web and the column are acceptable.

6. If the link beam is not located adjacent to the column (or for that portion which is not), the connection at the beam-column interface is the same as for conventional construction (i.e., bolted tab-plate or double-angles). The connection must be able to resist beam rotation due to accidental eccentricities out of the plane of the frame from the brace. The connection

must also be able to resist a torsional moment of at least $(0.01 F_y b_f t_f d)$ in order to prevent beam rotation under the large loads of the brace.

Brace to Link Connections. The use of traditional gusseted connections for the brace to link detail, as shown in Figure 4-50, is not recommended. Tests have shown that the large beam rotations which occur around the connection can induce buckling of the gusset. Instead, if a gusseted connection is desired, a detail such as that shown in Figure 4-49, which provides a stiffener at the unsupported edge, is recommended. The forward edge of the gusset stops at the end of the link beam because the UBC requires that no part of any connection may extend into the link beam (this does not prevent the member centerline "work points" from extending into the link beam, however).

More common details, despite the use of overhead welding, are shown in Figures 4-51 and 4-52. Note that, in Figure 4-52, vertical stiffeners are required in the beam, directly above the webs of the tube brace, in order to fully develop the strength of all four sides of the tube brace.

Link End Lateral Bracing. Lateral flange bracing must always be provided at each link end to maintain the stability of both the link and the diagonal brace through the large inelastic deformations required of the link. The code-required minimum strength of the link end braces is specified as 1.5 percent of the beam flange yield force, defined as $(b_f t_f F_y)$. Since moments are reversible, both the top and bottom flanges must be restrained. Typical bracing details are similar to those common for beams in SMF systems, and are described in Section 4.4.4.

Current research has suggested that lateral bracing for long link configurations may need to be designed for a somewhat higher brace force, perhaps as much as ten times

Fig. 4-50: Gusset Detail Susceptible to Buckling
(Not Recommended for EBF)

Source: Reference 4-15

Fig. 4-51: Brace Gusset Connection
WF Brace-to-Link

Source: Reference 4-8

higher than the 1.5 percent minimum. It is possible that the relatively large vertical displacements of the link beam may induce forces into attached components due to large-displacement effects. These effects can be reduced somewhat by always connecting braces and other members onto link beams (short or long) from one side only; not on both sides. Also, some conservatism in the design of lateral bracing is warranted, particularly for long link beams. On this basis, a design force for the design of link-beam lateral bracing might be taken as 4% to 5% of the beam flange yield force (vs. the 1.5% minimum required by the UBC).

Fig. 4-52: Brace Gusset Connection
Tube Brace-to-Link

4.6.3 GENERAL CONSIDERATIONS FOR EBF SYSTEMS

Additional design considerations for EBF systems are as follows:

1. In general, both the beam and brace must be treated as beam-columns in design. Because the capacity of the link beam is reduced by any axial forces which must be transferred through the link, it is advisable where possible to always orient the link so that any drag struts or collectors carrying large forces are attached to the non-link end of the beam.
2. Experimentally, as well as analytically, it has been demonstrated that the first floor links usually experience the largest plastic deformation. In extreme cases this may result in a tendency to develop a soft story. The plastic link rotations tend to attenuate at higher floors, and decrease with the increasing frame periods. Therefore, for tall structures (greater than 100 feet) in severe seismic applications, a conservative design for the links in the first two or three floors is recommended. This can be achieved by increasing the minimum design shear strengths of these links on the order of 10% above minimum design levels.
3. By changing the link lengths the stiffness of an EBF can be modified. In this manner the frame periods and vertical distribution of stiffness/mass can be optimized.

4.7 PREFABRICATED METAL BUILDINGS

Prefabricated, or pre-engineered, metal buildings are available in standard framing sizes and types from a variety of manufacturers as proprietary products. The general arrangement of a pre-engineered metal shell building consists of a standardized one-story frame, of modular width and height, and using as many frames as required to achieve the overall desired building length. Available frames can provide column-free spans varying from 10 to 80 feet, depending upon manufacturer and design load conditions. With interior columns, standard frame spans of up to 300 feet can be supplied. Most manufacturers feature computer-aided design and manufacturing which provides design flexibility, quick response to clients, and a maximum of economy in the structural design and use of materials.

4.7.1 STRUCTURAL SYSTEM

General aspects of most types of prefabricated metal building framing are shown in Figure 4-53. Typical building roof and wall cladding consists of painted, corrugated metal siding. This siding is generally attached either with metal screws or snap-lock fasteners. It is very flexible, and is not intended to function as a structural diaphragm. Concrete tilt-up or masonry walls can also be provided for the exterior siding, either attached to the structure as shear walls, or isolated from the structure as non-structural panels.

Where metal siding is used, the siding is supported against vertical and lateral loads by a system of girts (at walls), purlins (at the roof), and by an eave strut (at the roof eave lines), see Figure 4-53. Girts and purlins are commonly cold-formed "Z" or "C" shapes, although in traditional industrial buildings rolled channels or wide-flange shapes are also used. "Z" shapes, as shown in Figure 4-53, are always oriented so that the top or outward flange is turned up, or towards the ridge. "C" shapes can be turned in either direction, however, normally they are oriented so that the flanges are turned down, in order to reduce trapped dust and debris. Although systems of sag rods are often used in traditional industrial buildings to reduce bending and deflections of girts and purlins, in most light prefabricated buildings these sag rods are not normally used. Instead, girts and purlins may be cambered or supported during erection to maintain a straight profile. After the siding is installed, adequate strength and rigidity is assumed to be provided by the siding and its fasteners to prevent lateral bending of the girts and purlins.

In the building transverse direction, building frames generally consist of welded, tapered rigid-frame beams and columns. In the longitudinal direction and for roof diaphragm forces, tension-rod bracing is provided. Rod bracing is typically located between columns near each end of the building in order to brace the frame during erection, plus at additional interior locations as load conditions require.

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Historically, prefabricated metal buildings have typically performed very well during earthquakes, primarily because they are very light and because of the excellent performance of steel buildings in general. Where problems have occurred, they have typically been as a result of:

1. The absence of critical structure components, such as rod or flange bracing, which had been either omitted during erection or removed at a later date. Prefabricated metal buildings do *not* typically have redundant structure systems and are vulnerable to the absence of components.
2. Deflection-related problems. Prefabricated metal buildings are very flexible, both vertically and laterally and the design specification developed by the Metal Building Manufacturer's Association (MBMA) has very few limitations for either vertical or lateral deflections. Deflections can be a particular problem if concrete tilt-up or masonry siding is present on the exterior.
3. Siding systems on prefabricated metal buildings can be more brittle and rigid than the tension bracing systems provided. Damage has occurred where building deflections have caused siding fasteners and brittle siding panels to fail (Figure 4-54).

Significant aspects of the structural design are as follows:

1. The frame beam-to-column connections typically use joints with bolted

Fig. 4-54: Damage to Building Siding

Source: EQE Files, Coalinga EQ, 1983

end-plate connections to form moment-frames, as shown in Figure 4-55. The bolted moment connections generally use bearing-type bolts and relatively thin plates, which are subject to some slip and rotation while working.

2. An important, and often marginal, component of the building moment-frames is the girder flange bracing, shown in Figure 4-55. These diagonal braces connect to the bottom flange of the roof beam, and to the interior flange of the column, and extend diagonally to connect to an adjacent girt or purlin, typically at 8 to 12 foot intervals for a short distance from the column (until girder bending moments cause the inside flange to transition from compression to tension). Flange braces are necessary to prevent the compression flanges of the frame from buckling.

3. Building components are typically not designed for *any* more load than required by the building code or the design specification. Where additional sprinklers, ducts, or other items are planned to be added after the building is erected, the weight of these items is typically included by specifying a "collateral load." Larger, concentrated loads can also be included by specifying them into the design and providing drawings during procurement (preferred) or by marking-up building manufacturer to

Fig. 4-55: Detail of Frame

show load points (which may result in added cost).

4. Building designs can be provided in accordance with UBC or other major code requirements, or in accordance with the MBMA specification. The seismic provisions of the MBMA specification tend to follow behind the UBC specification by several years and may not meet current requirements. As an alternative, most manufactures can meet the OMF requirements of the UBC, and a few can also, by special order, meet SMF requirements.

5. Tension-rod connections typically fasten into beam and column webs with some eccentricity, and often utilize "shoes" which pull against the member webs (Figure 4-56). As a result, some local distortion or bending of the web area can occur. A better detail often available uses a wider shoe (often consisting of a split section of pipe) which extends across the full web width, as shown in Figure 4-57. It is recommended that a detail similar to that shown in Figure 4-57 be specified as part of the procurement package.

6. Design calculations are generally not provided unless required. Where contract documents require submittal of calculations for review, the calculations frequently consist of computer printouts summarizing details of the moment-frame design, plus a few calculations for required bracing.

7. Building manufacturers sometimes undersize anchor bolts by assuming that friction will resist shear forces. Because the UBC does not allow building weight to be included in sliding friction calculations, anchor bolts should be sized in accordance with either UBC shear allowables or a similar criteria (such as an ICBO evaluation report).

Fig. 4-56: Common Rod Shoe Connection
(Not Recommended)

Fig. 4-57: Alternate Rod Shoe Connection

8. Figure 4-58 shows an example of a connection detail (not recommended) for a masonry or tilt-up wall to a prefabricated metal building. The detail has the following problems:
- The wall is not well-anchored to the building to resist out-of-plane wall forces. The wall should be restrained near the eave line at 2 to 4 foot intervals to resist these forces. Bolting the wall to the metal eave-strut would be inadequate, as the strut is too flexible and weak.
 - A masonry or concrete wall is much more rigid as a shear wall than the standard building tension bracing. Therefore the wall, not the bracing, tends to resist forces in the longitudinal direction of the building.

4.7.2 RECOMMENDATIONS

Procurement Stage

1. It is more likely that a well-engineered building will be provided by a reputable, well established vendor than a small, inexpensive vendor. Vendors should show evidence of successful past projects.
2. Specify and verify that the manufacturer will design the building in accordance with UCRL-15910 requirements, including all pertinent UBC seismic requirements. Unless substantial documentation in the form of calculations is provided, the moment-frames should be designed as ordinary moment frames ($R_w = 6$), and the tension-rod bracing should be designed using an $R_w = 8$ and an additional $3(R_w/8)$ factor. Also, specify and verify that P-delta effects are considered during the evaluation of design forces.
3. Plan for and specify collateral loads, both uniform and concentrated, as required. Include allowances for any future loads which might occur. Where possible, provide procurement drawings which specify load locations and magnitudes.
4. Specify that maximum deflections may not exceed the following:
 - Vertical deflections of roof members: a maximum deflection of $L/240$.
 - Lateral deflection for wind or seismic: 0.005 times eave height. This requirement may be heavily contested (or ignored) by a building manufacturer, however, this criteria has been successfully met by metal building manufacturers in the past.
5. Specify that anchor bolt allowable tension and shear values must be based on the values given in the UBC or on an equivalent UBC standard.

Design and Construction Stage

6. Most problems during the design and erection phase are a result of communication problems. It is not uncommon to find that building drawings call for tension-rod bracing to be installed in the same location where a door or window opening is planned.
7. Compare the erected building with the manufacturer's drawings to verify that all structure elements have been properly installed.
8. If concrete or masonry siding is planned, a detail such as the one shown in Figure 4-59 should be considered to support the wall out-of-plane. If the roof bracing is

connected to the beam, then the beam can also be used to transfer longitudinal building forces into a shear wall.

- If the wall is designed as a shear wall, then at least some vertical rod bracing is still required as erection bracing (the standard rod bracing could remain so that the walls could be removed at a future date for expansion).
- If the wall is not designed as a shear wall, then elaborate details are required to support the wall out-of-plane while allowing the building to move independent of the wall longitudinally. The use of slotted-hole connections in this regard is not recommended, since if overtightened the connections will not move. A better approach is to use a series of tension-compression

braces, as shown in Figure 4-60. The braces extend far enough from the wall to easily articulate by bending when the building moves longitudinally.

Post-Construction Phase

9. Bracing elements should not be removed for alterations or additions without an engineer's written approval.

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DIAPHRAGM SYSTEMS

Diaphragms are horizontal-spanning systems which tie structure elements together, provide support for walls and other elements, prevent excessive torsional (plan) deformations, and transfer forces to shearwalls or frames below. Seismic forces resisted by diaphragms originate as inertial loads or as loads transmitted from shearwalls and frames above.

The use of floor and roof systems as diaphragm systems to distribute seismic loads can be very efficient, since most of the elements required for diaphragm action are also required for the vertical load system. Horizontal bracing systems provide diaphragm-like performance for applications where solid-surface construction cannot be used or is not required.

5.1 INTRODUCTORY CONCEPTS

Diaphragms are generally considered as similar to very deep horizontal girders. Like a girder, a diaphragm resists in-plane shear forces by its web, which is the floor or roof deck. And, like a girder, a diaphragm resists flexural forces by axial tension and compression in its flanges, which are the diaphragm chords or boundary elements. These forces are shown in Figure 5-1.

The girder analogy may be quite useful in analysis, but it does not explain the complete behavior of a diaphragm. Special properties of diaphragms include the following:

1. Because of their great depth (often equal to or greater than their span length) bending in a diaphragm resembles that of a deep-beam; that is:

- shear deformation accounts for the

- major part of diaphragm deflection, and
 - plane sections do not generally remain plane (stresses are not linear across the depth).
- 2. Web shear stresses are generally more significant than flexural stresses.
- 3. Diaphragm webs and flanges are often made of many separate components, which must be well-fastened together for proper diaphragm action.
- 4. Diaphragms may have many supports, which may be oriented parallel to, perpendicular to, or at a skew-angle to the direction of loading.
- 5. Diaphragms are not necessarily rectangular, and may be subject to local areas of stress concentrations.
- 6. In its most general case, a diaphragm should be considered as a flexible beam on flexible supports. The relative rigidity of the diaphragm and its supports are of great importance in determining the magnitudes of reactions at supports.

5.1.1 DIAPHRAGM RIGIDITY

Probably the single most important factor affecting a diaphragm's behavior is rigidity, or more specifically, its rigidity with respect to its supports.

The analysis of a diaphragm and its supports must include an evaluation of the relative stiffness of both the diaphragm system and the supporting walls and/or frames. If simplifying assumptions are not made, this analysis can only be performed using finite-element methods, which are time-consuming and relatively expensive in comparison to the cost of analysis made using simplifying assumptions.

As a simplification, diaphragms are usually assumed to be either completely rigid or completely flexible, in comparison to the rigidity of the supporting elements.

- Rigid diaphragms distribute loads to supporting walls and frames in proportion with the rigidity and relative position of the walls or frames in the diaphragm. This behavior is similar to that of a rigid beam on flexible supports.
- Flexible diaphragms distribute loads to supporting walls and frames based only on the position and spacing of the supports within the diaphragm, in that the load in each support is equal to the sum of the loads which are

tributary to that support. This behavior is similar to that of a flexible beam on rigid supports.

In reality, many diaphragms actually behave in an intermediate, or semi-rigid, manner. It is up to the design engineer to decide whether the assumptions made are appropriate. Examples of diaphragm behavior are shown in Figure 5-2.

Some factors influencing relative diaphragm rigidity are:

- The material, configuration, and detailing of the diaphragm system.
- The materials, location, size, and detailing of supporting shearwalls or frames.

When confronted with a diaphragm which exhibits semi-rigid behavior, the design engineer may either perform a detailed analysis which accounts for the rigidities of both the diaphragm and the supporting systems, or consider using both rigid and flexible assumptions and designing for the resulting "envelope" of forces.

For the most rigorous analyses, finite element models of diaphragms may be prepared. This provides an accurate estimate of how the semi-rigid diaphragm distributes loads to the frames and shearwalls below. In this case, diaphragm chords and collectors can also be modeled.

The purpose of any type of diaphragm analysis (rigid, flexible, or semi-rigid) is to determine the magnitudes of reactions in the supporting walls and/or frames to a given set of applied forces. After the pattern of reactions is determined, the procedures used to calculate shear and moment diagrams and to calculate forces in chord and collector elements, as described in the following sections, are basically identical for all types of diaphragm systems.

5.1.2 DIAPHRAGM DEFLECTIONS

Diaphragm deflections consist of flexural and shear components and, for some types of diaphragm construction, of a component due to slippage between connected sections of the diaphragm. The flexural component is mainly determined by the deformations of the

diaphragm chords, while the shear component is determined by deformations in the body of the diaphragm or "web." For most diaphragms, the shear component is dominant.

The properties used to calculate deflection depend on the material of construction, as follows:

- For concrete and concrete-topped diaphragm systems, deflections can be calculated using the elastic properties of the concrete section. For most concrete diaphragms, the stiffness of the section is quite large, and stresses are generally relatively low, except in concentrated areas. Thus, the use of uncracked, rather than cracked, elastic section properties is appropriate.
- For horizontal bracing systems, deflections can be calculated by treating the bracing as a horizontal truss.
- For metal deck systems and precast concrete diaphragm systems which are not topped with concrete, mechanical slip between elements makes up a large part of the web deflection. Deflection calculations are usually made based on a flexibility factor, "F," which is generally provided in technical literature provided by diaphragm system manufacturers. This factor typically accounts for both direct shear deformation and for deformation due to mechanical slippage or deformations between components.

Where flexibility factors are not available from a diaphragm system's manufacturer, Reference 5-6 provides equations to calculate flexibility factors for many types of construction.

5.1.3 TORSION IN DIAPHRAGMS

Applicability of Torsion Requirements. Torsional effects are important for diaphragm systems constructed using rigid or semi-rigid material, because the stiffness of these materials cause them to distribute non-uniform forces and deformations into adjacent resisting walls and/or frames via twisting of the structure.

For diaphragms constructed with flexible material, torsional effects are typically ignored, since flexible diaphragm systems permit non-uniform deformations to occur between adjacent walls and/or frames, thereby causing eccentric loads to be absorbed by the nearest resisting elements.

Flexible cantilevered diaphragms are an exception to this. Rotational forces created by the cantilever portion must be resisted by either: 1) the back-span portion of a flexible diaphragm, as shown in Figure 5-3(a), or 2) by rigid walls or frames which are positioned in

the location of the chords, perpendicular to the direction of force, Figure 5-3(b). The actual mechanism of resistance depends upon the relative rigidity of any perpendicular walls or frames present to the rigidity of the diaphragm back-span. This cantilever condition is not true torsion, since there is no rigid-body rotation of the diaphragm.

Diaphragms are considered rigid, and therefore capable of torsional behavior, whenever the stiffness of the diaphragm is greater than or approximately equal to the stiffness of the vertical elements. The UBC considers a diaphragm capable of torsion if the maximum lateral deflection of the diaphragm is less than two times the average drift of the associated story. Thus, typical corrugated metal diaphragms (without concrete topping) would not be considered capable of resisting torsion if the vertical elements were concrete or masonry shearwalls, but might be considered to resist torsion if the vertical elements were flexible steel frames.

Torsion Design Requirements. Torsion in diaphragms occurs when the center of forces acting on the diaphragm (most often at or near the center of mass) does not coincide with the center of rigidity (as shown in Figure 5-4).

In diaphragms which transmit torsion (i.e. rigid and nearly rigid diaphragms), the UBC requires that an additional 5% "accidental" torsion be applied to the diaphragm. This "accidental" torsion is an addition to, not a replacement of, the torsion caused by the distance between the calculated centers of mass and rigidity. It is intended to account for variations in stiffness and mass which may occur in the actual structure, and for twisting motions which the structure may be subjected to by the variation in ground motions.

When torsion is considered, diaphragm forces should be calculated for three load cases in each direction of loading: 1) the calculated eccentricity, 2) the calculated plus 5% eccentricity, and 3) the calculated minus 5% eccentricity. If the assumption of rigid behavior of the diaphragm is in question, forces may also be calculated by tributary areas, assuming flexible diaphragm behavior. The maximum forces from these load cases should be used for design.

For regular buildings with rigid diaphragm systems, it is common to calculate the total mass at each story and develop calculations and computer models by applying this mass at a single point (the center of mass). In this situation, it is convenient to account for accidental torsion by performing several analysis while sequentially shifting the location of the center of mass in a direction perpendicular to the applied seismic forces by distances of plus and minus 5% of the building's maximum perpendicular dimension.

For buildings which are highly irregular, such as the structure shown in Figure 2-29, this type of analysis may be neither convenient nor appropriate. Typically, this type of structure might be analyzed with a complex, three-dimensional finite-element model. Masses would be applied at distributed locations throughout the structure, rather than at a concentrated point, and semi-rigid, rather than rigid, diaphragm behavior would be modeled. However, accidental torsion effects should be included in some manner for even this type of structure. In this case, an alternative approach to including accidental torsional effects might be to impose an equal-but-opposite pair of forces against each end of the finite-element model. The magnitude of these forces would be equal to the code-specified accidental torsional moment, divided by the distance between the forces.

An alternate approach to meeting accidental torsion requirements, which has been accepted on several past DOE projects, is to arbitrarily increase stresses determined for seismic load cases calculated without accidental torsion by 10% to conservatively envelope accidental torsion effects.

5.1.4 DIAPHRAGM CONFIGURATION

Figure 5-5 shows several common diaphragm configurations. Typical anticipated deflection patterns are included in the figure with dashed lines. The configurations shown are structural variations within a rectangular structure geometry. Irregular structure geometries

require more complex combinations of these basic diaphragm configurations, leading to more complex behavior.

Irregularities in diaphragms can be detrimental to structure performance. However, awareness of irregularities and proper design to mitigate their consequences can lead to acceptable performance. Basic types of structure plan irregularities are described in Section 2.4.2. Several irregularity types relevant to diaphragm systems include:

Type A: Torsional Irregularity (Fig. 2-24).

Structures are considered torsionally irregular if they meet the deflection criterion defined in Section 2.4.2 (flexible diaphragm systems are excluded, since they do not transmit torsion). If torsionally irregular, the UBC requires that the accidental torsion must be increased by an amplification factor (defined in Section 2.4.2). For most structures, this is not required, since the magnitude of irregularity must be relatively large before a structure is classified as torsionally irregular.

Type B: Re-entrant Corner/Geometry Irregularity (Fig. 2-25).

This is commonly seen in buildings with complex shapes, as shown in Figure 5-6. Analysis methods for notched, or re-entrant, diaphragms are described in Section 5.3.1.

Type C: Diaphragm Discontinuities/Opening (Fig. 2-26). Discontinuities in diaphragm systems can result both from large openings and from changes in material used for the diaphragm system. Analysis methods for diaphragm openings are described in Section 5.3.2.

Type D: Out-of-Plane Offsets (Fig. 2-27). Out-of-plane offsets can result both from plan offsets of walls and/or frames and from large changes in stiffness in individual walls and/or frames between story levels. Diaphragms normally need to resist only the small portion of total structure force which is tributary to the diaphragm level. Out-of-plane offsets cause all or a portion of the forces from the irregular elements to re-distribute through the diaphragm system, greatly increasing the total force which the diaphragm must resist.

A specific type of out-of-plane offset sometimes encountered is the "stepped diaphragm," shown in Figure 5-7. Stepped diaphragms are usually found in roof structures and high-bay industrial structures. Where diaphragms change level, they must be considered as separate diaphragms or be attached in such a manner as to transfer shears both parallel and perpendicular to the step. Where design conditions require diaphragm continuity, special bracing members are required which transfer diaphragm shear and chord forces across the step. Columns are required at each end of the step to resist the overturning moment caused by the change in elevation. The UBC requires special design and detailing provisions for these columns, as described in Sections 2.4.2, 3.3.4, and 4.3.4.

Type E: Non-Parallel Systems (Fig. 2-28). Because structures are not always rectangular, supporting walls and/or frames are not always oriented either parallel or perpendicular to the

direction of design forces, and diaphragm elements such as chords are sometimes configured with changes in direction. Analysis methods for diaphragms with non-orthogonal supporting elements are described in Section 5.3.4.

5.2 ANALYSIS OF DIAPHRAGMS

This section presents the general UBC seismic criteria for diaphragm design, which includes a minimum design force criteria and special design requirements for structures with certain types of irregularities. It also reviews the basic assumptions and design approaches used for the design of diaphragm components, including chord and boundary elements, drag and collector elements, diaphragm-to-wall anchorage elements, and sub-diaphragms.

5.2.1 BASIC UBC SEISMIC DESIGN REQUIREMENTS

Minimum seismic design forces for diaphragm systems. Normal code-defined seismic forces are distributed to each building story over a height varying from zero at the base to a maximum at the roof. For diaphragms, the minimum seismic design force at each building story must be determined by redistributing these forces over the height of the building based on the following formula:

$$F_{px} = w_{px} \sum_{i=x}^n F_i / \sum_{i=x}^n w_i$$

$$F_{px} \text{ minimum} = 0.35 Z I w_{px}$$

$$F_{px} \text{ maximum} = 0.75 Z I w_{px}$$

where:

F_{px} = the seismic force resisted by the diaphragm at structure level "x."

w_{px} = the diaphragm weight at structure level "x."

n = the total number of building stories or structure levels

F_i = the basic code-defined seismic force at structure level "i."

w_i = the tributary structure weight at structure level "i."

The minimum value used is equal to that for a building of nearly zero period and an $R_w = 8$ (i.e., $C/R_w = 2.75/8 \approx 0.35$). Thus, the minimum only governs for relatively rigid structures, with $R_w > 8$. The maximum value used is based on the C_p value required for the design and attachment of many types of individual structure components.

These equations distribute seismic forces from a value proportional to the building base shear at the base to a maximum at the roof, as shown in Figure 5-8. In the figure, the force

distribution vs. height for diaphragms is shown, together with the force distribution of the UBC basic force equation. Note that, unless " F_{px} maximum" governs, the diaphragm design force is always equal to the basic structure design force at the topmost, or roof, level of a structure. Thus, for one story buildings, the calculation of F_{px} for the design of the roof diaphragm is unnecessary.

The requirement for special diaphragm design forces is based on consideration of higher modes of building response at various levels of the structure and is intended to account for local accelerations which may occur within a structure during an earthquake.

The force determined by this equation is used to design diaphragm components (web, chords, and collectors/distributors) only. Reactions calculated from a diaphragm to supporting walls and/or frames may be greater than the forces specified by the basic force equation, however, the supporting elements need not be designed for this larger force. Supporting walls and frames are designed for the story shear resulting from the basic code force equation.

Connections to Vertical Elements and Collectors. In Seismic Zones 3 and 4, structures having some types of plan irregularities, as defined in Section 2.4.2 are not allowed the normal 1/3 increase in allowable stress for connections between shearwalls, frames, and collectors, or from these elements to the diaphragm. Where diaphragm elements are designed using ultimate strength (USD or LRFD) procedures, this requirement may be satisfied by using a load factor of 1.7, in lieu of the normal 1.4 factor specified by the UBC for load combinations which include earthquake forces.

Independent Movement of Projecting Wings. In Seismic Zones 3 and 4, structures having re-entrant corners greater than 15% of the building plan dimension have special

Fig. 5-9: Movement of Projecting Wings

requirements for chords and drag elements (collectors/distributors). These elements must be designed considering independent motion of each wing, either in the same or opposing direction as other wings, using a force $F_p'x$ calculated for the tributary portion of the projecting diaphragm areas, as shown in Figure 5-9.

The requirement is intended to account for stress concentrations at re-entrant corners that might not otherwise be considered in a static analyses.

As an alternative, this requirement may be satisfied by a three-dimensional, dynamic analysis which includes the effects of diaphragm rigidity.

Caution: The finite-element program used must recognize diaphragm flexibility; programs which assume rigid diaphragm behavior artificially prevent independent motions of projecting wings from occurring. Several popular finite-element programs (i.e., some variations of TAB and ETAB) assume that diaphragms are infinitely rigid, and so will not recover the required stresses.

5.2.2 CHORD AND BOUNDARY ELEMENTS

In the girder analogy, tensile and compressive stresses at the diaphragm boundaries are resisted by the flanges. The diaphragm flanges consist of "chord" and local boundary elements. These elements may take the form of additional continuous reinforcement in concrete slabs or in concrete or masonry walls. Steel or concrete supporting members may also act as chords or boundary elements. When designing members to act as chords, both tensile and compressive forces must be considered. For concrete diaphragms, special boundary elements are required when compressive stresses due to diaphragm moments exceed $0.2 f'_c$. These requirements are described in Section 5.6.

Calculation of chord forces at any point in a diaphragm may be accomplished by calculating the diaphragm bending moment at a point and resolving it into tensile and compressive forces at the diaphragm boundaries. For a rectangular diaphragm which is supported at each end this becomes a simple calculation, as shown in Figure 5-10. Flexible diaphragm systems are normally designed entirely on this basis, by assuming that each diaphragm span between supporting elements is a simple span (i.e., no moment continuity across supports).

Fig. 5-10: Maximum Chord Forces in a Simple Diaphragm

Rigid diaphragms often exhibit much more intricate behavior. Where reactions due to torsion or multiple lines of resistance are present, calculations are more elaborate. In Figure 5-11, a diaphragm is shown with four lines of lateral resistance parallel to the load. The forces assumed result from a uniform load. Figure 5-12 shows the resulting diaphragm chord force diagram.

Additional problems, discussed in Section 5.3.3, result where torsional stresses are present.

5.2.3 DRAG AND COLLECTOR/DISTRIBUTOR ELEMENTS

Where shearwalls and frames are attached to the diaphragm, drag or collector/distributor elements should be provided to transition the concentrated reaction forces from walls or frames into uniform shear stresses within the diaphragm. In many cases, the design of collector elements requires only slight size increases and/or strengthening of connections of beams or girders already required for vertical loads.

Calculation of collector design forces is most easily accomplished by the use of an axial-force diagram such as shown in Figure 5-13. In the figure, the uniform shear accumulated (collected) along the line of resistance is shown above the collector; the shear resisted by

Fig. 5-11: Forces Against and Reactions in a Rigid Diaphragm

$$M(x) = \sum_{i=1}^4 \left[-V_i (x - x_i) \right] - \frac{F_\perp d_\perp x}{\ell} + \frac{W_1}{2} x^2 - \frac{(W_1 - W_2)}{6} x^3$$

$$F_{\text{CHORD}} = M/d$$

Fig. 5-12: Chord Force in Rigid Diaphragm

(distributed to) the shearwalls and frames is shown below. The net axial force in the collector/distributor member is shown in the lower figure. To satisfy static equilibrium, the collector force should always be zero at each end.

The arrangement of shearwalls or frames along a line of resistance can greatly influence the required collector forces. In general, the lowest collector forces arise from a near uniform distribution of walls or frames across the diaphragm. A single shear resisting element located at the end of a diaphragm results in the highest collector force.

5.2.4 DIAPHRAGM-TO-WALL ANCHORAGE

Diaphragms provide lateral support to exterior building walls for seismic forces which are perpendicular to the plane of the walls. When exterior walls are constructed of

concrete or masonry, the UBC requires a "positive and direct connection" between walls and diaphragms. This is especially important for bearing walls. Special attention has been paid to this problem since the 1971 San Fernando Earthquake, when numerous concrete tilt-up buildings partially collapsed due to inadequate roof diaphragm-to-wall anchorage. Although the impetus for this provision was the failure of numerous wood-framed roofs, diaphragm-to-wall anchorage should be carefully considered, regardless of the diaphragm construction materials.

Minimum lateral forces used to design diaphragm-to-wall anchorage and sub-diaphragm systems (see next section) are based on the following formula:

$$F_p = Z I C_p W_p$$

where:

F_p = the minimum design lateral force resisted by the structural or nonstructural component.

W_p = the weight of the structural or nonstructural component.

C_p = a force coefficient. Tabulated values of C_p for different structural and nonstructural components are defined in the UBC.

For structure elements and components which are either non-rigid or flexibly supported (defined in the UBC as having a period greater than 0.06 seconds), the value of C_p used must be taken as twice the tabulated C_p value listed in the UBC. As a special case, the UBC requires that the value of C_p used to determine diaphragm-to-wall anchorage forces for flexible diaphragm systems must be increased by 50 percent from the tabulated C_p value listed in the UBC for the center one-half of each diaphragm span, as shown in Figure 5-14.

Fig. 5-14: C_p Increase Factor for Flexible Diaphragms (Plan View)

The design lateral forces determined using this equation are somewhat greater for any one component than the design forces used for the structure as a whole (defined in Section 2.1.5). This accounts for locally greater forces which might occur within individual components during an earthquake. The component design forces or stresses determined using this equation should not be added or superimposed onto the general forces used to analyze the structure as a whole.

5.2.5 SUB-DIAPHRAGMS AND CROSS-TIES

Sub-diaphragms are local sections of the main diaphragm that resist inertial forces from diaphragm-to-wall anchorage elements, as shown in Figure 5-15. These forces are then transferred in a positively connected and rational manner into a system of "cross-ties" which extend across the full width of the diaphragm to hold the diaphragm together. Although the primary use of sub-diaphragms and cross-ties is for wood diaphragm systems, the concept of these systems is also applicable to a limited extent for the flexible metal diaphragm systems described in Section 5.4.

Sub-diaphragms are sized and located based upon their span-to-depth ratio and tributary load levels. Although limiting span/depth ratios are only defined in the UBC for wood diaphragm systems, sub-diaphragms of any material should not be permitted to span distances of more than 4 times their depth. Where support member spacing causes spans to exceed this limit, additional sub-diaphragms are sometimes used to collect diaphragm-to-wall forces and transfer them

Fig. 5-15: Sub-Diaphragm (Plan View)

into successively larger sub-diaphragm systems. Reactions from sub-diaphragms are resisted by cross-ties. Cross-ties generally consist of beams or girders which extend continuously between opposing walls and which have adequate tension/compression connections to transfer the sub-diaphragm reaction across the building width.

Critical design elements of sub-diaphragms include the following:

- *Shear stresses within the sub-diaphragm.* Increased nailing, deck welding or mechanical fastening may be required in some areas to provide greater shear strength.
- *Sub-diaphragm chords.* Bending moments and chord stresses must be calculated for the sub-diaphragm span as if the sub-diaphragm were disconnected from the remainder of the roof. Since sub-diaphragms are usually located to utilize vertical-load beams and joists as chords, chord stresses are rarely a problem.
- *Cross-ties and cross-tie connections.* If vertical-support beams are used as the cross-ties, member stresses should not be a problem; however, special connections may be required at member ends in order to transfer tension and compression forces. For metal deck systems, the metal decking, if oriented parallel to the direction of force, may be able to transfer tension/compression cross-tie forces of small magnitudes directly, possibly removing the need for additional sub-diaphragms in some instances.

5.3 ANALYSIS OF COMPLEX CONDITIONS

Complex diaphragm conditions which typically require more elaborate analysis measures, include the following:

- Notched, or re-entrant diaphragm areas
- Diaphragms with openings
- Rigid diaphragm with torsion
- Diaphragms with non-orthogonal supporting elements

Suggested analysis approaches for these conditions are described in the following sections.

5.3.1 NOTCHED DIAPHRAGMS

Re-entrant building corners create notches in diaphragms that must be reinforced. Two drag struts are required at every notch, one extending each way into the diaphragm. In one direction, the chord strut functions to transition diaphragm chord stresses from the notch. In the other, the drag strut functions as a collector/distributor, to transition shear stresses accumulated from the "deep" side of the diaphragm into the "shallow" side.

The analysis of forces adjacent to a notch, as shown in Figure 5-16, is typically done as follows:

1. The diaphragm moment immediately adjacent to the notch point is calculated.
2. The smaller diaphragm depth is used to determine the chord force adjacent to the notch.
3. A drag strut, in line with the notched diaphragm chord, is continued into the diaphragm parallel to the main diaphragm chord in order to transition the chord force into the diaphragm.

Since the strut force is transferred to the main chord by shear, the length of the strut must be adequate to develop the notched diaphragm chord force into the main diaphragm without exceeding the allowable shear stress of the diaphragm in the zone in which the transfer occurs.

5.3.2 DIAPHRAGM OPENINGS

Local reinforcement must also be provided to resist chord and drag forces which are developed around openings in diaphragms. Figure 5-17 shows an example of a diaphragm

with a large opening. The free body shown includes only the opening and the adjacent diaphragm segments.

1. Shear is distributed to segments (1) and (2) in proportion to their length.
2. Applied lateral loads are distributed to segments (1) and (2) based on the forces tributary to each segment.
3. The segment moment is calculated assuming that each end of the segment deflects as a fixed-end beam ($wL^2/12$, however, many engineers conservatively use $wL^2/8$).
4. Sub-chords for each segment are sized based on the segment moments.
5. Sub-chord forces calculated at the diaphragm boundary are added to the chord forces calculated for the main diaphragm chords.

Fig. 5-17: Diaphragm Opening

The edges of the openings parallel to the load should also be reinforced for collector forces (as described in Section 5.2.3). Sub-chords and collectors should extend beyond the opening at least far enough to develop the required forces into the diaphragm without exceeding its allowable shear stress.

5.3.3 DIAPHRAGMS WITH TORSIONAL MOMENTS

Additional considerations are required to balance diaphragm shear and moment diagrams: 1) where accidental torsion forces are included, and 2) where a portion of torsional moments are resisted by elements perpendicular to the applied direction of force.

Accurate shear and moment calculations can be complex for rigid diaphragm systems. This complexity results from the following:

1. The applied forces against a rigid diaphragm develop a torsional moment, M_T , which consists of two components: the torsional moment due to mass eccentricity, plus the code-required accidental torsion moment. Expressed mathematically:

$$M_T = (F_x e) + M_{AT}$$

where:

M_T = the total torsional moment

F_x = the lateral seismic force acting against the diaphragm in the "x" direction

e = the eccentricity from C.M. to C.R.

M_{AT} = the accidental torsion moment

M_{AT} can be applied against the diaphragm either as a linearly varying uniform load, or as an equal-but-opposite pair of forces located at each end of the diaphragm. The effect on shear stresses due to M_{AT} is less if concentrated forces are used.

2. The resisting elements resist this torsion by developing force-couples about both axis of the structure. Thus, the torsional moment within the resisting system, M_{TR} , also consists of two components:

$$M_{TR} = -M_T = M_{TX} + M_{TY}$$

where:

$$M_{TX} = \sum (J F_{RX} / R_y)$$

the torsional moment caused by resisting elements parallel to the direction of the applied forces. This torsion is accounted for by

the support reactions used when calculating diaphragm shears and moments in the direction parallel to the applied forces.

$$M_{TY} = \sum (J F_{RY} / R \times)$$

the torsional moment caused by resisting elements perpendicular to the direction of the applied forces.

F_{RX} = reaction forces in the resisting elements which are in the "x" direction, parallel to the applied forces.

F_{RY} = reaction forces in the resisting elements which are in the "y" direction, perpendicular to the applied forces.

$$J = \sum R x^2 + \sum R y^2$$

R = rigidity of a resisting element

x, y = distance to the center of rigidity

To summarize, the diaphragm forces applied as uniform and concentrated loads may induce a torsion equal to " $F_x e$," to which the applied accidental torsion, M_{AT} , must be added. This torsion is resisted with a torsion " M_{TX} ," due to reactions in the direction of the applied forces, and a torsion " M_{TY} ," due to elements perpendicular to the direction of the applied forces. In order to balance internal forces and moments when constructing shear and moment diagrams, all of these torsional components must be included. The remainder of this section reviews how these components can be included.

Since the forces resisted by perpendicular elements are transferred into the diaphragm as uniform forces via collector elements, it is appropriate to treat " M_{TY} " as producing a uniformly distributed moment (internal torque), which is distributed across the diaphragm width (i.e., ft.lbs./ft.). As an example, Figure 5-18 shows a simple rigid diaphragm, 100 ft. by 50 ft. in size, in which the centers of mass and rigidity coincide. As shown at the top of the figure, a total lateral force of 100 kips is applied as a uniform load of 1 kip/ft against the diaphragm. This force results in reactions of 50 kips in the moment frames located at column lines "1" and "4." An accidental torsion moment, M_{AT} , is also applied, equal to 5% times 100 ft. times 100 kips = 500 ft.kips. This moment is applied in the example as a pair of forces applied against each end of the diaphragm (500 ft.kips / 100 ft. = 5 kips), as shown at the top of the figure. Based on assumed rigidities for the moment frames along column lines "1" and "4" and the walls along column lines "A" and "C," the accidental torsion produces reactions of 8 kips in each of the two perpendicular walls and 1 kip in each of the two moment frames. The net reactions in the walls and frames are shown in the structure plan view.

Diaphragm shear forces in the direction parallel to the applied forces are shown in the diagram below the plan view. At the left side, the diaphragm shear is 51 kips (frame

reaction) minus 5 kips (applied concentrated force), or 46 kips. Likewise, at the right side, the diaphragm shear is 54 kips. Because the accidental torsion is reversible, the diaphragm design shear force at each side would be $54 \text{ kips} / 50 \text{ ft.} = 1.08 \text{ kip/ft.}$

Diaphragm shear forces in the direction perpendicular to the applied forces, caused by the portion of M_{AT} which is resisted by the perpendicular elements, are shown in the diagram to the right of the plan view. The reactions in the two perpendicular walls are transferred from the diaphragm to the walls by collector/distributor elements. The force between the diaphragm and wall is therefore assumed to be uniformly distributed along the entire collector length. In the example, this results in a uniform shear force of $8 \text{ kips} / 100 \text{ ft} = 80 \text{ lb/ft}$ along column lines "A" and "C." These uniform forces resist " M_{TY} " by developing a

uniformly distributed torsional moment, equal to $(2)(80 \text{ lb/ft})(25 \text{ ft}) = 4 \text{ ft.kips/ft.}$, as shown in Figure 5-18 in the "Internal Torque Diagram." Note that 4 ft.kips/ft. times 100 ft. is equal to $400 \text{ kip ft.} = M_{TY}$, the portion of M_{AT} which is resisted by the perpendicular elements (8 kips times 50 feet).

The development of an internal torque diagram can be useful when calculating diaphragm bending moments and associated chord forces, since the diaphragm moment at any point can be expressed as the area under the shear diagram up to that point, minus the corresponding area from the internal torque diagram. This is because the bending moment at any point is equal to the sum of the moment caused by shear forces, *plus the sum of any moments applied*.

Chord and drag/collector force diagrams are shown in Figure 5-18 for the chord/drag elements along column lines (A) and (C). Note that the maximum diaphragm moment does not occur at the location where the diaphragm shear force is equal to zero. Instead, the maximum moment occurs at the center, and is equal to $w L_x^2/8$. The corresponding chord force, as shown in the figure, is equal to the diaphragm moment divided by the 50 foot diaphragm depth. The drag/collector force diagram, shown at the bottom of Figure 5-18, was calculated as shown in Figure 5-13, using a force of 8 kips which is distributed from a 100 foot wide diaphragm into a 40 foot long wall.

Where perpendicular elements are located at the perimeter of the diaphragm, calculated drag forces must be added to the chord forces resulting from diaphragm moments. Thus, the maximum force in a combined chord/collector member does not necessarily occur at the point of maximum moment. In the example, the maximum of 25 kips occurred at the center, but the net chord/drag force at two adjacent points was very close ($21 \text{ k} + 2.4 \text{ k} = 23.4 \text{ k}$).

Also shown in Figure 5-18 is a diagram for moments in the perpendicular direction. Moments in this direction are zero because the shear forces in this direction are balanced by the internal torsional moments generated the loads and reactions in the other direction.

5.3.4 NON-ORTHOGONAL SUPPORTING ELEMENTS

In structures which contain only orthogonal elements, all frames or shearwalls are oriented either parallel or perpendicular to the direction of applied forces. In such structures, only elements which are parallel to the applied forces experience reactions. Non-orthogonal elements occur when frames or shearwalls are oriented at non-orthogonal, or "skew," angles to the applied forces. In structures which one or more non-orthogonal elements are present, reactions occur in all resisting elements, irrespective of whether they are parallel or perpendicular to the applied forces. Thus, the reactions exhibit *coupled behavior*. This

coupling is caused by vector force components which are exerted by the non-orthogonal elements as they resist force.

The behavior of non-orthogonal systems is often quite complex and difficult to predict without the use of a computer. As with orthogonal systems, flexible and rigid diaphragm assumptions can result in considerably different reactions in resisting elements if more than a few walls are present. Since standard analysis techniques for flexible diaphragm systems have difficulty in predicting the behavior of diaphragm geometries containing non-orthogonal elements, rigid diaphragm systems are in general more suitable for structures with non-orthogonal geometries than flexible diaphragm systems. Where flexible diaphragm construction is used, lateral-force calculations should be based on conservative assumptions.

Where a rigid (concrete slab) diaphragm system is used, the reaction between the diaphragm and resisting elements is generally assumed as a uniformly distributed force. At non-orthogonal elements, this reaction can be interpreted as two vector-component uniform loads, which is resisted by anchors or hooked reinforcing rods which extend into the diaphragm in each orthogonal direction. In each direction, the reinforcing used to transfer this uniform force must extend far enough into the diaphragm, and be lapped with slab reinforcing bars of adequate capacity, to provide a mechanism of transfer for this force into the general diaphragm system.

Where a flexible diaphragm system is used, or where semi-rigid diaphragm system with beam-type framing is used, the connection between the diaphragm and resisting elements generally consists of some combination of beam, or drag, connections, and direct diaphragm sheathing connections. Figure 5-19 shows a typical arrangement of support framing near a non-orthogonal element (wall). Generally, joists and decking are turned to align with non-orthogonal elements, because 1) the diaphragm attachment (and resulting shear strength) is more reliable, and 2) because the cost of oblique connections is less for joist-to-beam connections than for joist-to-wall connections.

For a diaphragm such as shown in Figure 5-19, two separate force-transfer mechanisms are possible:

Fig. 5-19: Typical Framing at Non-Orthogonal Elements (Plan View)

1. Where the "shear" rigidity of the diaphragm sheathing is sufficiently greater than the "axial" rigidity of the attached beam/chord elements, the reaction between the diaphragm and the walls or frames is a uniform load which is uniformly distributed along the wall's or frame's length. This behavior is the same as that of rigid diaphragms.
2. Where the "axial" rigidity of the attached beam/chord elements is sufficiently greater than the "shear" rigidity of the diaphragm sheathing, the reaction between the diaphragm and the walls or frames occurs as concentrated reactions from the beam/chord elements. As an example of this behavior, if the triangular section of diaphragm sheathing adjacent to the non-orthogonal wall shown in Figure 5-19 were removed and replaced with a skylight, the wall would still be able to resist forces if it were adequately restrained by the "trusswork" of beams connected at each end.

Since the actual distribution of forces between the diaphragm-to-wall and beam-to-wall connections is generally difficult to predict, it is generally advisable to conservatively "envelope" the possible range of behavior when determining the minimum required connection capacities. In this regard, it is particularly important to provide adequate axial-force capacity where drag elements and diaphragm chords connect to non-orthogonal elements.

Another problem which occurs to diaphragms which have non-orthogonal edges or resisting elements is caused by diaphragm chords and drag members which "harp" by pushing or pulling against diaphragm corners, causing out-of-plane reactions against the diaphragm. Earthquake damage has been observed where walls at diaphragm "harping points" have separated from the diaphragm due to this effect, when an adequate connection was not provided to transfer or resist each component of the force.

At a typical "harping point," shown in Figure 5-20, the vector forces components of all attached chord and drag members must be balanced. The resulting forces can be relatively large in magnitude, and typically requires beams or other tension-compression members to be located within the immediate vicinity of the harping point. Adequate connections must be provided between the resisting members to resist both the tension and compression directions of forces.

5.4 METAL DIAPHRAGM SYSTEMS

5.4.1 CONSTRUCTION

Metal deck diaphragms without structural concrete topping are used mainly for roofs. Metal decks are usually supported by structural steel framing, in the form of rolled shapes or open-web steel joists, for gravity loads. A metal deck system is constructed of corrugated light-gauge (16 to 24 gauge) ASTM A446 steel, usually galvanized. Figure 5-21 shows sections of typical metal deck profiles. Decking depths range from 1½" to 7½," however, economic aspects dictate that 1½" and 3" deep decks are normally used. Sometimes two sheets, a ribbed sheet and a flat sheet, are shop-welded together to produce a single composite section, called "cellular deck," with increased vertical load and shear strength properties (although the normal reason for its use is to allow wiring to be routed through the hollow cells).

Vertical load section properties (section modulus and moment of inertia) of deck section profiles are generally calculated and provided by deck manufacturers, in accordance with the recommendations given in the *Specification for the Design of Cold Formed Steel Structural Members* (Reference 5-12).

Metal deck panels are usually installed in 36-inch widths, with lengths varying from 20 to 30 feet. Individual panels are interconnected at the seams by spot-welding or by mechanical connections (button punches or self-tapping screws). Metal deck diaphragms are connected to the supporting members, and to chord and collector/distributor members, most often by welding, in which small "puddles" of weld are melted through the decking and into the supporting members. For deck thicknesses lighter than 22 gauge, 14 gauge weld washers are used when welding the deck to the support. Testing and inspection requirements for these welds are given in AWS D1.3 *Specification for Welding Sheet Steel in Structures*. Other fastening methods, such as self-tapping screws or powder-driven fasteners, are also sometimes used.

Metal deck roofs are often covered with lightweight insulating concrete (perlite or vermiculite) for thermal and acoustical insulation, and for fire rating. Manufacturers provide tabulated shear-strength values for metal deck with insulating concrete topping which are typically slightly greater than the shear strength of the metal decking alone, and substantially less than the shear strength of an equivalent thickness of structural lightweight concrete or concrete.

Metal deck roofs must be sloped for drainage. A minimum slope of 1/8" per foot is acceptable if a built-up roof membrane is used. For exposed metal roofs, 1" per foot is a usual minimum with 2" per foot preferred. Roof slopes are generally established by varying column heights. At locations where slope direction changes cause ridge and valley lines in the decking, continuous structural framing members must be provided under the ridge or valley line to support the deck edges.

5.4.2 DESIGN

Metal deck diaphragms are usually designed as flexible or semi-flexible systems. Flexibility factors, based on the equations given in Reference 5-6, are generally tabulated in manufacturers' literature along with allowable shear values. Flexibility factors and allowable shears depend on the metal gauge thickness, the depth and spacing of deck ribs, the type of inter-sheet connections and the distance between vertical supports.

5.4.3 DETAILING

Chords and Collectors. Chords and collector members in metal diaphragm systems are provided by supporting steel framing members beneath the diaphragm. If steel bar-joists are used for vertical support, it is recommended that rolled sections be used at chord and collector element locations. Although slender top flange of bar-joists can theoretically be designed for additional compressive loads to act as chords and collectors, they are very likely to buckle when loaded by real (vs. code-level) earthquake forces. The use of rolled sections in these locations provides a more ductile diaphragm. Beam-to-beam and beam-to-column connections of all beams used as chord and collector members must be adequate to transfer tension forces across the connections.

Connections to chords and collectors may be made by welding, by self-tapping screws, or by powder-driven fasteners. Where deck flutes run parallel to chords and collectors, connections may be made directly to the lower flute as it contacts the steel member. The top flange of all steel members which are oriented parallel to the deck ribs should be at least one inch wider than the deck rib spacing (i.e., for a 6-inch rib spacing, use a flange width, b_f , of 7 inches) to account for randomness of deck rib placement. Where deck flutes run perpendicular to chords and collectors, consideration of minimum flange width is not necessary.

Diaphragm Openings. In metal deck roofs, small openings are often required for duct, pipe or conduit penetrations. Holes large enough to intersect vertical deck ribs should be reinforced for both vertical and lateral forces. Reinforcing typically consists of steel angles, tubes, or plates which are applied to the top side. The reinforcement members should run at right angles to the direction of the deck and be approximately three times the width of the opening. Normally, openings are cut only after the reinforcing has been welded in place.

If an opening exceeds either the width of the deck sheet or 3 feet, then supplementary framing, attached to the structural framing, should be added around all four sides of the opening. If significant, local chord forces should be calculated as described in Section 5.3.2 and members provided to resist these forces.

Out-of-Plane Forces from Walls. Resistance to out-of-plane anchorage forces from concrete or masonry walls or from precast concrete panels requires special attention in bare metal deck construction. Figure 5-22 shows an example of how support framing is typically arranged beneath metal decking. Sections shown in the figure refer to figures described later in this section.

Some metal decks consist of two sheets of metal; a ribbed sheet plus a flat sheet. Cellular, two-sheet systems are generally adequate to transfer out-of-plane forces directly through ledger angles on all sides, as shown in Figure 5-23.

The design of single-sheet metal decking systems to resist out-of-plane forces, however, requires separate consideration for forces applied parallel and perpendicular to the direction of the deck ribs.

In the direction in which the metal deck rib spacing is parallel to the wall, supporting beams or joists are normally spaced anywhere from 4 to 10 feet apart. Out-of-plane forces should not be assumed to transfer directly from the wall into the deck; corrugated ribs too easily deform and flatten when subjected to direct tensile forces (Figure 5-24). Instead, one of the following two alternative resistance methods should be used:

1. Span the wall horizontally between supporting members and transfer via a tension connection into the supporting roof members. The members then function as local drag struts to transition the tensile force into a shear force within the diaphragm. Where this approach is used, member spacing should not exceed about 8 feet unless the lateral wall deflections are shown to be compatible with the roof system.

Fig. 5-24: Rib Flattening Due to Cross-Rib Forces

2. Where an adequate tension connection of the supporting members is not obtainable, or where the spacing of framing members is excessive, additional secondary "drag strut" elements, consisting of steel angles or straps, can be used to transition the wall forces into the diaphragm via a small "sub-diaphragm" (described in Section 5.2.5). The length of these elements (i.e. depth of the sub-diaphragm) should be adequate to prevent overstressing the diaphragm in shear.

In the direction in which the metal deck rib spacing is perpendicular to the wall (parallel to the direction of forces), supporting beams are oriented parallel to the wall direction, and few or no members are directly connected to the wall. Out-of-plane forces in the direction which induces deck tension are able to directly transfer through the ledger angle into the roof deck. However, out-of-plane forces cannot transfer compression directly into the decking; the decking can buckle too easily under compressive loads. Instead, stiffener elements consisting of steel angles or small beams should be provided to take the out-of-plane loads into the diaphragm (Figure 5-25). Alternatively, the support framing members can be laid out to provide a reduced deck span adjacent to walls which is adequate to resist the compression force.

5.5 CONCRETE-AND-METAL-DECK DIAPHRAGM SYSTEMS

5.5.1 CONSTRUCTION

Metal deck in conjunction with a structural concrete topping is used extensively for floor diaphragms, and occasionally for roof diaphragms. Metal deck panels are usually installed in the same manner as metal decks without topping (Section 5.4). Concrete topping may be regular weight (145 pcf) or structural lightweight (110 pcf) concrete.

Generally, the concrete topping is also used to resist vertical loads as a part of an unshored composite framing system, in which metal studs, welded through the metal deck onto steel supporting beams, develop composite bending behavior with the concrete topping. Topping thicknesses range from a code-prescribed minimum of 2" to a normal maximum of about 4½" over the metal deck ribs.

The concrete topping normally contains only light welded mesh or deformed reinforcing to resist shrinkage and temperature cracking; additional reinforcing can be added, however, to resist vertical loads or seismic forces. Diaphragms with supplemental reinforcing should be designed as described in Section 5.6.

5.5.2 DESIGN

Concrete-and-metal-deck diaphragms are usually designed as rigid or semi-rigid systems. Flexibility factors are generally tabulated in manufacturers' literature along with allowable shears. Flexibility factors and allowable shears depend on the thickness and type of concrete topping, the amount of concrete reinforcing, the metal gauge, the spacing of welds at supports, and the distance between vertical supports. Allowable shears may be increased by treating the concrete topping as a concrete slab diaphragm (Section 5.6) and reinforcing it accordingly, using an effective slab thickness equal to the clear thickness of

concrete above the metal decking. To transfer the higher shears to and from the reinforced diaphragm, shear studs, welded onto the chord and collector/distributor elements, are then required.

5.5.3 DETAILING

Chords and collectors in concrete-and-metal diaphragm systems are provided by steel framing members below the diaphragm or by additional reinforcing within the concrete topping.

Connections to chords and collectors may be made by welding of the deck directly to supports or by shear studs welded to the steel framing. Where shear studs are provided, they are most often welded directly through the deck, if the deck is 16 gauge or thinner.

Deck reinforcement around openings is required as described in Section 5.4.2. Reinforcing plates and angles are normally sized to fit within the depth of concrete, and are then embedded in concrete topping. Reinforcing bars are normally provided around larger openings; these bars can be used to resist local chord forces around the openings.

Shear transfer at diaphragm boundaries is made either through reinforcing bar ties into adjacent concrete elements or by shear transfer through the metal deck into supporting steel framing elements.

Out-of-plane forces from attached concrete or masonry walls are normally resisted by the concrete-and-metal-deck, in tension and compression, by reinforcing bars extending from adjacent walls or frames into the concrete topping, or by transferring the forces from walls or attached cladding to the steel framing directly. Consideration of rib flattening or deck buckling is not necessary, since the structural concrete topping stiffens and reinforces the metal decking.

5.6 CONCRETE SLAB DIAPHRAGM SYSTEMS

5.6.1 CONSTRUCTION

Structural concrete slabs are used for both roof and floor diaphragms, and are commonly found in structures of all-concrete construction. Concrete may be regular weight (145 pcf) or structural lightweight (110 pcf) concrete. Slabs may be as thin as 2" and may be much thicker, depending upon vertical load and deflection requirements. Normally, however, concrete slabs are about 4" to 6" thick.

Slabs may be supported by concrete columns alone (two-way flat-slab systems) or by concrete beams and columns (spanning either one-way or two-ways). Alternatively, slabs may be part of a concrete waffle-slab or joist-slab framing system.

Structural concrete slab diaphragms are most often cast monolithically with supporting concrete beams, boundary and collector members. Concrete slab diaphragms are usually connected to frames and/or shearwalls with reinforcing bar dowels, relying on direct tension and/or shear-friction to transfer lateral forces from the slabs into the frames or walls.

Concrete slab diaphragms are usually designed as rigid or semi-rigid systems.

5.6.2 UBC SEISMIC ZONE 1 AND 2 DESIGN REQUIREMENTS

At least one #5 bar is required around the entire perimeter of each diaphragm, plus not less than one #5 bar around all openings, extending a minimum of 24" beyond the openings.

Shear Strength. The total shear strength of a diaphragm is calculated as:

$$V_u \leq \phi (V_c + V_s)$$

where,

$$V_s = (p f_y) d t$$

$$V_c = (2 \sqrt{f'_c}) d t$$

$$\phi = 0.85$$

f'_c = concrete compressive strength, psi

f_y = reinforcing yield strength, psi

p = the ratio of reinforcing steel area provided to gross concrete area

d = the depth of the diaphragm (see discussion below)

t = thickness

The maximum factored design shear force permitted in any segment of a concrete diaphragm should not exceed:

$$V_u \leq (8 \phi \sqrt{f'_c}) d t$$

Any required shear reinforcement should be considered additive to reinforcement required for vertical loads; that is, the net area of steel provided should be at least the area required due to flexure from vertical loads, plus the area required due to shear from earthquake loads.

The minimum area of reinforcing steel in each direction is equal to 0.0025 times the slab thickness times the diaphragm length. The minimum reinforcing area can be split between reinforcing layers when two curtains of reinforcing are used. The maximum permitted

spacing between bars is 18" or two times the slab thickness, whichever is less. Actual reinforcing required may be greater depending on shear strength (V_s) and vertical load requirements.

Flexural Strength. Chord reinforcement required to resist lateral bending moments must be located at the perimeter of the diaphragm. Although a strain-compatibility approach might suggest that reinforcing which is distributed across the interior of the web might also participate in resisting bending moments, normal practice is to ignore any contributions from these bars and utilize only special chord reinforcing which is located within a band of one to three feet from the edge of the diaphragm. The design tension in the chord reinforcing is calculated as the diaphragm moment divided by the diaphragm depth; many engineers actually divide the moment by a distance of about 90% or 95% of the depth, to account for the fact that the actual distance between the centers of tension and compression, " j d," is somewhat less than the full depth.

For concrete beams, it is desirable for the flexural strength of an reinforced, cracked concrete beam to be greater than that of an uncracked concrete beam. The strength of an reinforced, cracked concrete beam is greater than that of an uncracked concrete beam when the ratio of $A_{\text{steel}}/A_{\text{gross}}$ provided is greater than $200/F_y$. When this ratio is met, bending failures are preceded by visible increases in cracking and deflections, permitting evacuation beneath affected areas. When the amount of reinforcing provided is less than this value, sudden and brittle bending failures can occur. The code requires that either a minimum amount of reinforcing must be provided to meet this requirement, or the amount of steel provided must be increased by 1/3 from the calculated minimum.

For concrete diaphragms, the amount of chord reinforcing required is considerably less than $200/F_y$. However, the one-third increase is does not specifically required by the UBC for diaphragm chord reinforcing. Certain conservative factors are present in concrete diaphragms which reduce the need for this increase. These include: 1) relatively large amounts of reinforcing are generally present near the chord reinforcing to resist vertical, shrinkage and temperature stresses, but which also are able to resist a portion of chord forces, 2) concrete diaphragms generally contain a small amount of shrinkage cracks and therefore do not necessarily behave as uncracked sections, and 3) the bending failure mode of concrete diaphragms is generally not directly hazardous to structure occupants. In spite of these factors, many engineers conservatively choose to provide the increased reinforcement to meet the concrete beam criteria.

5.6.3 SEISMIC ZONE 3 AND 4 DESIGN REQUIREMENTS

For seismic zones 3 and 4, all seismic zone 1 and 2 requirements described in Section 5.6.2 apply, with the following exceptions:

V_c , the shear strength contributed by the concrete, is calculated as follows:

$$V_c = (2\sqrt{f'_c}) dt, \quad \text{where the ratio of span/depth} \geq 2; \text{ (shallow diaphragm)}$$

$$V_c = (3\sqrt{f'_c}) dt, \quad \text{where the ratio of span/depth} \leq 1.5; \text{ (deep diaphragm)}$$

Linear interpolation is used for diaphragm span/depth ratios between these values. Separate concrete shear strength values must be calculated for diaphragm segments caused by openings or other irregularities in the diaphragm, based on the span/depth ratios of each segment.

Also, ϕ , the strength reduction factor, should be taken as:

$\phi = 0.60$, if the nominal shear strength is less than the shear corresponding to development of the nominal flexural strength of the diaphragm (e.g. if shear governs the strength of the diaphragm).

$\phi = 0.85$, if flexure governs the strength of the diaphragm (but $\phi = 0.60$ may conservatively be used at any time).

5.6.4 DETAILING

Additional longitudinal reinforcing should be provided for chord and collector elements. Chord forces in topping slab diaphragms are resisted by reinforcing located in diaphragm boundaries. Reinforcing within the web acts to resist vertical dead and live loads and shrinkage and temperature stresses, and should therefore be neglected in resisting diaphragm chord forces.

Diaphragm boundary elements, similar to those described in Section 3.3.4 for concrete shear walls, are required at diaphragm perimeters when the factored stresses due to diaphragm moments, based on an uncracked M_c/I analysis, exceed $0.2 f'_c$. Design and reinforcing requirements for slab boundary elements are identical to the requirements for boundary elements in concrete shear walls. Concrete beams located at the perimeter of a slab may be used as boundary elements, however, the longitudinal and confinement reinforcing provided must meet boundary element requirements. For example, closely-spaced closed hoop reinforcing must be provided, rather than open stirrup shear reinforcing.

Diaphragm boundary elements must be continuous within the portion of diaphragm where they are required, but they may be discontinued at a point where the factored diaphragm stresses, calculated as above, fall below $0.15 f'_c$. Where diaphragm boundary elements are not required, tension reinforcing required for chord or collector elements, when located within supporting concrete beams, should be considered additive to the normal beam reinforcing provided to resist vertical dead and live loads. Reinforcement splices in these elements should conform to requirements for tension splices.

In diaphragms of great length, lateral deformations of the diaphragm can induce an end rotation of the diaphragm, which may induce a twisting deformation to attached walls. Unless at least a portion of the continuous diaphragm chord reinforcing is used to tie the diaphragm to the attached walls, some cracking or separation of the diaphragm from the walls can then result.

5.7 PRECAST CONCRETE SYSTEMS

5.7.1 TYPES OF CONSTRUCTION

Precast concrete systems are used for both roof and floor diaphragms. Typical precast units, shown in Figure 5-26, consist of precast girders, beams, hollow-core deck units, and other structural shapes, which are manufactured under *plant-controlled conditions*.

Three basic types of precast diaphragm systems are commonly used in construction. Specific concerns and requirements contained in this Section refer to these types by number, as follows:

Type 1. Untopped diaphragms in which precast elements are joined along their edges by grout keys, mechanical connectors, or lateral post-tensioning. Historically, these systems have exhibited somewhat poor seismic performance, which has been attributed to the fact that the connections are generally the "weak-links" in the structure, and because the connections have shown a tendency to deteriorate or fail with age due to internal shrinkage stresses or corrosion. Type 1 systems are not recommended for Seismic Zone 3 or 4 construction.

Vertical irregularities between adjacent precast units in the finished surface cause this system to be unsuitable for most types of finished flooring, unless some form of topping is applied.

- Type 2. Untopped diaphragms in which the individual units are joined by cast-in-place, spanwise pour strips, reinforced by well-anchored transverse steel. Historically, these systems have shown fair performance during earthquakes, if properly reinforced.

Generally, pour strips serve to even out much of the vertical irregularities between precast units, however, remaining irregularities and changes in camber growth with time cause this system to be a somewhat marginal surface for most types of finished flooring.

- Type 3. Topped diaphragms, in which the precast elements are covered by 2½ to 4 inches of reinforced, cast-in-place concrete poured monolithically after the units are erected. In addition, these occasionally employ mechanical connectors between precast elements, either as an aid to alignment during erection or for increased redundancy during extreme overload. Historically, these systems have shown good performance during earthquakes, if properly reinforced.

The use of concrete topping covers vertical irregularities between precast units, creating a surface which is acceptable "as-is" for most flooring materials.

5.7.2 SERVICE LOAD BEHAVIOR OF PRECAST SYSTEMS

The design of structures utilizing precast elements must consider all loading and restraint conditions from initial fabrication to completion of the structure, including form removal, storage, transportation and erection. Both initial and long term deflections due to shrinkage, creep, temperature, elastic deformation, wind and earthquake must be considered, including their effects on interconnected elements.

Prestressed members tend to shorten with age due to creep and shrinkage, and this must be allowed for in the layout of the structure and the design of individual elements. Although a large fraction of shortening and creep contractions occurs in the short period before the precast member is installed, considerable axial shortening and changes in vertical camber can occur for several years after casting.

Member shortening is a function of the length, amount of prestress, and the degree of restraint caused by connections and attached structure elements. Member shortening is greatest in the members longitudinal axis (the direction of prestressing). Significant vertical movements can also occur to members due to growth or relaxation of camber. Transverse shortening tends to be slight if the member is not prestressed in that axis.

In non-seismic areas, creep and shortening of members is often accommodated by using connections which provide either sliding joints, or "soft" connections which permit relative horizontal moments (as well as end rotations) of precast members. For earthquake resistance, however, connections are required to minimize relative movements under earthquake loading. These tend to create "hard" connection points, which tend to prevent relative movements due to volume changes. Where these connections prevent movement, restraint forces can develop in the connections and loss of prestress can occur in the prestressed elements.

Prestressed members located adjacent to long shear walls may experience substantial and damaging shear forces in roof-to-wall connections, as shown in Figure 5-27. The stresses in these connections are greatest at each end of the member, and decrease to zero at the midspan of the member (the point about which shortening occurs).

To prevent this type of damage, the structure geometry must either be arranged to permit movement in the direction of the shortening, as shown in Figure 5-28, or the walls and slabs must be designed to accommodate the forces associated with these movements. Shear walls should be both short and located near the midspan point of precast members, in order to reduce the amount of shortening which occurs across their length. The resulting connections can either be detailed as rigid connections which prevent differential movements from occurring between the beam and the wall, or as elastic, or "soft" connections, which permit a degree of movement to occur without a significant loss of strength. Connections required for transfer of diaphragm shear forces into these walls

should either be designed to resist the full shear capacity of the diaphragm or be designed with at least a 1.5 increase factor against the design loads.

Differential vertical movements can also be a concern for precast members. When installed, adjacent precast beams or planks typically do not have equal amounts of deflection or camber, and often require adjustment to roughly align the diaphragm. The internal forces which result from this adjustment, and from any long-term increase in camber, can be a major component in the forces which occur in the connections between adjacent precast members, or between precast members and an adjacent wall.

Concrete topping functions to reduce axial creep and shortening deformations in precast systems. For floor systems constructed using relatively light precast units, such as hollow-plank flooring systems, the rigidity provided by the topping is about equal to that of the precast units, and the topping is able to generally prevent most shortening from occurring, by developing internal compression stresses in the topping. For larger precast units, such as large beams, shortening is still likely to occur, however.

5.7.3 DESIGN AND DETAILING OF SYSTEMS WITHOUT TOPPING

Precast systems without concrete topping can exhibit behavior ranging all the way from rigid to flexible, depending upon the type and spacing of connections used between precast members.

Type 1 Systems. These systems exhibit variable diaphragm rigidity, from semi-rigid to flexible, which must be evaluated on a case-by-case basis. Where these systems are used, member-to-member and member-to-structure connections should be located uniformly throughout the diaphragm and not concentrated in local areas. All connections should be

designed for at least 2 times the calculated seismic forces. In addition, connections should be designed for any forces caused by vertical loads and for internal forces caused by differential camber or shrinkage which might occur between precast elements.

Type 2 Systems. These systems tend to exhibit rigid to semi-rigid diaphragm behavior. Although the design of these systems is similar to that of concrete diaphragms (Section 5.6), the design should consider the effects of creep and shrinkage in the precast members.

5.7.4 DESIGN AND DETAILING OF SYSTEMS WITH TOPPING

In seismic zones 3 and 4, cast-in-place topping is required to be at least 2½" thick, and must be designed to resist the entire diaphragm seismic force (neglecting any contributions from precast elements).

It is customary that no construction joints are detailed into the topping. Where joints must be used, they should be located away from high-shear areas of the diaphragm. Although reinforced, some shrinkage cracking will occur. To control this, the use of a concrete mix with a relatively low water/cement ratio and having good workability, properly cured, is important. Temperatures should be controlled to the extent possible while the topping is properly cured.

Precast systems with structural concrete topping are usually designed as rigid or semi-rigid diaphragms. Allowable diaphragm shears are calculated based on the concrete topping strength and the strength and amount of shear reinforcing. Examples of connections for shear transfer to walls are shown in Figure 5-29. Reinforcing dowels connecting the diaphragm to walls or frames should be sized to resist the maximum of shear or out-of-plane forces, and should be spaced no more than four feet apart.

Vertical loads to the precast units and topping are distributed on the assumption that the precast units are unshored when the topping is poured, and that composite behavior occurs. That is, the precast units alone resist the entire dead load of their self weight, plus the topping weight. Any remaining dead load, plus all live loads, are resisted by the precast elements and the topping as a composite section. In most areas, therefore, little or no reinforcing may be required in the topping to resist vertical loads; only shrinkage and temperature effects. In cantilever slab areas, however, topping reinforcing may be required for vertical loads.

The design and detailing requirements for reinforcing of chords, boundary elements, and other items are identical to the requirements defined for cast-in-place concrete diaphragms, described in Section 5.6.

5.8 HORIZONTAL BRACING SYSTEMS

5.8.1 CONSTRUCTION

Horizontal bracing systems are most commonly found in steel industrial buildings and structures. Horizontal bracing is typically used:

- to transfer roof forces horizontally in buildings with nonstructural metal or fiberglass roof decks.

- to transfer floor forces horizontally in areas where open bar-type grating is used (since open grating is not a suitable diaphragm material).
- to transfer forces horizontally in open industrial structures where conventional floor or roof systems may not be present.

In addition, horizontal bracing members are also often used as "secondary bracing," to provide lateral support for flexural members and trusses.

Horizontal bracing elements generally consist of steel rods and turnbuckles, for very light loads (permitted in one- or two-story structures only), or of steel angle, double-angle, or tee sections, for moderate loads (or taller structures). For very heavy load resistance, wide-flange sections are also occasionally used.

Diaphragm chords and struts should be rolled sections whenever possible. The use of open-web trusses and bar-joists as major diaphragm components should be avoided.

5.8.2 DESIGN

Horizontal bracing systems can be develop either flexible, semi-rigid, or rigid diaphragm behavior, depending upon the bracing configuration used.

Configuration. Figure 5-30 shows three basic types of horizontal bracing configurations, labeled (a) though (c). In (a), horizontal bracing is continuous over the braced area. This configuration is the most rigid, both flexurally and torsionally, and creates essentially rigid diaphragm behavior. In (b), bracing extends continuously around the perimeter of the structure. This increases the overall torsional rigidity of the system (because the bracing

forms a closed geometry) and allows direct transfer of horizontal wind and seismic forces from perimeter walls directly through the bracing to the vertical resisting system. This creates semi-rigid to flexible diaphragm behavior. Finally, in (c), bracing consists of two horizontal trusses, extending in each direction across the width of the building. This configuration offers some flexural rigidity, but is extremely flexible torsionally, resulting in essentially flexible diaphragm behavior.

For most structure configurations, the bracing arrangement shown in (b) is usually the most economical and appropriate system to use. Where heavy forces or rigid behavior are a concern, configuration (a) might be selected. Configuration (c) may be acceptable for light loads, depending upon the acceptability of resulting deflections, as well as the adequacy of strut elements to transfer compression forces to the bracing system.

When bracing is located within the depth of the framing members, as shown in Figure 5-31(b), a separate brace is required for each "bay" between members. Alternatively, when bracing is located immediately beneath framing members, as shown in Figure 5-31(a), longer, continuous members may be used, which generally is less expensive. However, the deck framing members of configuration (a) are subject to both greater lateral bending spans and laterally unbraced lengths, which can offset the possible savings in bracing cost. Some "beneath the beam" bracing configurations are possible (those with tension-only members or those in which the bracing work-points never intersect at framing members) which do not provide any lateral support to the framing members (i.e. the braced span is equal to the column spacing).

Design Forces. The design of horizontal bracing systems should be based on the seismic forces specified for diaphragm systems, F_{px} . The vertical self-weight of bracing members should also be considered, particularly for long, slender bracing members. Although relatively light, dead load stresses in bracing members frequently are a major stress component, particularly for tension-compression bracing. A convenient alternative is to provide vertical support to bracing members by attaching the bracing to adjacent roof or

floor members, or to use short hanger rods attached to nearby vertical-load members to support the midspan of bracing members.

Code requirements for the design of vertical bracing components (struts and diagonals), which include β -Factors and design rules for chevron and "K" bracing systems, are based on tests of vertical bracing systems only. It is inappropriate to arbitrarily impose these requirements on horizontal bracing systems. In the absence of specific requirements, the following guidelines may be used:

- For diaphragm systems which are highly redundant, such as continuously braced diaphragms, and which do not resist upper-story forces from discontinuous vertical systems, it is probably appropriate to ignore the seismic criteria established for vertical bracing and use non-seismic steel bracing design criteria.
- For non-redundant, minimal bracing configurations, and for systems which must transfer substantial seismic forces from discontinuous vertical elements, the additional factor of safety provided by β -Factors may be appropriate (although design force penalties required for vertical chevron and "K" bracing systems are not appropriate).

Where a horizontal bracing system is required to transfer forces due to a horizontal offset of a vertical lateral force resisting element, the horizontal bracing involved should be designed according to the requirements defined for structures with "Out-of-Plane Offsets" (Type D plan irregularity), as described in Section 2.4.2.

Connections of all horizontal bracing components should be designed for at least 1.5 times the calculated component design force.

5.8.3 DETAILING

Connections in horizontal bracing systems may be bolted, welded, or threaded (in the case of rod bracing). Connections should transfer bracing forces to framing members and other braces as directly as possible, minimizing eccentricities.

Figure 5-32 shows a detail of a poor horizontal bracing connection, which utilizes an "L" shaped plate to avoid a column. The exaggerated geometry of the gusset plate is highly prone to buckling during brace compression, particularly if the plate is also required to support even a moderate dead load from the brace's tributary weight. Alternate details are possible which might avoid or mitigate the configuration shown, however, these details typically would require specially-shaped gusset plates, with considerable welding. The real problem with the detail shown is that the brace is framing into a location which is already crowded by three other members. Thus, the least expensive and preferred alternate is to

orient the bracing so that it avoids the crowded intersection. In Figure 5-30, all of the configurations shown utilize bracing patterns which intentionally avoid connections of diagonals to the beam-column joint.

Figures 5-33 and 5-34 show two basic alternatives for connecting bracing diagonals to beam intersections. In Figure 5-33, the bracing is located at the top flange of the framing members. Similar configurations are possible which would connect to the bottom flange.

In Figure 5-34, the bracing is located within the depth of the beam framing. In this detail, the gusset for the horizontal brace is welded to a girder and to a tab plate which supports the beam. The tab plate which supports the beam is then welded to both

the web and the underside of the top flange of the supporting girder.

Normally, a gusset plate which is located about $3\frac{1}{2}$ " below the "top of steel" of the members will fit between standard bolt spacing, however, girders with very thick flanges may require increased clearance.

Single-angle bracing should be used only with caution. Although theoretically possible, it is impractical to load such members concentrically. Eccentricities in practice are often quite large and should be considered during design. If such members are used, they should be designed to include reduction, "Q" factors for slender compression elements, as specified by the AISC specification (Reference 5-7).

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FOUNDATIONS

A *foundation system* provides support for a structure and its loads and includes the soil or rock of the earth's crust and any special part of the structure which serves to transmit loads into the soil or rock. *Foundation engineering* is concerned with the twin problems of evaluating the ability of the earth to support the structure's loads while minimizing movements, and designing the proper transition member (structural element) to transmit the superstructure loads into the ground. In this chapter, the term foundation is applied in a restricted sense to refer only to the foundation's structural elements, and not to the foundation system as a whole. However, the strength and safety of the foundation structural elements is limited by the capabilities of the soil or rock beneath, and must also be evaluated by considering the foundation system as a whole. The evaluation of the foundation system as a whole is generally done in consultation with a qualified geotechnical engineer.

The intent of this Chapter is to discuss only basic structural design aspects of the foundation system. Therefore, many effects which earthquakes have on soils and soil-foundation systems, such as site amplifications or fault-induced or shaking-induced soil failures such as liquefaction, are not fully described. Basic structural design aspects of foundation systems which are described in this chapter include the overall resistance of the structure against sliding or overturning with respect to the earth, described in Section 6.1, and the seismic design of foundation structural elements, described in Section 6.2.

6.1 SLIDING AND OVERTURNING

When lateral inertia forces are applied against the superstructure, horizontal shear and overturning forces result at the base of the structure. These forces must be resisted by the foundation and soil system in order to prevent lateral sliding or overturning/uplift between either the superstructure and foundation or the foundation and earth. Sliding and overturning/uplift concerns must therefore be addressed at two separate structure locations: 1) at the superstructure to foundation interface, and 2) at the foundation to soil interface. The following paragraphs discuss the role of the foundation system in resisting these loads without allowing excessive movements.

6.1.1 SLIDING RESISTANCE

Foundation systems must be designed for the seismic forces transmitted by the shear walls and frames of the lateral force-resisting system. Horizontal forces between the superstructure and the foundation must be resisted by direct anchorage; lateral sliding at this level should not generally be permitted.

Note: An exception is unanchored storage tanks, where limited uplift from code-level forces is permitted if the attached piping and conduits have adequate flexibility. However, it should be noted that anchored tanks have historically demonstrated greater seismic capacity than unanchored tanks.

Horizontal forces between the foundation and the soil may be resisted by any combination of the following mechanisms:

- ***Friction between the floor slab and the ground.*** Slabs-on-grade which are structurally continuous and doweled into walls or footings can function as diaphragms to transfer forces through friction into the subgrade. Reduced friction will occur if sheets of plastic vapor-barrier material are provided beneath the slab. Also, if the floor slab is structurally isolated from the structure/foundation system, no mechanism of shear transfer from the structure to the slab is present.

Slabs-on-grade are generally isolated from the footings (thus preventing their use as diaphragms) when significant differential settlement is expected between the footings and slab. Isolated slabs are also often used as a measure to prevent shrinkage cracks in the slab caused by foundation restraints. These slabs are usually unreinforced or lightly reinforced.

- ***Friction between bottom of footing and ground.*** This is probably the most reliable mechanism of shear transfer, however, it generally is effective only if significant vertical structure loads are supported by the foundation. Where site soils permit only a low friction value or if vertical loads are not sufficient to prevent sliding, a common design practice is to provide a tie beam between adjacent foundations to distribute the horizontal force between a number of foundations along a line, thereby "accumulating" enough frictional resistance to prevent sliding along that column line.
- ***Passive resistance of earth against vertical surfaces of footings, grade beams, piles, or basement walls.*** This mechanism can be effective, but is very dependent upon soil properties. If the method of construction used requires soil to be backfilled against foundations, the design engineer should be cautioned that the material used for backfill may not be the same quality as the standard site materials. Also, the degree of compaction used in backfilling may cause a wide range in the resulting passive pressures, ranging from loose to densely overconsolidated. Therefore, relatively conservative soil resistance values may be required for foundations which are not placed against undisturbed soils.

6.1.2 OVERTURNING/UPLIFT RESISTANCE

Overturning and uplift resistance are required to ensure moment stability when a structure is subject to horizontal loads. Common terms used in calculating overturning effects are as follows:

- The Overturning Moment (OTM) is defined as the moment caused by the eccentricity of applied horizontal forces above the base of a structure or component. Overturning moments are normally calculated at both the top and bottom-of-foundation elevations.
- The Resisting Moment (RM) is defined as the moment caused by the weight of the structure, times the distance to the point about which overturning occurs (the "toe" of the structure), plus the capacity of any uplift anchors present, times the distance between the anchors and the toe.

For an object supported on a rigid material, both uplift and overturning occur simultaneously, when the OTM exceeds the RM. For an object supported on an elastic material (such as a spread footing on soil), overturning occurs when the OTM exceeds the RM, but uplift occurs at an earlier point, when the OTM equals about 1/2 of the RM.

For most lateral load conditions, the code requires that the RM must be at least 1.5 times the OTM, unless uplift restraint is provided. For seismic force conditions, the UBC does not include this requirement, although many engineers prudently include this margin when anchoring structure components, such as equipment. Also, for shallow, soil-supported foundations, it is generally unrealistic to permit an RM of less than 1.5 times the OTM, since the soil pressure beneath such foundations becomes very large (theoretically equalling infinity when the $OTM = RM$).

Overturning is not usually a problem for the structure as a whole, but uplift and overturning may be a problem for individual footings such as column pads or shearwall strip footings, depending upon the geometry of the structural system used to resist lateral forces.

Where overturning or uplift restraint is required, it must be resisted by anchorage of the superstructure into the foundation, and by the weight of the foundation system and/or resistance of tension piles or caissons.

For structures which are regular (i.e., do not contain vertical or plan irregularities described in Section 2.4), and which are designed using the equivalent static-force method, the UBC permits the overturning moment at the foundation-to-soil elevation to be calculated based on the assumption that $F_t = 0$, where F_t is the portion of the total design seismic force which is applied to the top of tall, slender structures. While the F_t force provides a more realistic

estimate of the story shear forces which can occur in tall, slender structures due to "whipping" modes of vibration, overturning moments calculated using static lateral forces in conjunction with F_t tend to overestimate the equivalent overturning moments calculated using the dynamic analysis method.

However, the omission of F_t is not permitted when calculating the OTM at the top of the foundation for the purpose of determining structure-to-foundation anchorage requirements.

6.1.3 THE USE OF REDUCED FORCES FOR UPLIFT AND SLIDING

Sliding and overturning effects are evaluated using the same reduced, design-basis seismic forces which structure elements are designed to resist. The use of reduced forces to evaluate structure behavior provides a somewhat artificial prediction of actual structure sliding and overturning response. Because actual seismic force levels can be greater than the design lateral forces, it is possible for the actual overturning moment (which is subject to a seismic force-reduction factor in the design process) to exceed the resisting moment (which is based on structure weight; not subject to a force-reduction factor), and, likewise, for the actual base shear to exceed the sliding resistance. Thus, some degree of uplift and sliding not predicted by calculations may occur in real structures.

Because each cycle of seismic force is of very short duration, the actual duration of peak overturning and sliding forces which can occur to a structure during an actual earthquake is relatively short. Within this short duration, the amount of uplift or sliding displacement which can occur in a properly designed structure is quite small, only fractions of an inch. Small movements of this type are not necessarily harmful, and in fact, provide significant damping and energy absorption. Design requirements do not prevent small movements, but require uplift and sliding-restraints when large movements are possible. Large uplift motions can lead to impact damage which is detrimental to the vertical stability of a structure.

Uplift-restraining elements between structure and foundation elements are required whenever uplift is calculated to occur; that is, when the overturning moment caused by the code-defined lateral forces is greater than the weight of the structure, as represented by the resisting moment. Where required, uplift restraints typically consist of added reinforcing in concrete frames or shearwalls, or of anchor bolts for steel frames. If properly detailed, these elements are capable of providing a ductile yield mechanism which is able to limit structural damage in below-ground foundation elements.

The use of defined seismic forces for designing uplift restraints is consistent only if the anchorage provided is ductile. Non-ductile anchorage elements include wedge or expansion-type anchors. These elements should not be used unless a $3(R_w/8)$ factor is used in the determination of uplift forces.

6.2 SEISMIC DESIGN OF FOUNDATIONS

Foundations may be divided into two classes for the purposes of discussion: *shallow* and *deep*. A shallow foundation is one which extends less than about 25% of the structures width beneath the structure; a deep foundation is one which extends considerably deeper. Topics specific to each class of foundation are described in Sections 6.2.1 and 6.2.2, below. The following discusses general aspects of the structural design of foundation components.

Design Forces. Superstructures supported by foundation systems are designed for earthquake effects using the seismic design code force specified in Section 2.1.5. For foundation designs, this force is then treated as a reaction force which is applied to the top-of-foundation level. Normally, this seismic force is used with no additional increase in seismic forces due to foundation self-weight inertial forces.

The weight (and resulting seismic inertia force) of the foundation itself is normally excluded from design seismic force calculations for the following reasons:

- because the foundation weight is normally greater than the weight of any typical floor, the inclusion of the foundation weight into the superstructure design force would significantly increase the superstructure design force. This force when distributed vs. the height of the structure would cause unnecessarily increased design forces at upper levels of structures.
- because inertial forces generated within the foundation system are not highly dominated by direct inertial-force effects but instead are affected to a great degree by soil-structure interaction effects (Section 8.4). Soil-structure interactions generally tend to reduce the internal inertial forces generated during an earthquake.

For some soil or structure configurations, it is possible that some increase in seismic inertial force can occur within the depth of the foundation. When this occurs, the geotechnical engineer normally should account for these effects by providing lateral soil-pressure and pile capacities as *net* values, by deducting the portion of inertial force generated from the gross calculated lateral-force capacity.

Design Margin. Damage resulting from overstresses in the foundation can be difficult to detect and repair. As a result, this damage may only become apparent during the long-term performance of the structure, or during a future earthquake. Although the UBC does not impose greater factors of safety for seismic design of foundations, it is prudent to provide additional margins in the foundation strength where convenient and cost effective.

The cost of most foundation materials, once forms are erected and reinforcing is placed, is relatively low, and it usually not a burden to provide additional margins in foundation thickness, reinforcing, and other strength-related foundation aspects. However, additional margins which are provided in order to reduce soil pressures are not necessarily beneficial, since the factor of safety included in most allowable soil pressure values is already quite large. As a rational approach to accomplishing these two objectives, most foundation components can be designed by either:

1. designing concrete elements using a strength reduction factor, $\phi = 0.50$, or
2. providing ductility in the foundation elements.

Foundation design forces should be based on the same reduced, design-basis seismic forces which superstructure elements are designed to resist.

6.2.1 SHALLOW FOUNDATION SYSTEMS

Shallow foundation systems consist of shallow spread footings, continuous strip footings, and mats. Shallow foundations are generally sized to prevent column uplift, sliding or overturning of the structure, and to limit pressures beneath the foundation to less than the allowable soil pressure which has been determined for the site.

Where a structure is supported by a group of foundations, the differences in soil pressure due to gravity plus live loads beneath adjacent foundations should be kept as small as possible in order to reduce differential settlements beneath the structure. Although some larger foundation sizes may be required beneath a structure to prevent overturning or limit soil pressures from seismic forces, the soil pressures beneath these footings due to gravity plus live loads should still be maintained within the range of other, smaller footings which do not resist seismic forces.

Allowable soil bearing pressures are generally based on the long-term loading behavior of soils. For the short term loads which earthquakes produce, various types of soils react differently. Normal code design requirements permit a 1/3 increase in the allowable soil pressures for transient load conditions such as wind or earthquake. For some types of soils, greater increases may be appropriate. The UBC permits a greater increase factor to be used if an appropriate factor is justified by the geotechnical engineer responsible for the site investigation.

Earthquake vibrations may cause consolidation or liquefaction of loose soils, and the resultant settlement of building foundations may not be uniform. Differential settlements can result in damage to the building structure if such settlements have not been properly accounted for in the design. Where the geotechnical investigation indicates a possibility of excessive seismic settlements, stabilization of the soil prior to construction, or the use of

piles, caissons, or deep caissons bearing on a firm stratum, should be considered. Where significant differential movements between foundations might occur, it is prudent to tie individual foundations together to prevent relative lateral displacements and associated damage from occurring. A properly designed and detailed continuous slab-on-grade is a structurally and cost-effective way to tie individual foundations together.

6.2.2 DEEP FOUNDATION SYSTEMS

Deep foundation systems consist of either vertical and/or battered (sloped) piles or deep caissons (piers). Pile foundations typically consist of a reinforced concrete "cap," which supports the superstructure. The cap is supported by one or more piles, usually 12 to 24 inches in diameter. The piles are generally installed by forcing them into the ground without previous excavation. Pile compression capacity is generally developed by bearing of the tip of the pile against an underlying firm soil stratum and/or by friction of the sides of the pile against the surrounding soil. Pile tension capacity relies on skin friction, and is not always feasible to utilize for resisting seismic forces, depending upon the nature of the soils.

Caisson foundations consist of cylindrical concrete columns, typically 18 to 48 inches in diameter, which are installed by excavating a cylindrical hole, inserting a reinforcing cage, and filling with concrete. Often, a "bell" is formed into the base of the caisson to increase the bearing surface area, thereby increasing both its compressive and tensile load capacity. A bell is not necessary, however, if the caisson bears on hard rock, or if vertical loads are resisted by friction and not end bearing.

Due to their larger diameter, caissons have much greater vertical and lateral load capacity individually than piles. When groups of caissons are required to resist loads, the minimum spacing requirements between caissons is generally correspondingly greater than that of equal-length piles, and the net vertical load capacity over a given area is about the same. However, the lateral resistance of a group of caissons is usually somewhat greater than that of a similar group of piles.

The lateral resistance and load capacity of caissons are generally determined by assuming behavior similar to that of piles and by limiting the maximum lateral displacements to some value (typically 1/4 to 1/2 inch). Lateral forces are generally assumed to be resisted entirely by soil pressures against the side of the caisson, and not by any rotation at the caisson's tip (bottom). Because of their larger tip size, bell caissons which are reasonably shallow can, however, exhibit behavior similar to that of embedded spread footings, and can have inherently greater lateral load resistance and capacity than is generally estimated using the typical pile analysis.

Deep foundation systems are generally analyzed by assuming that the piles or caissons can be designed for vertical loads as well-confined columns, and for lateral loads as beams on an

elastic foundation. These assumptions are somewhat simplified, because they assume that: 1) very little deformations occur within the soils, and 2) that adjacent piles or caissons have little influence on each other. Analysis of the true behavior of the soil-pile system during seismic motions is extremely complex, however. Bending and shearing forces within the piles or caissons result from both the forces induced by the supported structure and from the shaking deformations of the ground. Large lateral forces can be exerted against piles below the ground surface due to earth movements caused by slope failures or areas of liquefaction.

Standard design practice should base the design of deep foundations upon the determination of compression, tension, and bending forces in the piles or caissons due to the inertial forces in the structure above. Piles should be flexible enough to deflect with the motions of the ground, or should have adequate strength or ductility to resist the forces which ground deformations could impose without fracturing. This is particularly important for structures on piles through loose fills and marshland or bay muds, where large ground distortions, liquefaction, or slope failures could occur during an earthquake.

Lateral forces in deep foundation systems can be resisted by passive pressures against vertical surfaces of pile caps and/or grade beams, or against the sides of piles and caissons. If the piles or caissons are used to resist lateral loads, the size and number of piles or caissons required may be governed by the magnitude of lateral forces, rather than by vertical loads.

Lateral bearing forces against caissons or piles cause internal shear forces and bending moments, which must be resisted by providing reinforcing. Where the upper soils have poor lateral bearing strengths, the resulting bending moments are correspondingly higher, since the effective length of bending is increased. The lateral capacities of piles and caissons are dependent on the properties of the soil; the size, length, and material of the pile; and the grouping and spacing of the piles or caissons. Where lateral bending of the pile can occur, bending reinforcement must be provided in the upper part of the pile, extending at least past its effective "depth of fixity." The distance from the top of the pile to the point where the pile shear is equal to zero when a lateral force is applied to the top of the pile is generally defined as the "depth to fixity." In some soil conditions, the pile depth to fixity can increase during repeated load cycles (such as earthquake forces) due to deformation of the ground away from the pile (i.e., enlargement of the hole).

The UBC requires pile caps and caissons to be interconnected with tie beams capable of a resisting tension/compression force of at least 10% of the larger factored vertical load combination occurring from columns at either end of the tie beam unless it can be demonstrated that the soil-pile system can adequately resist these forces without excessive lateral deflection. Particularly in soft ground, piles and caissons must be tied together to act

as a unit. If a slab-on-grade system is provided between the foundations, the design of the tie beam should consider the following:

- If the slab-on-grade is a continuous system (unjointed) which is attached to the foundations via reinforcing dowels, the slab itself can be used as a tie beam. That is, reinforcing elements can be provided within slab to tie the foundations together.
- If the slab-on-grade is discontinuous, and could experience settlements which are different from the foundation system, the tie beam system should be cushioned or separated from the slab sufficiently so that slab settlement will not damage the tie beams.

Where very poor surface soils occur, it is frequently economical to drive battered, or raked, piles which are able to resist horizontal forces through their axial compressive and tensile capacities. Battered piles are similar to structural bracing. Battered pile systems are not very ductile, and should therefore be designed using a lateral force at least equal to $3(R_w/8)$ times the structure lateral forces. In addition, the design of battered pile systems should consider the effects of any bending moments which could occur in the piles and pile cap.

When determining the configuration of vertical and battered piles within a pile cap, the design engineer should be aware that eccentricities between battered and vertical pile positions can lead to substantial bending moments in the piles and pile cap. Also, the designer should be aware that the connection detail between the pile and pile cap can significantly affect the induced moments and shears and the lateral stiffness of the pile and pile cap system. Pile and pile cap connections are generally assumed as either fixed or pinned when the foundation is designed. The details and construction used for this connection must be consistent with the assumed fixity condition.

The effects of eccentricity in the layout of piling are demonstrated in Figures 6-1 through 6-3. In the figures, the effects of a lateral force "V" are shown for three pile cap configurations. Pile tension and compression forces due to the lateral force are indicated with letters "T" and "C," respectively. In all of the pile caps, the axial stiffness of the battered piles is assumed to be much greater than the lateral bending stiffness, so that all of the lateral force "V" is resisted by the two battered piles.

In the pile cap shown in Figure 6-1, vertical piles are installed concentrically with the battered piles by offsetting the piles laterally. In this configuration, the adjacent vertical pile is able to directly resist the vertical component of force from the battered pile. Thus, the pile cap is a concentric system, and little bending occurs in the pile cap.

It is interesting to note that the direction of rotation of a pile cap with battered piles can be opposite to that normally associated with overturning moments. In the pile cap shown in Figure 6-2, vertical piles are offset from the battered piles. In this system, a "center of rotation" is formed at a distance " H " above the cap where the lines of force from the battered piles intersect. The moment of V times H is resisted by the two vertical piles by developing axial forces equal in magnitude to VH/d . Depending upon the ratio of H/d , this vertical force couple can be quite large. As a result, the axial elongation of the piles can be large, resulting in lateral displacements of the pile cap as shown by the dashed line. Lateral foundation displacements can cause a significant portion of the lateral forces to be resisted by lateral bending in the piles. Therefore, pile bending moments should always be included where significant lateral pile displacements may occur.

The effect of this eccentricity is emphasized by the pile cap shown in Figure 6-3, in which the pile configuration provided is unable to resist lateral forces by axial forces alone. Because the axial-force vectors from all of the piles intersect at the same point (the center of rotation), the pile configuration is unable to resist lateral forces using truss-like behavior. All

of the lateral forces must therefore be resisted by the bending resistance of the vertical and battered piles, which results in the displacements shown by the dashed line.

Where fixed-headed piles are used to transfer bending moments between a pile and foundation, both pile axial forces and the transferred bending moment must be considered when evaluating the punching shear strength of the foundation or pile cap. This potential failure mode is similar to the concern described for concrete two-way slab moment-frame systems described in Section 4.2.2. An example of a punching shear failure mode of a foundation due to extreme bending moments from raked piles is shown in Figure 6-4.

Figure 6-4 also shows a potential failure mode which can occur when lateral forces in conjunction with liquefaction occurs beneath a battered-pile supported structure. Slender piles require nearly continuous lateral support from the surrounding soil to prevent compressive buckling from occurring to the pile within the ground. Where liquefaction of a zone of soil could occur, piles within the zone should have a diameter sufficient to permit the piles to span across the liquefaction zone without lateral support.

An additional hazard to foundations where soft soils or a liquefaction potential exist is that of lateral movement or sliding of the ground. Lateral soil movement is capable of exerting large lateral forces against piles or caissons. Most soft or liquefiable soils tend to occur at relatively level sites, where the potential for lateral movements is greatly reduced, however, fill materials are frequently placed to raise portions of a site (for example, for bridge abutments), and lateral movement in or beneath these soils is possible.

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PARTITIONS AND CLADDING

This chapter discusses the general behavior of non-structural partitions and cladding systems. An overview of system behavior and UBC design requirements is presented in Section 7.1. Specific partition system design considerations is presented in Section 7.2, for both masonry and metal stud construction. Design considerations for cladding systems are discussed in Section 7.3, for architectural precast concrete construction, and Section 7.4, for metal wall cladding construction.

7.1 INTRODUCTION

In typical buildings, all or most of the interior partitions and exterior cladding elements are considered to be non-structural elements. A non-structural element is one which could theoretically be removed from the structure without significantly affecting the structural capacity of the building, or weakening the lateral-force resisting system. Non-structural elements are not intended or designed to resist floor or roof gravity loads. Under earthquake loading, non-structural elements should not inadvertently participate in the lateral-force resisting system.

7.1.1 PARTITION AND CLADDING BEHAVIOR

Non-structural partitions and cladding elements should either be designed to be isolated from the primary structure, or be flexible enough to accommodate the lateral displacements of the structure during an earthquake. At the same time, the wall elements must resist the inertial loads generated by their own mass. Proper design and detailing to achieve these two goals is crucial to the satisfactory performance of these building elements. In this chapter, the term "primary structure" refers to the building walls, columns, beams, diaphragms, collectors, and other load-carrying elements that are part of the main vertical and/or lateral force resisting system. The term also includes any secondary members, framing, or brackets which provide support or bracing to cladding or partition elements from the structure.

The lateral displacements that a structure experiences when subject to seismic forces are the primary source of earthquake damage to non-structural partitions and cladding. The magnitude of these displacements varies, depending upon a host of factors including the nature of the lateral force resisting system and the intensity of the ground motion. Selection of appropriate connection details depends largely upon the magnitude of lateral building displacements anticipated. As discussed in Chapter 2, building codes limit the amount of permitted lateral story-to-story displacement, or story drift, under design lateral loads. The

story drift is limited both to ensure overall structural stability and to control the damage suffered by other building elements, such as non-structural walls.

Even when design story drifts are within the code prescribed limits, care must be taken to ensure that non-structural wall and cladding elements, which are often rigid and brittle, are not excessively damaged by building displacements.

7.1.2 DESIGN CONSIDERATIONS

The UBC specifies minimum design forces and story drift tolerances for non-structural exterior walls and cladding systems. The code requires that:

1. Joints and connections for non-structural cladding elements must be designed to accommodate a drift equal to the greater of:
 - the calculated story drift multiplied by $3(R_w/8)$, or
 - 1/2 inch per story.
2. The non-structural walls and cladding systems must be designed to resist lateral inertial forces due to their own mass.

For exterior wall elements, the connections to permit movement in the plane of the wall must be designed to accommodate the lateral building movement: through sliding using connectors in oversize or slotted holes; by bending of steel; or by other methods which provide a ductile mechanism for tolerating the story drift. The UBC does not directly require that story drift effects be considered for interior non-structural partitions, however, experience in past earthquakes has shown that where rigid elements such as partition walls have not been properly isolated from the primary lateral system, they have suffered severe damage.

In addition to consideration of story drift, non-structural partition walls (as well as cladding elements) must be designed to resist inertial forces generated by self-weight. Attachment of the wall elements to the primary structure must be made by direct, mechanical connections. The design force levels for wall elements are generally higher than the building design base shears, to account for higher accelerations experienced in the upper levels of multi-story structures, and to account for the lack of redundancy and/or energy absorption properties in the wall elements and their connections.

Both non-structural partition and curtain wall systems are generally designed using the UBC equivalent force approach specified for non-structural building components. The UBC requires minimum seismic design forces for non-structural components to be determined using the following equation:

$$F_p = Z I C_p W_p$$

Where:

F_p = Design Lateral Force

Z = Seismic Zone Factor (for DOE facilities, see UCRL-15910, Table 4-4)

I = Importance Factor

C_p = Lateral Force Coefficient

W_p = Weight of the Non-structural Component

For the design of exterior cladding element connections, the value of I need not exceed 1.0 for the entire connection. The C_p coefficient provided by the UBC for walls and cladding elements is 0.75, however, special values are specified for certain connection design and cantilevered element conditions. For example, for precast cladding connections, the UBC requires increased design forces, described in Section 7.3.5, to be used.

In addition, interior walls and partitions must be designed for a minimum lateral force of 5 pounds per square foot, applied perpendicular to the walls. Masonry partition walls must be anchored to both the floors and roofs that provide lateral support for the wall with positive, direct connections. These connections must be capable of resisting an out-of-plane lateral force of 200 pounds per lineal foot of wall, or the computed design force, whichever is greater.

7.2 PARTITION SYSTEMS

The primary goal when designing non-structural partition systems for earthquake effects is to simultaneously: 1) provide adequate anchorage to resist both in- and out-of-plane lateral forces, and 2) isolate the walls from the main structure, thus minimizing damage to the partitions resulting from structure displacements. The design approach chosen for the partition system depends upon both the chosen materials of partition construction and the magnitude of structural deflections anticipated. Common materials of non-structural partition construction include:

- Stud wall construction, using either wood or metal studs which are sheathed with gypsum wall board, plywood, or plaster.
- Masonry, either unreinforced (not suitable for seismic design) or reinforced.

Partitions which use stud wall construction possess a number of properties which enhance their seismic performance, including light weight and the inherent ductility of the wood or metal studs. Partitions which use masonry construction, with their greater weight and lower inherent ductility, require more careful detailing to achieve the desired level of performance.

Irrespective of the materials used, there are, in general, two separate types of partitions which are found in structures:

- **Full-Height Partitions.** Partitions which are required to extend for the full floor-to-floor or floor-to-roof height. This type of partition is usually used where fire-resistive barriers are required, such as around stairwells or mechanical/electrical rooms, or where security barriers are required.

Where this type of partition is anchored to resist lateral forces at both its top and bottom edges, it is not normally vulnerable to damage from lateral inertial forces. However, it is very vulnerable to damage from imposed structure lateral deflections, unless special construction is used to accommodate structure deflections.

- **Partial-Height Partitions.** Partitions which are only required to extend from floor to ceiling (or to some fixed distance above the floor). This type of partition is used to create rooms or offices.

Because its solid wall sheathing does not extend to upper floors, partial-height partitions are not normally vulnerable to damage from imposed structure lateral deflections. However, if lateral bracing elements are not provided to structure elements located above the partition (i.e. if it is required to cantilever from the floor or if it is constructed as a free-standing structural system) this type of partition may be vulnerable to damage from inertial forces. Generally, it is preferable to provide diagonal braces at the top of the systems to support the partition against out-of-plane forces, but which are flexible enough to isolate the partition from structure lateral deflections.

Partition walls are generally designed to span from floor to floor for out-of-plane loading, and to act as cantilevered wall elements for inertial loads which act in the plane of the wall. Critical considerations in the design of partition walls include:

- The connection between the top of the partition wall to the floor or roof above. This connection must provide sufficient lateral support for out-of-plane loading, while isolating the wall from the effects of lateral drift of the primary structure.
- Connection of the bottom of the wall and the floor, which must provide adequate shear resistance, and an adequate chord connection to resist overturning moments from loads parallel to the wall.

- The partition wall must have sufficient strength and rigidity to sustain out-of-plane loading without excessive deflections.
- For infill partitions, the vertical joints between the partition walls and the structure columns must be of sufficient width to allow the structure to laterally displace without distorting the infill wall.
- For full-height partitions, allowances must be made for duct and utility penetrations through the walls. These penetrations can be of substantial size, and must be accounted for in the design.

Partitions may be used to support wall-supported equipment or shelving, or to provide lateral restraint for the top of floor-mounted equipment items. Where significant, the effects of such equipment should be considered when determining the design lateral load for the partition.

Where partitions intersect building columns or other vertical elements, it is necessary to maintain a gap between the partition and the vertical element which is sufficient to allow the building to distort laterally without loading the partition. In many buildings of steel construction, this can be done in a concealed manner where the steel columns, braces, etc. are already enclosed with metal studs and wallboard. Where concrete columns occur, a separation gap can be provided which is sealed with a flexible joint material, flashing can be provided to cover a gap, or a recessed pocket can be formed into the column.

Failure to provide adequate space between any rigid infill walls and the structural system allows the beams and columns of the structure to contact the infill wall when the structural frame displaces laterally (Figure 7-1). The infill wall, which has high in-plane lateral rigidity, tends to act as a diagonal compression "strut" across the infill bay, which stiffens the structure and draws lateral forces away from the normal lateral-load resisting elements, at least until the stiffness of the infill wall has been degraded.

Where partitions are subjected to in-plane loading from structure displacements, damage may result to the partitions, the structural frame, or to both. Where relatively rigid (masonry or concrete) infill partitions do not extend for the full column height (partial-height partitions), if not adequately isolated the partitions tend to reduce the effective bending length of the

Fig 7-1: Effects of story drifts on infill walls

column. This results in the following effects:

- The partition tends to stiffen the column relative to the stiffness of other lateral-force resisting elements in the building. This increases the lateral force which the column is subjected to.
- By reducing the effective length of the column, the partition tends to increase the total bending moment capacity of the column, without causing a corresponding increase to the shear strength capacity.

As a result of these effects, the short "trapped" section of column located above the partition may become vulnerable to a brittle shear failure, as shown in Figure 7-2.

The connection of the head of a non-structural wall to the upper support (floor or roof) must be detailed to allow the building stories to displace relative to one another without imposing excessive stresses in the partition. Isolating non-structural walls from the primary structure can be accomplished by providing a gap equal to $3R_w/8$ times the computed story drift between load-bearing columns and the non-structural partition walls. A wall head connection detail, that allows the beam, floor, or roof structure to "slide" along the top of the wall must also be provided.

If the partition is built beneath a beam, and vertical deformation of the beam under lateral load is expected, a gap between the top of the partition and the bottom of the beam should be provided which is sufficient to allow for elastic deformations due to live load, plus deformations due to lateral load multiplied by $3R_w/8$. For long beam spans it is also prudent to provide an additional gap associated with deflections which could occur due to vertical seismic ground motions. For partition systems that are not freestanding (i.e., cantilevered from the floor), adequate lateral out-of-plane support must be provided at the top of the wall to provide for out-of-plane seismic loads. This support needs to be provided over the full range of vertical displacements that the primary structure may experience (i.e., the support should not bind or fall free at the maximum possible vertical extension). Note that if a beam-column joint of the load-carrying frame has a nominal moment resistance, then some beam deformations due to lateral load should be expected, even if the beam is not part of the designated lateral-force resisting system.

Fig 7-2: Short-Column Failure at Partition

Source: EQE Files, Miyagi-Ken-Oki, 1978 (M7.4)

7.2.1 STUD-FRAMED CONSTRUCTION

Stud-framed partition systems are constructed of wood or light gage metal studs sheathed with gypsum wall board, plywood, or plaster. The metal studs are typically cold formed steel members, with depths varying from 1-5/8 inch to 8 inches. Studs are "C" shaped and may or may not have a return on the flange ends. The metal studs used usually vary from 25 to 14 gauge in thickness, and are generally spaced at between 12 to 24 inches on center. At the base of the wall, the studs are set in a cold formed track (also referred to as a sill or runner), which is a channel section with a clear width between flanges equal to the stud depth. The track is anchored to the floor using bolts, expansion anchors, or shot pins (powder-driven fasteners), and the studs are then nested in the track and screwed, crimped, or welded in place.

There are a number of different approaches which can be taken for making the top, or "head," connection of a stud partition. In addition to structural requirements, the choice of a detail depends heavily upon architectural and fire/life safety requirements. The amount of space available between ceiling and primary structure, and the size and distribution of ducts, pipes, and other mechanical systems above the ceiling line will also influence the selection of the wall head detail.

Where the stud wall sheathing need not extend above the ceiling, the studs often terminate in a top track, or "runner," which in turn is braced diagonally to the main structure above. An example of such lateral bracing is shown in Figure 7-3. Alternatively, some or all of the studs can continue up to the underside of the floor or roof above, and the wall sheathing can be terminated just above the ceiling. These types of details can generally provide sufficient flexibility to allow story drifts parallel and perpendicular to the partition to be tolerated with minimal damage due to either story drifts or inertial forces.

Where the stud wall framing extends the full story height, provision must be made at the wall head connection for vertical floor movements due to live loads, as well as relative floor displacement due lateral story drift.

Figure 7-4(a) shows a typical head connection that utilizes a special "flex" track. Flex track

accommodates vertical displacement of the primary structure through bending of the angled portions of the track legs. No provision is made in this detail to allow for lateral story drift parallel to the partition. This detail is not recommended for use in seismic areas, except in cases where the sheathing on both sides of the partition terminates just above the ceiling line, allowing story drift parallel to the wall to be accommodated through weak axis bending of the wall studs. This detail is also acceptable for use in structures where the design story drift ratio multiplied by $3R_w/8$ is less than 0.0025.

Figure 7-4(b) shows a wall head detail that utilizes nested tracks. The nested tracks slip to accommodate story drift parallel to the partition. The fixed track is connected directly to the structural slab/deck overhead with shot pins, expansion anchors, bolts, or welding. There is no mechanical connection between the fixed track and the sliding track. Therefore, the overlap provided should be sufficient to accommodate deflections in each direction. The stud-to-track connections, and the connections for the wall sheathing are all made to the sliding track.

Where partition walls intersect, details similar to those shown in Figure 7-5 may be provided to prevent interaction between the wall segments that could bind or otherwise inhibit the sliding action of the head detail.

For loads perpendicular to the partition wall, the following must be considered:

- Flexural capacity of the wall.
- Anchorage of the top and bottom runners to the main structure. Where the distance between anchors is large, bending of the tracks between anchor points should be considered in the design.
- For heavily loaded walls using the nested track head detail, local bending of the leg of the fixed track should be considered in the design.

For most common stud wall construction to UBC standards, the 5 pound per square foot minimum lateral load will govern the design of the wall for lateral loads. Under this load, the out-of-plane deflection of the partition is limited to $1/240$ of the wall span for partitions with brittle finishes (such as plaster) and $1/120$ of the span for walls with flexible finishes (such as gypsum wallboard). The presence of wall mounted equipment, or floor mounted equipment

which relies upon the stud wall for out-of-plane support, should be accounted for when determining the design lateral load. To control damage to the partition and finish, the deflection limitations outlined above for different wall finishes should be adhered to when the partition supports equipment loads.

Fig 7-5: Stud crosswall head detail

7.2.2 MASONRY PARTITIONS

Masonry partition systems must be carefully detailed for good seismic performance. The basic design and detail requirements are similar to those for stud-wall partitions, but special consideration must be made in the design for the greater weight and lower overall ductility of masonry partition walls. In addition, full height masonry partitions are extremely rigid in plane, and if not isolated from the primary structure, can significantly alter the overall dynamic response of the building. In past earthquakes, improperly isolated masonry partitions have contributed to increased building damage and collapse, by heightening torsional response, creating discontinuities in lateral stiffness from story to story, and causing abrupt transfers of lateral force when the partition elements failed.

Unreinforced Masonry Partitions. Although unreinforced concrete masonry unit (URM) partitions are commonly used in many areas of the country, unreinforced masonry partitions are not suitable for use in areas of moderate or high seismic hazard. Because URM lacks inherent ductility and tends to fail in a catastrophic manner, the primary seismic concern is that the walls will become falling hazards, or, because of their common use as fire walls surrounding exit paths, may become impediments to safe and rapid egress. Collapse of the walls can be caused by out-of-plane failure, either in bending or due to inadequate top

support, or by brittle in-plane shattering, brought about by shear forces induced by structural deformation. Experience in many earthquakes has shown that unreinforced masonry partitions are a hazard to life safety.

Reinforced Masonry Partitions. The design of partially or fully grouted and reinforced masonry non-bearing partition walls is generally identical to the procedures outlined in Section 3.2 for structural masonry walls. The major differences are in the magnitudes of the seismic design forces, and in the connection details used at the boundaries between the partition walls and the primary structure. Non-bearing partition walls are subject to the same minimum reinforcement requirements as load bearing walls. Special design considerations for non-structural masonry partitions include:

- Non-structural walls need only be designed for inertial loads due to their own mass, plus any wall mounted equipment.
- Non-bearing walls must be isolated from the primary structure, as discussed in Section 7.2.
- Direct positive connections are required between the partitions and supporting floor and roof elements. However, these connections must be detailed to isolate the partition walls from story drift.
- When non-structural partitions using hollow-cell masonry units are constructed after the floor and/or roof structures are complete, it is necessary to use either open-end or broken (face shell only) masonry units at the reinforced cells to work around the vertical reinforcement.

Depending upon wall configuration and layout, there are several design approaches available for lateral support of non-structural masonry partitions. Walls can be designed to span floor to floor for out-of-plane loads, and to act as cantilevers for in plane loads.

Figure 7-6 illustrates some typical masonry partition base details. Connections at the base of the masonry partition are generally made with reinforcing steel dowels extending into the concrete deck. The "tail" of the dowel which is embedded in

Fig 7-6: Masonry Partition Base Details

the slab, and the lap splice of the dowel with the partition vertical reinforcement, should follow typical detailing practice for reinforced masonry construction. If dowels in the slab are not provided, reinforcing dowels can be epoxied into drilled holes or sleeve-type expansion anchors can be used to provide a positive mechanical connection for the base of the partition.

As with stud framed partitions, the head connection of masonry partition walls will vary, depending upon how far the partition must extend above the finish ceiling.

Full-Height Systems. Figure 7-7 illustrates two approaches for providing sliding or ductile connections at the head of full-story masonry partitions. If continuous lateral support for the wall is not provided and the spacing between the lateral support points is greater than 4 feet, the bond beam at the head of the partition must be designed to span between the anchor points for out-of-plane lateral loads.

Partial-Height Systems. For walls that need not extend for the full story height, a number of lateral bracing methods are possible. The most common, and generally the preferred, method is overhead bracing, shown in Figure 7-8.

Fig 7-9: Walls/Partition Intersection Details (Plan View)

Overhead bracing attaches to the top of the wall and extends diagonally to the floor or roof framing above. The bracing used should have some degree of lateral flexibility, so that structure deflections parallel to the wall do not transmit forces into the system. Where vertical deflections due to dead, live and seismic forces could occur, a slotted hole or some similar provision should be made to prevent vertical-loading of the wall.

Self-Standing System. If there are cross walls present in the partition layout, it may be possible to design the partition walls to span horizontally between the cross walls. The advantage of this approach is that the top of the walls need not be mechanically connected to the primary structure above. The partitions can therefore be completely isolated from the direct effects of story drift. The distance the partition can span horizontally between crosswalls limits the application of this design approach. Horizontal girts at the heads of the walls can be used to increase the distance the partition can span between crosswalls. It may also be possible to design the partitions to span floor-to-floor in one direction, and to span between cross-walls in the orthogonal direction.

Where masonry partition walls spanning floor to floor intersect and connections for out-of-plane loading at the heads of the partitions are provided for nonparallel wall segments, the nonparallel walls must be isolated from one another at the junction (Figure 7-9). This will prevent accidental in-plane restraint of the walls by out-of-plane anchors for the nonparallel wall segments.

7.3 PRECAST CONCRETE CLADDING SYSTEMS

This section discusses seismic design and detailing of architectural precast concrete cladding (curtain wall) systems.

7.3.1 INTRODUCTION

Architectural precast concrete cladding systems are often selected for building exteriors where durability, fire resistance, and sound transmission characteristics are important to the function of the structure. Cladding panels are typically 4 or more inches thick, constructed of either standard or lightweight concrete and reinforced with mild steel bars or wire mesh. Although prestressing of cladding panels is done, it generally is only found in the largest units, and then chiefly for crack control. Standard practice for design and detailing of precast concrete varies considerably from region to region. In this chapter, only non-structural systems are presented.

In areas outside seismic zone 4, precast cladding units tend to be large, often several stories in height. Although designed as non-structural elements, the panel connections are frequently rigid enough to allow for the transfer of some in-plane lateral seismic forces from the primary structure through the cladding. Because out-of-plane wind loads govern the design of the cladding units, typical bearing and lateral connections often lack any engineered capacity for resisting forces in-plane (parallel to the panel face).

For good seismic performance, neither of these conditions is desirable. During seismic motions, a substantial amount of inelastic behavior can occur, and story drifts are of much larger magnitude than the drifts which occur due to wind forces, which are in the elastic-range (even when the magnitude of the "design" seismic forces is less than the "design" wind forces). Seismic inertial forces can occur in both the in-plane and out-of-plane directions.

In seismic zone 4, the design of precast concrete cladding has evolved around seismic design considerations. Precast concrete curtain walls tend to consist of a greater number of smaller pieces, and strong emphasis is given to designing connection details to ensure that the precast panels are decoupled from the primary structure to as great an extent as possible. Connection details are designed for lateral forces which can be applied in any direction, in-plane or out-of-plane.

Because the UBC limitation for structure story drift is the same for all seismic zones, the actual drift which buildings with precast cladding systems experience is typically about the same for structures in Seismic Zones 1, 2, 3, or 4, irrespective of design forces. For this reason, many design practices for cladding joints, connection details, and anchor layouts developed for Seismic Zone 4 are appropriate for all regions where seismically resistant cladding systems are desired.

7.3.2 PRECAST PANEL CONFIGURATION

Non-structural precast concrete cladding panels typically fall into one of several categories (Figure 7-10). Among the most common types are spandrel panels, which are long horizontal elements that generally span from column to column at the elevation of the floor or roof planes; column cover units, slender vertical elements, which may be several stories tall; and window panels, which typically span from floor to floor, and are penetrated by an opening for a window. In areas of low seismicity, multi-story window panels are common. Solid wall panels, one or two stories in height are also common. Precast prestressed concrete double tees are sometimes also used as solid wall panels for low-rise construction.

Some panel arrangements commonly used outside seismic zone 4 are difficult to adapt to seismic design criteria. Among the practices which pose the greatest difficulty are those that involve stacking of panels and multi-story wall panels. Two typical arrangements for stacked, multi-story panels are shown in Figure 7-11. In the section shown at the left, a portion of the weight of the cladding is supported via corbels by the structure. In the detail shown at the right, all the weight of each panel is supported by the panel beneath, and is accumulated at a continuous foundation located beneath the panels; thus, the structure provides only lateral support to the panels.

Panel configurations in which the vertical support for the panels is provided by the panels below, while tie-back connections are provided to resist out-of-plane loads, are vulnerable to progressive and catastrophic failures, since a failure of a single tie-back connection a lower level could result in a progressive collapse of wall panels for the full height of the structure. In addition, the typical panel-to-panel connections used between stacked units are often relatively rigid, which causes the stack of panels to behave as a rigid body when laterally loaded in the plane of the cladding units. This results in increased deformation demands in the lateral tie back anchors to accommodate building story drift at the upper floors.

Where multi-story or stacked cladding units are used, but detailed to attempt to accommodate story drifts, each unit or stack of panels must be designed to resist in-plane inertial forces parallel to the wall by connections at a single story level. If these connections are not located near the center of mass of the panel (or in the case of stacked panels, are not located at grade), relatively high overturning moments due to panel inertial loads are generated. The difficulty in designing the panel connections and supporting structure to tolerate these loads without distress or excessive deflection has historically outweighed the erection advantages of stacked or multi-story wall panel systems in high-seismic zones, leading to the development of the smaller, suspended panel systems described herein.

Fig. 7-10: Typical Precast Cladding Unit Arrangements

7.3.3 ACCOMMODATION OF STORY DRIFT

As discussed in the Section 7.1, the ability to accommodate story drift is the single most important factor controlling the seismic performance of exterior cladding systems. Cladding panels can experience stresses due to story drift both in-plane and out-of-plane to the panel. In general, two types of support mechanisms, shown in Figure 7-12, are used to enable cladding panels to accommodate in-plane story drift. The mechanism most commonly used is the sway mechanism, shown at the right side of the figure. In the sway mechanism, the connections along either the top or bottom edge of the panel are fixed, and the connections at the opposing edge are designed to accommodate the story drift by bending of rods or plates or through the use of bolted connections in horizontally slotted holes. The other mechanism used to accommodate in-plane story drift is the rocking mechanism, shown on the left of Figure 7-12. The rocking mechanism accommodates the story drift using

Fig 7-11: Stacked Precast Panel Systems

Source: Reference 7-4

Fig 7-12: Mechanisms for Accommodating Story-Drift

connections with vertical slots or oversized holes that allow a rocking motion. In this system, the bearing anchors are detailed to not restrain uplift.

For panels that are completely supported by a single beam, such as spandrel units, no rocking or sway will occur within the unit. Thus, the entire story drift for a given floor must be tolerated by those elements, such as glazing, which are located between the spandrels (Figure 7-13). Window

systems installed between precast spandrel panels may be forced to accommodate the full story drift in half the story height, or less. Adjacent vertical precast or structural elements, that sway or rock, may impose additional demands on the ability of the glass pane/frame assembly.

For story drift out-of-plane to the cladding unit, good performance depends upon the ductility and rotation capacity of the bearing connection. Bearing anchors with flexible bodies generally provide better performance than those with more rigid bodies.

For a given architectural treatment of a wall elevation, there are usually a number of possible panel configurations that will yield an acceptable appearance and behavior. The selection of joint location and panel geometry will depend upon a variety of factors, including:

- Location of structural supports. The panels should be arranged such that they can be supported with a minimum of secondary supporting structure elements.
- The joints between panels should be arranged and detailed to minimize the amount of interaction between adjacent units due to building story drift, including the effects of structural member rotations. For the purposes of studying the effects of different panel joint layouts, the precast elements may be considered rigid bodies.
- The width of the panel-to-panel joints must be determined. Typical panel joint widths range from 1/2 to 3/4 inch. Joints less than 1/2 inch in width may not provide sufficient tolerance for story drift and variations in panel size. However, joint width may be influenced by esthetic considerations.

Fig 7-13 Effect of Story-Drift on Glazing Between Spandrels

- Economy and ease of erection. Every attempt is made to hold the number of different panel types and variations to a minimum. Panels must be of a size suitable for placement by the available cranes, considering the reduction of lift capacity with increasing reach. Connection details that require alignment and field welding while the panel is suspended by the crane are less desirable than those that allow the panel to be set, aligned, and welded off later.
- Overall panel size and shape are limited by the maximum allowable dimensions for truck loads. These maximum dimensions vary from state to state. If the cladding elements are site cast, this restriction need not be considered.

The layout of panel joints can have a significant effect on the performance of the cladding system. Interaction, or "bumping" between adjacent cladding units, which can occur if the panel joints close due to building story drift, can result in substantial and sudden impact loads to the panel connections.

When determining panel joint layouts to minimize the possibility of interaction between units, the story drift mechanisms for adjacent units, as well as possible effects of panel support rotations, and the effects of the rotation of the columns, and the beams near the beam/column joints must be considered (Figure 7-14). Panels supported on columns may undergo substantially different rotations than those supported on beams.

Panel layout at the building corners is especially critical. Two common building corner conditions are illustrated in Figure 7-15. Panels with mitered joints at the corners can

Fig 7-14: Effect of Story-Drift on Panel Joints

perform well, but the mitered joint undergoes extreme distortions during lateral building movement. Corner panels with square joints are somewhat more susceptible to bumping (Figure 7-16).

A common corner condition features panels that are "L" shaped in plan (Figure 7-15). This configuration can be designed to provide satisfactory results, but connection arrangements where the panel is anchored to the building frame in two orthogonal directions should be avoided. When "L" shaped corner panels are connected to the structure in two orthogonal directions, the panels must warp to accommodate story drifts, which can result in severe cracking.

For tall narrow cladding units, such as column covers, the rocking mechanism has several advantages over the sway mechanism. When the sway mechanism is used, the lateral connections for forces parallel to the building frame are often located at the center of the column cover, which minimizes connection overturning (rocking) forces due to the in-plane panel inertia forces. This causes a lateral reaction to be applied against the primary structural element (typically a column) at mid-height, which may not be desirable. In addition, structural rotations of the structural column may rotate the cladding unit, resulting in variations in joint widths between the column cover and adjacent panels. In contrast, when the rocking mechanism is used for column covers, it allows the lateral inertial loads to be resisted by connections at the top and bottom of the cladding element, which reduces demands on the supporting column. In addition, differential joint movements caused by column rotations are less severe for tall narrow panels which use the rocking mechanism.

Because panels using the rocking mechanism rotate to accommodate lateral story drift, while cladding elements using the sway mechanism translate, the joints between any adjacent units which use the different mechanisms must be carefully evaluated.

7.3.4 CONSIDERATIONS IN CONNECTION DESIGN

Layout and placing of precast concrete cladding connections are controlled by a number of factors, including panel configuration and the nature of the supporting structure. In general, panel connections may be classified as "lateral" type anchors, which resist loads parallel and/or perpendicular to the face of the panel, and "bearing" type anchors, which resist vertical loads, and may also resist lateral loads. Sections of some typical cladding units detailed using seismic design practices are shown in Figure 7-17.

When laying out the panel anchor points, the following should be considered:

- Panel volume change effects and building movements. Precast concrete panels experience thermal movements, long term shrinkage due to curing of the concrete, and movements of the supporting structure from applied loads and thermal effects. Where possible, connection configurations that will restrain these movements should be avoided. Determinant force resisting systems are preferred. If indeterminate support configurations can not be avoided, such as multiple out-of-plane lateral supports for long spandrel panels, then the design of the connections and panel reinforcement should include consideration of forces due to thermal bowing, length changes, shrinkage, and forces induced by primary structure deformations.
- The weight of the entire panel should be supported entirely at one level. For spandrel and single-story wall panels, the load support connections

Fig 7-17: Anchor Arrangements for Precast Cladding Units

should be placed at the top of the panel where possible. Panels with load support connections located at the bottom of the panel are more vulnerable to catastrophic failure, because should one of the panel lateral connections fail, the panel will rotate away from the building and fall.

- Tall narrow panels, such as column covers and multi-story wall units using the sway mechanism for story drift should have lateral connections for forces parallel to the building frame placed as near to the center of gravity of the panel as possible, to minimize overturning effects. Alternately, these panels can be designed to accommodate story drift using the rocking mechanism.
- The weight of glazing systems and other architectural elements supported by the cladding units should be included in the panel design.

Connections for precast concrete cladding panels can be fairly complex, depending upon the size and configuration of the panel, the nature of the supporting structure, and the relationship between the panel anchor points and their supports. Connections in the panel are usually made using cast-in place headed studs or anchor bolts, reinforcing steel, or embedded structural steel shapes or plate. Steel plates or shapes are welded or bolted to the embedded items, and are in turn bolted or welded to the supporting structure. Many of the standard connection details commonly used in low seismic zones lack the necessary ductility and the ability to sustain load reversals to be appropriate for seismically resistant

designs. An example of this are connections that rely on wedge or slotted inserts, or seat connections that rely on friction..

The design of precast connections is strongly influenced by "secondary moments" induced by load eccentricities. The principal eccentricities which influence the design are the unavoidable result of cladding geometry and the relative position of the plane of the precast panel to the support locations on the primary structure. However, these eccentricities may be increased by field conditions.

Precast concrete connections require adjustability to allow for the erection and fabrication tolerances of both the supporting frame and the precast item. For appearance sake, it is generally not acceptable for cladding elements to "follow the frame". The magnitude of the tolerance needed to accommodate building deviations depends upon the nature of the structural frame of the building and the height of the structure. To accommodate erection tolerances for a steel frame fabricated in accordance with the AISC "Code of Standard Practice," a cladding panel must be able to accommodate variations in plan of the support points of up to 3 inches. In addition, fabrication tolerances for the cladding component must be included, further increasing the support point dislocations which may occur. Because of the high design loads for precast cladding connections, and the large magnitude of the erection tolerances, relative to the connection dimensions, provisions for tolerances can have substantial impact on the design of connections. Failure to allow for erection tolerance in the design can result in extensive field modifications, and if uncorrected, can result in overstress in the connection elements and premature failure of connection components.

7.3.5 CONNECTION DESIGN USING THE UBC

The UBC recognizes that precast concrete panel connections are critical, and are vulnerable to earthquake damage. In past earthquakes, failure of panel connections in strong ground motion has often been catastrophic, resulting in cladding elements falling from buildings. The UBC precast panel connection design criteria reflects the crucial importance of each panel connection, due to:

1. the lack of redundancy in precast panel anchorage systems.
2. the limited ductility and energy absorption capacity of panel connections.
3. the falling hazard in the event of panel connection failure.
4. the potential for interaction of the cladding panels with each other and the primary structure.

Precast concrete cladding elements must be supported by positive, direct mechanical connections. Acceptable connections may be achieved using cast-in-place concrete or with

mechanical connections and fasteners. In addition to the lateral loads specified in the UBC, vertical seismic loads should be considered in the design of cladding elements such as soffit panels and panels that are cantilevered from the primary structure. The UBC specifies high lateral design loads for connection components (Figure 7-18). Portions of the connection deemed to be non-ductile must be designed for four times the lateral forces used to design the body of the panel (i.e., $4 \times 0.75 ZW = 3 ZW$). Non-ductile portions of the connection include:

1. Inserts, studs, dowels, and steel shapes cast in the concrete panel.
2. Bolts and welds (this would include the threaded rod in "push-pull" type connections).
3. Other elements of the connection that might fail in a non-ductile fashion.

The remainder of the connection assembly, or "body of the connection" is designed for one and one-third times the panel body design load (i.e., $1.33 \times 0.75 ZW = ZW$). The intent of the code is that all inelastic behavior be confined to the ductile body of the connection.

To ensure ductile behavior of precast concrete cladding connections, the design should insure that the failure mechanism of the connection is governed by tension or bending in a steel element. The strength of inserts, bolts, studs, or bars embedded in concrete, compressive buckling strength of steel elements, and weld capacities should always be greater than the tension or bending strength of the ductile steel element, or "body of the

connection". Design of the components of the connection assembly should be based upon strength design principles, rather than working stress design.

Connections should be detailed such that the analysis of force transfers through the connection is as straightforward as possible. The number of transfers of force in the connection should be kept to a minimum, to minimize the complexity and maximize the efficiency of the connection. All force components that the connection will be subject to must be considered in the design. Examples of schematic connections, illustrating the number of force transfers are given in Figure 7-19.

When welded rebar is used in connections, ASTM A706 grade bars must be specified. Most reinforcing bars which meet ASTM A615 Grade 60 are not weldable, and bars meeting ASTM A615 Grade 40 may or may not be. Reinforcing bars, bolts, and studs used in connections should not be tack welded, and welding should not be permitted within 4 bar diameters of bends.

The design of precast concrete connections must include the effects of all secondary moments and torsions due to eccentricities within the connection. Where possible, embedded bolts, bars and shapes should be hooked or otherwise terminated to effectively transfer pullout forces directly to the panel reinforcing bars.

Members of the primary structure which support precast concrete cladding connections must be adequately braced for torsions and eccentricities due to panel loads, with adequate provision made for variations in the point of application of loads due to erection and fabrication tolerances. Members with inadequate rigidity or lateral bracing may deflect or rotate under panel reactions, resulting in increased loading, prying, and premature failure of the precast connections (Figure 7-20). Members supporting column and beam covers are especially prone to excessive rotation under the effects of panel reactions. The design

eccentricity for the primary structural supporting members will vary, depending upon the configuration of the connection system.

Bearing connections resist vertical gravity loads, as well as vertical seismic forces that result from panel configuration and anchor locations. In addition, bearing connections may provide resistance to lateral forces as well. Examples of typical bearing connections are shown in Figure 7-21.

7.3.6 LATERAL CONNECTIONS

Lateral anchors are detailed to resist only specific force components. Care must be taken to ensure that a reliable mechanism is provided to allow the desired movement in the connection, and ample provision must be made for fabrication and erection tolerances. Connection movement can be achieved using either sliding "slotted-hole" mechanisms, or by bending of steel elements. In general, connections that rely on sliding are generally less reliable, because they rely on an uncertain degree of friction and because "slotted-hole" bolts are often overtightened during erection. Examples of the two types of lateral connections are shown in Figure 7-22.

Connections that allow movement by bending of steel elements can be made using either steel plates or rods. While it is desirable that the bending elements remain essentially elastic

Fig 7-22: Lateral Connections

through the full range of connection movement, some inelastic behavior is often unavoidable. To minimize the possibility of low cycle fatigue, the yielding element should be mild steel, and inelastic rotations should be limited to hold material strains below the threshold of strain hardening. Lateral connections using threaded rods, known as "push-pull" connections in the western United States (Figure 7-22a) are commonly used for lateral connections. Some research has suggested that there may be low-cycle fatigue problems in these connections when the rod is subject to repeated cycles of large-strain inelastic bending. Therefore, it is prudent to design these connections using rods of sufficient length to minimize inelastic bending in the threaded rods. Threaded rods fabricated from very high strength steels, such as coil rod, should be avoided if plastic bending is expected, since they may be vulnerable to low-cycle fatigue failures when cycled inelastically.

Lateral connections that allow movement through weak axis yielding of steel plates can be highly effective. By balancing the design of the embedded items, bolts, and welds of the connection, yielding of the plate can be assured, resulting in a highly reliable mechanism. These types of connections are especially suitable for bearing anchors on panels utilizing the rocking mechanism for drift tolerance. As with rods, inelastic yielding in the strain-hardening range should be avoided.

Lateral connections that rely upon sliding mechanisms (Figure 7-22b) must be carefully detailed and installed to perform adequately. A number of factors influence the behavior of sliding connections, including:

- The slot must be of sufficient length to allow for the full range of motion of the structure. Allowance must be made for fabrication and erection tolerance.
- The sliding bolt must not be over-tightened, or sliding action of the connection will be inhibited. Drawings will often call for bolts or nuts to be installed "finger tight". Such instructions are subjective, and actual installation depends largely on the skill of the worker. In addition, it is difficult to verify in the field that the bolts are correctly installed and tightened.
- When designing sliding connections, the clamping effects of axial loads in the connection must be considered. Static coefficients of friction for steel-to-steel contact vary widely, from 0.2 to over 1.0, depending upon the condition of the surfaces. Burrs, fireproofing, rust, and other surface irregularities may inhibit the onset of sliding. The connection should be designed so that sliding can be initiated under worst case conditions at shear loads below the ultimate shear capacity of the connection. Low friction washers may be necessary to reduce the force needed to initiate sliding.

7.3.7 SUPPORT AND PANEL BODY DESIGN CONSIDERATIONS

For some panel configurations, it is necessary to provide panel-to-panel connections, that is, some or all of the inertial forces are resisted by adjacent cladding units. When some panel support points are provided by adjacent cladding elements, and others connections are made directly to the primary structure, extreme care is necessary when considering relative movements of the combined cladding units and the primary structure.

The structural members supporting precast concrete must be sufficiently rigid and adequately braced to prevent rotation or excessive deflection. Structural members with insufficient rotational or bending stiffness can result in dramatic increases in the connection loads of the supported precast concrete cladding units. Because of the high concentrated loads on the primary structural supports at the precast concrete connections, local bending of webs and flanges, torsion, and web crippling should be checked for structural steel supports, including the effects of field and erection tolerances. Embedded plates and shapes in reinforced concrete sections of the primary structure that provide support for precast concrete cladding connections should be designed to the same load criteria as the panel connections themselves.

When designing and detailing the panel reinforcement for precast cladding units, confinement of the concrete in the vicinity of highly loaded connections is recommended.

While not considered in the design of embedded bolts and shapes, the presence of confining reinforcement increases the ductility of embedded items substantially. Aside from the connection regions, loads associated with lifting and handling of the cladding units generally govern the design of the panel reinforcement.

Considerations for glazing design for window systems supported by precast cladding units are similar to those discussed in the following section.

7.4 METAL CLADDING WALL SYSTEMS

Metal cladding (or curtain) wall systems are popular alternatives to precast concrete cladding. The metal systems possess several advantages over precast concrete, including substantially lower weight, great flexibility in design and layout, and relative ease of construction. As discussed in Section 7.1.2, metal curtain wall systems must meet the same drift criteria as precast concrete cladding systems.

Metal cladding wall systems may be assembled at the construction site, and installed piece by piece on the building elevation, or assembled offsite and erected onto the structure in "panelized" units, similar to precast concrete cladding panels. In either case, the cladding system usually consists of a grid of steel or aluminum members, in which glass panes or metal panels are installed. The supports for the glass or metal panels are detailed to allow for thermal movements of the system, and to allow for lateral building movements. In general, industry standard details may not allow sufficient movement for story drift under seismic loads.

Because metal cladding systems are lightweight and their connections are generally lighter, the impact of cladding inertial loads on the supporting structural members is generally minor. For almost all typical curtain wall applications, wind forces will govern the design of the cladding wall system and its connections. From a seismic design perspective, the primary objective is proper detailing of the wall system to allow for building lateral story drift.

Metal cladding systems are much less rigid than their precast concrete counterparts. The cladding system is generally designed to move with the structural frame. Story drifts are accommodated by racking of the glazing frame (Figure 7-23). Under high story drifts, the frames may become severely distorted, but since these assemblies are lightweight and inherently ductile, connection failure and separation from the primary structure is considered unlikely. However, special precautions must be taken to protect glass panes from the effects of frame distortion due to story drift.

Although glass has considerable in-plane strength and out-of-plane flexibility, it is unable to tolerate in-plane racking, such as that which occurs if the corners of the pane are contacted by the cladding frames distorted by story drift. The minimum gap which is required to

Fig 7-23: Drift Accommodation in Glass and Metal Curtain Walls

prevent racking of the glazing is a function of the magnitude of deflections expected, the size and shape of the glazing panel, and the type of gaskets used to seat the pane into its mullions. It is often assumed that seismic movements will be accommodated by the gaps between the vertical mullions and the glass panes, but the minimum gap size is not specified by code, and it is rarely checked against the installation drawings in the field.

While the dimensions of the glass panes or metal panels are controlled, control of the frame opening dimensions is much less rigid. Erection tolerances can cause opening sizes to vary considerably. For good seismic performance, the detailing and installation of the glass and frame must therefore be carefully reviewed.

As with precast concrete cladding units, the effects of story drift on metal curtain wall systems will vary, depending upon the relationship between the frame system, the primary structure, and adjacent rigid structural and non-structural elements. Panelized metal cladding (cladding panels which are pre-assembled into larger units before erection) wall systems can be designed using an approach similar to that for precast concrete cladding. Full advantage can be taken of the lighter weight and higher overall ductility of the metal system. When designing panelized systems, the main goal should be the prevention of racking of the glazing elements.

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DESIGN APPROACHES FOR SPECIAL SITUATIONS

8.1 COMBINATIONS OF STRUCTURAL SYSTEMS

8.1.1 TYPES OF COMBINATIONS

Basic structural system types defined by the UBC are described in Section 2.2.1. These systems provide lateral resistance to structures against one or more direction of seismic forces. However, structure design requirements sometimes require that a structure be designed using combinations of systems.

The UBC defines three specific types of combinations which are permitted, subject to the special design requirements described below: dual systems, combinations along different axes (directional), and vertical, or stacked combinations.

Combinations which do not meet these descriptions tend to be random horizontal mixtures of walls and frames which usually result from poor design practices or from a building with a history of structure additions.

Structures which are mixed combinations of moment frames, walls, and bracing systems, particularly if the layout is not symmetric, tend to perform poorly. Even where the aggregate center-of-rigidity of the combined elastic system is reasonably concentric, the post-elastic behavior of each system is different. Therefore, as yielding occurs, the center of rigidity may shift, and the load path of seismic forces will be hard to predict, resulting in greater damage to some elements.

For any type of combination of structural systems, elements and components which are common to the separate systems must be designed and detailed using the more restrictive detailing requirements of each system.

Where special ductile systems such as Special Moment Frame (SMF), Intermediate Moment Resisting Frame (IMRF), or Eccentric-Braced Frame (EBF) systems are used, or where special ductile elements such as concrete boundary elements are required, the UBC requires that the special detailing of these elements must be continued down to the foundation for all elements which are required to transmit forces from these elements.

8.1.2 DUAL SYSTEMS

A dual system, as defined in Section 2.2.1, is a specific type of Basic Structural System, which consists of a parallel combination of two separate types of structural systems.

Generally, this combination consists of a relatively rigid primary system, using walls or braced frames, plus a special-moment frame secondary system. The intent of the secondary system is to provide improved ductility to the primary system.

The UBC requires that the combined system must be designed to resist the entire basic code-specified static force defined for dual systems. In addition, the backup special-moment frame must be designed to resist at least 25 percent of the total code-specified force.

In order to ensure that shifting of the center of rigidity when yielding occurs does not subject the structure to unusual torsional stresses, the structure layout of both systems should provide a center of rigidity in approximately the same location.

8.1.3 COMBINATIONS ALONG DIFFERENT AXES

For structures which exhibit orthogonal behavior (i.e., do not contain any non-orthogonal, or skew elements), it is common to analyze and design each orthogonal axis of the structure for seismic forces separately. It is also common to utilize a different type of basic structural system for each orthogonal axis of the structure.

In seismic zone 3 and 4, structures utilizing a bearing wall system to resist seismic forces in one direction must use an R_w -factor in other direction which is no greater than that of the bearing wall system.

8.1.4 VERTICAL COMBINATIONS

Vertical combinations of structure systems are common where above-ground structures, either rigid or flexible, are constructed on top of generally rigid below ground structures.

Normally, structures which include vertical combinations of structural systems are highly prone to the vertical irregularities described in Section 2.4.1, particularly stiffness irregularities, in-plane discontinuities, and weak stories. Specific design requirements for each type of irregularity are described in Section 2.4.

The value of R_w used in each direction to design any structure story must be less than or equal to the value of R_w used in the story above, even for the case of ductile framing (associated with high values of R_w) on top of brittle (low R_w value) framing. There are two exceptions to this:

1. for light "penthouse" structures, where the weight of the supported structure is less than 10 percent of the total structure, and
2. where the supported structure is significantly more flexible than the lower structure (the opposite of a "soft story" condition), as described below.

If the structural system of the upper portion has greater lateral stiffness than the lower portion, a "soft story" irregularity will result. In this case, seismic forces in structure elements must be determined using the dynamic analysis method.

If the stiffness of both systems is roughly equal, within the range of the definitions of irregularity, the static analysis method may be used, if no other irregularities are present which would require the dynamic method.

If the stiffness of the upper structure is sufficiently flexible with respect to the lower portion, the structure is permitted to be designed using the static analysis method with, in some cases, separate R_w values for each structure portion (as described in Section 8.1.5). Before this procedure can be used, it must be shown that:

- Each structure section (upper and lower), when considered by itself, would be classified as regular (having no significant vertical or plan irregularities).
- If the stiffness of the lower structure is greater than 10 times that of the upper structure.
- If the period of the entire structure is not greater than 1.1 times the period of the upper structure alone, when considered as a separate structure fixed at the base.

The UBC requires that the value of the R_w -factor used in the design of each building story must be less than or equal to the value of the R_w -factor used for the design of the story above. Normally, this means that the forces which an entire structure must resist are determined using the lowest value of R_w of any lateral-force resisting system used in the structure.

However, for the special case consisting of a flexible upper portion supported on a rigid lower portion, as defined in Section 8.1.4, a special static analysis method is defined, as follows:

- If the rigid lower portion has a higher R_w -value than the flexible upper portion, then the forces which the entire structure must resist are determined using the R_w -value of the rigid lower portion.
- If the rigid lower portion has a lower R_w -value than the flexible upper portion, then:
 1. The flexible upper portion is designed as a separate upper structure, supported laterally by the lower portion, using the R_w -value in each direction defined for the structural system used in the upper structure portion.

2. The rigid lower structure is then designed as a separate structure, using its own R_w -value. Reactions from the upper portion, amplified by the ratio of $(R_w)_{Upper} / (R_w)_{Lower}$ are applied as forces against the lower portion. These reactions should include both the story shear forces and the vertical forces which result from overturning moments.

8.2 BUILDING ALTERATIONS AND ADDITIONS

Structures are often altered or added to at some point in their existence. Because of this, engineers are often called on to design alterations or additions to existing structures or facilities. The seismic design of these features can be technically challenging, particularly where existing structure elements have previously been designed to outdated or substandard requirements.

An addition can be configured as either an attached "lean to" structure, or as a structurally separate structure (using a seismic separation joint). For very small attachments or penthouses, the scope of seismic analysis which is appropriate may be of the portion which is added only. For larger additions, it is appropriate to include (and thus, re-qualify) the entire existing structure, unless a suitable seismic separation joint is provided (sometimes, permitting agencies require re-qualification of the existing structure even where a separation is provided, if the addition is of sufficient size).

At what size is a full analysis of the combined structure and addition required? The UBC does not provide clear guidance, but states that alterations or additions shall not be made which result in a violation of the code or cause the building to become unsafe. This is a moot point when the existing structure is already an unsafe structure due to its age, condition, and/or seismic design basis.

California Title 24 Requirements (IR-16-1) suggest a more useful criteria for determining when the combined structure/addition must be reviewed and, where necessary, strengthened. Major structural alterations or additions, defined as alterations which would reduce the story shear resisting capacity by more than 10% or increase the story shear force by more than 10%, require a complete structural analysis of the combined building and additions for conformance with the current building code (including required Importance Factors).

No matter what the size of the alteration or addition, the design engineer should be required to demonstrate a clear understanding of the structural system and behavior of both the existing building and the planned alteration or addition. This does not need to involve calculations, but should include a description of the basic lateral-force system which includes a summary of the principal resisting elements in the existing structure as well as an estimate of the possible lateral deflections.

Specific analysis and design considerations should include the following:

- If the existing building diaphragm is flexible, the existing building may be able to be evaluated solely in terms of the tributary building portion which will be effected by the addition, either in terms of tributary forces or of modified or added resisting elements. On the other hand, if the existing building's diaphragm is rigid or semi-rigid, the structure may need to be evaluated as a whole to determine the effects of the addition.
- The analysis should include an analysis of the compatibility of the existing and new structural systems. Even a small addition can be hazardous if it consists of a rigid shearwall structure which is eccentrically attached to a flexible moment-frame structure.
- The analysis should include a consideration of building drifts at separation joints. If calculated deflections are not available for the existing structure, assume that the deflection is equal to the code permitted drift.
- If the addition is attached to the existing structure, consideration should be given to whether existing building drag and collector members are adequate to transfer forces to/from the addition.

8.3 SEISMIC RETROFITS

Where existing structures are required to be reviewed and potentially strengthened, many special analysis, design, and construction problems must be addressed. These problems include 1) the selection of the appropriate analysis and design criteria, which is typically based upon a desired level of seismic performance, 2) problems related to combining different types and ages of construction, 3) finding ways to locate required retrofit measures into the structure while minimizing the impact on building use, both during and after construction, and 4) design and construction problems unique to modifying and working in existing structures.

8.3.1 STRUCTURE TYPE

Types of construction which historically have represented varying degrees of hazard during earthquakes are as follows, in their approximate order of decreasing seismic hazard:

1. Unreinforced masonry (URM) bearing wall construction; typically in conjunction with wood floor framing which is poorly or not tied to the walls.

2. Under-reinforced concrete bearing wall construction; typically with concrete floor framing which is nominally tied to the walls.
3. Precast concrete systems lacking continuity ties and homogeneous diaphragms.
4. Non-ductile concrete moment-frames.
5. Steel or concrete frames with URM masonry infill.
6. Non-ductile steel or concrete frame with randomly located reinforced infill walls.
7. Older steel frames with concentric bracing having minimal ductility or weak connections.
8. Older steel frames or flexible diaphragm systems which do not meet deflection criteria (affects primarily non-structural contents).

8.3.2 RETROFIT DESIGN CRITERIA

In order to establish an appropriate design criteria, an appropriate performance objective must be selected, and the design engineer must assess what R_w and Importance Factor is appropriate to use for an existing structure.

Seismic design criteria for retrofit projects are typically based on one of two basic performance descriptions; life safety and damage control.

- *Life safety retrofits.* This level of strengthening is intended to prevent a structural collapse; major damage is possible, and could include a small amount of potentially dangerous falling debris. Typically, the rigidity of life-safety retrofit measures relative to the existing structure is not a significant concern, since the stiffness and strength of existing elements is allowed to degrade until the relative rigidity of the retrofit measures becomes significant.
- *Damage control retrofits.* This level of strengthening represents an increase to the lateral capacity of the building, with the intent that stresses and/or deflections which will occur during an earthquake will not cause significant damage to the structure. Damage control retrofits are appropriate for important facilities, where an extended shut-down of operations for inspections and repairs following a major earthquake cannot be tolerated.

For damage control retrofits, the rigidity of the retrofits measures, relative to that of the existing structure, can be a major issue, since the stiffness of the retrofits must be great enough to reduce lateral drift to a level which will control damage.

Structure R_w -factors provided in the current UBC are intended for modern materials and methods of construction and may be inappropriate for archaic materials and methods. Also, UBC factors assume a certain level of quality in the construction, which is ensured by both periodic and special inspections. Older structures may not have been inspected, may have an uncertain quality of construction workmanship, and may contain details of construction which are undesirable from a seismic standpoint.

Where specific factors are not available, the design engineer must interpolate from existing factors and judge what an appropriate design criteria should be. Additional guidance in this regard is provided in Reference 8-3.

8.3.3 DESIGN AND CONSTRUCTION ASPECTS

Once construction begins, expect to be surprised. Construction work will uncover structure aspects which will conflict with design assumptions, and details and components of the existing structure will be discovered which interfere with the proposed retrofit. If the retrofit has been designed with little or no tolerance in both the stress margins and the fit of installation of retrofit components, major problems and redesigns can result. In order to reduce these problems, the design engineer should always provide adequate tolerances in the design to accommodate field changes. In addition, a generous contingency budget should be provided in the approved construction budget (generally 10% to 20% of the projected cost, but potentially as much as 25%) to accommodate the cost of field changes and rework.

8.4 STRUCTURE AND GROUND INTERACTION

Normally, the seismic analysis of a structure assumes that the structure foundations are fixed against movement. Uniform ground motions translate into inertial forces which are applied to all parts of the structure simultaneously. For structures which are founded directly on bedrock, this assumption is approximately correct. For structures founded on softer soils, however, the actual behavior of the ground during an earthquake is much more complex. Specific variations in behavior include site amplification effects, surface wave and traveling wave effects, soil-structure interaction effects, and the effects of structure embedment within the soil.

8.4.1 SITE AMPLIFICATION EFFECTS

The majority of ground motions during an earthquake result from wave motions which are transmitted through bedrock and then transmit upward through the soil layer. As an elastic medium, the soil layer has natural frequencies and mode shapes which are independent of the structure. This has the effect of *amplifying* motions within some frequency ranges

(those within the range of the natural frequency of the soil mass). These effects are already included in seismic design procedures in terms of either site-dependent response spectra or the code soil-type factor in the basic seismic force equation.

8.4.2 SEISMIC WAVE EFFECTS

When seismic motions propagate upward from bedrock, the resulting surface motions are not equal at every point of the ground surface. Some seismic motions propagate horizontally as *surface waves*. A portion of seismic motion at any point is *random motion*, resulting from refraction and reflection of wave motions. As a result of these effects, large structures will be subjected to *average* translational motions which are less than the maximum translational motions which would be measured by a seismic instrument located at any single point. In addition, structures may be subjected to deformations due to undulating ground motions, and structures which are very long may be subjected to torsional and/or out-of-phase motions due to spatial ground motion variations, called *traveling wave effects*. These effects are not normally specifically addressed during design, although these effects are included to some extent in the code requirement of tying foundations together, and in the requirement to add an additional "accidental eccentricity" to the calculated torsional response of the structure. In general, the effect of wave passage on lateral structure response is greatest for rigid structural systems (Period < 0.06 sec.), significantly smaller for moderately stiff systems (0.06 to 0.5 sec.), and negligible for flexible systems.

8.4.3 SOIL-STRUCTURE INTERACTION

For large structures, the size and weight of the structure may be sufficient to affect the ground motion which occurs in the immediate vicinity of the structure. Also, the structure and foundation may exhibit some rigid-body rocking and translation motions in conjunction with an immediate "attached" portion of soil. These effects are known as *soil-structure interaction* (SSI).

Soil-structure interaction refers to the phenomenon of modification of earthquake response of a structure founded on soil because of the deformability of the soil. The SSI phenomena is most predominant for large, massive structures, founded on relatively soft soils.

The primary effect of SSI is to increase the period of vibration of the coupled soil-structure system, and to increase the total damping of the system. The increase in period of vibration (e.g. frequency shift) may shift the response of the structure to either a higher or lower response region of the response spectrum. All other SSI effects, such as the large values of soil damping (material and radiation) that are associated with the soil modes, result in a reduction in the response levels (inertial forces) within the structure.

It is generally reasonable to assume that soil-structure interaction affects only the first mode of vibration of a structure. On this basis, Veletsos and Meek (Ref. 8-9) have suggested that consideration of soil-structure interaction is only warranted where:

$$v_s/(fH) < 20$$

where:

v_s = the shear wave velocity in the soil half-space, in feet/second.

f = the fixed base frequency of the single degree-of-freedom structure (or the frequency of the fundamental mode of a more complex structure), in cycles/second.

H = the structure height, in feet

Note that this formula is ratio of the soil wave velocity to the "wave" velocity of the structure's primary vibration mode; hence, it is simply a check for possible wave-coupling effects.

When significant SSI effects are expected, the change in period (frequency) of the soil-structure system must be estimated. It is permissible to ignore other effects of SSI. However, there may be cases where it is very conservative to ignore these effects.

Where consideration of SSI effects cannot be ignored, the two methods currently used to analyze structures are:

- the direct approach, and
- the substructure approach.

In the direct approach, the structure and the surrounding soil medium are modeled together and usually analyzed in one step by finite element techniques. In the substructure approach the soil medium is replaced by frequency-dependent impedance functions to represent its dynamic stiffness and damping effects. Additional guidance in the selection of analytical methods is available in References 8-6 through 8-9.

The dynamic analysis of soil-structure systems is one of the most complex and demanding tasks in earthquake engineering. The selection of analysis parameters such as equivalent linear soil properties, the control point at which the input motion is defined, and the control "input" motion is surrounded by some degree of uncertainty; thus the interpretation of analytical results must be tempered with engineering judgment. Some of this uncertainty can be bounded in the analysis by varying the soil properties and performing multiple analysis. The cost, complexity, and validity of such exercises are major considerations which should be addressed before such an exercise is undertaken.

8.4.4 UNDERGROUND EMBEDMENT OF FOUNDATIONS

A special type of soil-structure motion occurs when a structure is partially or completely embedded below ground. A rigorous analysis of embedment effects requires a soil-structure analysis, however, embedment is common, and it is impractical to always approach the analysis rigorously. Thus, simplified design assumptions must be made.

The code requires that a complete load path must be provided between the point of origin of forces and the load resisting elements. It also requires that seismic forces must be calculated using weight, w_i , and height, h_i , which are calculated as extending above the structure's "base," in which the "base" is defined as the level in which seismic motions are imparted from the ground into the structure.

For structures containing underground elements (i.e., basements), this means that seismic forces must be transferred to the point in the foundation system where the soil is determined to provide resistance. For buried structures this point is usually located below the level of grade, sometimes even at the lowest level of the structure. Therefore, while passive soil pressures against exterior basement walls may provide some degree of soil restraint, it is common to assume that all lateral forces are transferred into the surrounding soil at the foundation level beneath the basement. Exterior basement walls should be provided with adequate capacity to resist possible soil pressures resulting from seismic soil pressures, however. Reasonably conservative values of seismic soil pressures to use for design purposes can generally be provided by a geotechnical consultant.

Where more accurate predictions of these effects are required, the interpretation and prediction of seismic soil pressures should be developed in conjunction with an experienced geotechnical engineer. References 8-4 and 8-5 describe procedures for determining soil pressures which have been shown to provide reasonable agreement with test results. A rigorous analysis of embedment effects should consider:

- the rigidity (deformation) characteristics of the substructure;
- the general size, shape, and stiffness of the superstructure;
- the stiffness and depth of both the surrounding and the supporting soil mass;
- the degree to which site soils may be disturbed by the construction of the structure, including the types and the degree of compaction of any backfill materials used; and
- the degree to which seismic motions are in tune with structure periods vs. the natural periods of the surrounding soils.

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Address all comments to:

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