Seismic and Wind Design of Concrete Buildings

(2000 IBC, ASCE 7-98, ACI 318-99)

S.K. Ghosh and David A. Fanella

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First Printing: June 2003
Second Printing: March 2004
Third Printing: July 2004

ISBN # 1-58001-112-8

PRINTED IN THE U.S.A.

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PREFACE

The landmark volume, *Design of Multistory Concrete Buildings for Earthquake Motions* by Blume, Newmark, and Corning, published by the Portland Cement Association (PCA) in 1961, gave major impetus to the design and construction of concrete buildings in regions of high seismicity. In the decades since, significant strides have been made in the earthquake resistant design and construction of reinforced concrete buildings. Significant developments have occurred in the building codes arena as well. However, a comprehensive guide to aid the designer in the detailed seismic design of concrete buildings was not available until PCA published *Design of Concrete Buildings for Earthquake and Wind Forces* by S.K. Ghosh and August W. Domel, Jr. in 1992.

That design manual illustrated the detailed design of reinforced concrete buildings utilizing the various structural systems recognized in U.S. seismic codes. All designs were according to the provisions of the 1991 edition of the *Uniform Building Code* (UBC), which had adopted, with modifications, the seismic detailing requirements of the 1989 edition of *Building Code Requirements for Reinforced Concrete* (ACI 318-89, Revised 1992). Design of the same building was carried out for regions of high, moderate, and low seismicity, and for wind, so that it would be apparent how design and detailing changed with increased seismic risk at the site of the structure.

The above publication was updated to the 1994 edition of the UBC, in which ACI 318-89, Revised 1992, remained the reference standard for concrete design and construction, although a new procedure for the design of reinforced concrete shear walls in combined bending and axial compression was introduced in the UBC itself. The updated publication by S.K. Ghosh, August W. Domel, Jr., and David A. Fanella was issued by PCA in 1995.

A similar publication of more limited scope by David A. Fanella and Javeed A. Munshi was issued by PCA in 1998. The detailed design of three reinforced concrete buildings in high seismic zones utilizing the various structural systems recognized in the 1997 UBC was illustrated. The designs for the combined effects of gravity, seismic, and wind loads were by the 1997 edition of the UBC, which had adopted the 1995 edition of ACI 318.

In the ensuing years, the building code situation in the country has changed drastically. In 1994, the three model building code organizations: BOCA, the Building Officials and Code Administrators International, the publishers of *The BOCA National Building Code* (BOCA/NBC); ICBO, the International Conference of Building Officials, the publishers of the *Uniform Building Code* (UBC); and SBCCI, the Southern Building Code Congress International, the publishers of the *Standard Building Code* (SBC), formed the International Code Council (ICC) with the express purpose of developing a single set of construction codes for the entire country. Included in this family of *International Codes* is the *International Building Code* (IBC), which represents a major step in a cooperative effort to bring unity to building codes. The first edition of the IBC was published in April 2000.

The seismic design provisions of the First (2000) Edition of the IBC represent revolutionary changes from those of the model codes it was developed to replace. The ground motion maps and parameters used in seismic design are completely different. Also, since the inception of seismic design in this country, soil at the site of the structure has not been given as much importance in seismic design, as it is given in the 2000 IBC. This results in a significant impact on the cost of construction in many parts of the country, particularly if such construction is to be founded on softer soils. This has created a need for a publication similar to the volume first issued by PCA in 1992. This publication seems to fill that need. The purpose of this publication is to assist the engineer in the proper application of the seismic and wind design provisions of the 2000 edition of the *International Building Code*. This code has adopted, for concrete design and construction, with some modifications, the 1999 edition of *Building Code Requirements for Structural Concrete* (ACI 318-99).

In Chapter 1, an introduction to earthquake-resistant design is provided, along with summaries of the seismic and wind design provisions of the 2000 IBC. Chapter 2 is devoted to an office building utilizing a dual shear wall-frame interactive system in one direction and a moment-resisting frame system in the orthogonal direction. Designs for Seismic Design Categories (SDC) A, C, D, and E are illustrated in both directions. Chapter 3 features a residential building, which utilizes a shear-wall frame interactive system in SDC A and B and a building frame system for lateral resistance in SDC C, D, and E. Chapter 4 presents the design of a school building with a moment-resisting frame system in SDC B, C, and D. A residential building utilizing a bearing wall system is treated in Chapter 5. Design is illustrated for SDC A, B, C, D, and E. The final (sixth) chapter is devoted to design of a precast parking structure utilizing the building frame system in SDC B, C, and D. The Seismic design Category represents a combination of seismic risk at the site of the structure, occupancy category of the structure, and soil characteristics at the site of the structure. While design is always for the combination of gravity, wind, and seismic forces, wind forces typically govern the design in the low seismic design categories (particularly A), and earthquake forces typically govern in the high seismic design categories (particularly D and above). Detailing requirements depend on the seismic design category, regardless of whether wind or seismic forces govern the design. This publication is designed to provide an appreciation on how design and detailing change with changes in the seismic design category.

Although every attempt has been made to impart editorial consistency to the seven chapters, some inconsistencies probably still remain. Since this is the very first edition of a fairly extensive volume, some errors, almost certainly, are also to be found. The authors would be grateful to any reader who would bring such errors or inconsistencies to their attention. Other suggestions for improvement would also be most welcome.

ACKNOWLEDGMENTS

Saravanan Panchacharam, Structural Engineer, and Arnaldo T. Derecho, Senior Consultant, S.K. Ghosh Associates Inc., were major contributors to this publication. Their dedicated help is most sincerely acknowledged.

The writer is deeply grateful to his co-author, David A. Fanella, for "taking over" this publication since joining S.K. Ghosh Associates Inc. in late 2002. Dedicated professionalism and much hard work on his part finally brought this project to fruition.

Northbrook, IL *S.K. Ghosh*
June 2003

CHAPTER 1

INTRODUCTION

1.1 OVERVIEW

The purpose of this publication is to assist the engineer in the proper application of the seismic and wind design provisions of the 2000 edition of the *International Building Code* (IBC) [1.1] (numbers in brackets refer to references listed at the end of this chapter). The emphasis is on how to apply the IBC provisions in the design of reinforced concrete structures utilizing the various structural systems recognized in the code. The seismic and wind provisions of the IBC are discussed in detail below and are illustrated in subsequent chapters of this publication with examples of typical building and parking structures located in regions of low, moderate, and high seismic risk and founded on different types of soil. In the examples, structural members are designed and detailed according to Chapter 19 of the IBC, which is based primarily on *Building Code Requirements for Structural Concrete (318-99)* [1.2], with a few modifications.

The 2000 IBC was created by the International Code Council (ICC), a nonprofit organization established in 1994. The three model code bodies—Building Officials and Code Administrators International, Inc. (BOCA), International Conference of Building Officials (ICBO), and the Southern Building Code Congress International, Inc. (SBCCI)—are the founding members of the ICC. The 2000 IBC is the culmination of years of cooperative effort by committees representing the three model code bodies and other organizations.

Chapter 16 of the IBC addresses the design requirements for various types of loads and design load combinations. The ASCE Standard *Minimum Design Loads for Buildings and Other Structures* [1.3] is the basis of all provisions related to nonseismic forces. Thus, design for wind forces according to the 2000 IBC follows ASCE 7-98.

The seismic design provisions of the 2000 IBC are based primarily on the 1997 *NEHRP Provisions* [1.4], with some of the features of the 1997 *Uniform Building Code* (UBC) [1.5] included. A brief history of the evolution of seismic codes in the U.S. as these relate to the determination of design forces is given in the next section.

Throughout this publication, section numbers from the 2000 IBC are referenced as illustrated by the following: Section 1616 of the 2000 IBC is denoted IBC 1616. Similarly, section references from ASCE 7-98 and ACI 318-99 appear as ASCE 6.5 and ACI 21.2, respectively.

1.2 EARTHQUAKE-RESISTANT DESIGN

1.2.1 Background

1.2.1.1 Nature of Earthquake Motion

Ground motion resulting from earthquakes presents unique challenges to the design of structures. The forces that a structure must resist in an earthquake result directly from the distortions caused by the motion of the ground that supports it. The response—magnitude and distribution of forces and displacements—of a structure resulting from such ground motion is influenced by the properties of the structure and its foundation, as well as the character of the exciting motion.

Earthquakes produce large-magnitude forces of short duration that must be resisted by a structure without causing collapse and preferably without significant damage to the structural members. Lessons from past earthquakes and research have provided technical solutions that will minimize loss of life and property damage associated with earthquakes. For materials such as concrete that lack inherent inelastic deformability or ductility, a critical part of the solution is to provide special detailing of the reinforcement to assure a ductile response to lateral forces. Inelastic deformability is the ability of a structure to sustain gravity loads as it deforms laterally beyond the stage where the deformations are recoverable, i.e., beyond the stage where no residual deformations remain in a structure once the earthquake motion subsides. Irrecoverable deformations are associated with damage, while recoverable deformations are associated with no damage.

Figure 1-1 illustrates a simplified representation of a building during an earthquake. As the ground on which the building rests is displaced, the base of the building moves with it. However, the inertia of the building mass resists this motion and causes the building to suffer a distortion (greatly exaggerated in the figure). This distortion wave travels along the height of the structure. The continued shaking of the base causes the building to undergo a complex series of oscillations.

It is important to draw a distinction between forces due to wind and those produced by earthquakes. These forces are often thought of as being similar, just because codes specify design wind as well as earthquake forces in terms of equivalent static forces. Although both wind and earthquake forces are dynamic (varying in time) in character, a basic difference exists in the manner in which they are induced in a structure. Whereas wind loads are external loads applied, and hence proportional, to the exposed surface of a structure, earthquake forces are essentially inertia forces. The latter result from the distortion produced by both the earthquake motion and the inertial resistance of the structure. Their magnitude is a function of the mass of the structure rather than its exposed surface. Also, in contrast to structural response to essentially static gravity loading or even to wind loads, which can often be validly treated as static loads, the dynamic character of the response to earthquake excitation can seldom be ignored.

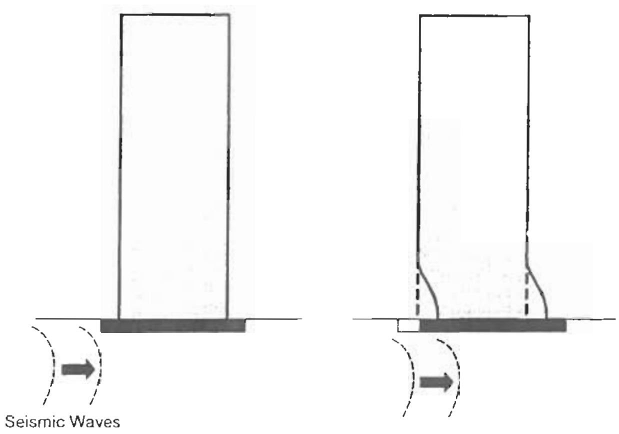


Figure 1-1 Simplified Representation of Building Behavior During an Earthquake

The earthquake ground motion quantity most commonly used in analytical studies is the time-wise variation of the ground acceleration in the immediate vicinity of a structure. At any point, the ground acceleration may be described by horizontal components along two perpendicular directions and a vertical component. In addition, rocking and twisting (rotational) components may be present; however, these are usually negligible. Because buildings and most other structures are most sensitive to horizontal or lateral distortions, it has been the practice in most instances to consider structural response to the horizontal components of ground motion only. The effects of the vertical components of ground motion have generally not been considered significant enough to merit special attention. In most instances, a further simplification of the actual three-dimensional response of structures is made by assuming the horizontal acceleration components to act non-concurrently in the direction of each principal plan axis of a building. It is implicitly assumed that a building designed by this approach will have adequate resistance against the resultant acceleration acting in any direction.

The complete system of inertia forces in a structure can be determined only by evaluating the acceleration of every mass particle. The analysis can be greatly simplified if the deflections of the structure can be specified adequately by a limited number of displacement components or ordinates. The number of displacement components required to specify the positions of all significant mass particles in a structure is called the number of degrees of freedom of the structure. In the so-called lumped-mass idealization, the mass of the structure is assumed concentrated at a number of discrete locations. Because the floor and roof elements (diaphragms) in a building are relatively heavy, a large proportion of the building mass is concentrated in these elements. For structural analysis purposes, the mass of other building components such as walls and columns, and that associated with the superimposed dead loads, are normally assumed concentrated at the floor and roof levels [1.6].

A multi-degree of freedom system possesses as many natural modes of vibration as there are degrees of freedom. The distinguishing feature of a mode of vibration is that a dynamic system can, under certain circumstances, vibrate in that mode alone. During such vibration, the ratio of the displacements of any two masses remains constant with time. These ratios define the characteristic shape of the mode; the absolute amplitude of motion is arbitrary. Each natural mode shape m has a natural period of vibration T_m associated with it. The period T_m is the time required for one cycle of motion in the deflected shape characteristic of the m^{th} natural mode of vibration. The term natural is used to qualify each vibration quantity to emphasize the fact that these are natural properties of the structure, depending on its stiffness and mass, when it is allowed to vibrate freely without any external excitation. An idealized n-story building possesses n natural periods of vibration $(T_1, T_2, ... T_n)$ arranged from the largest to the smallest, corresponding to natural modes 1 through n. The longest period of the first or the fundamental mode is designated T_1. Figure 1-2 shows that any arbitrary displaced shape of a structure may be expressed in terms of the amplitudes of the mode shapes. The equation of motion of any mode m of a multi-degree system is equivalent to the equation of motion for a single-degree system.

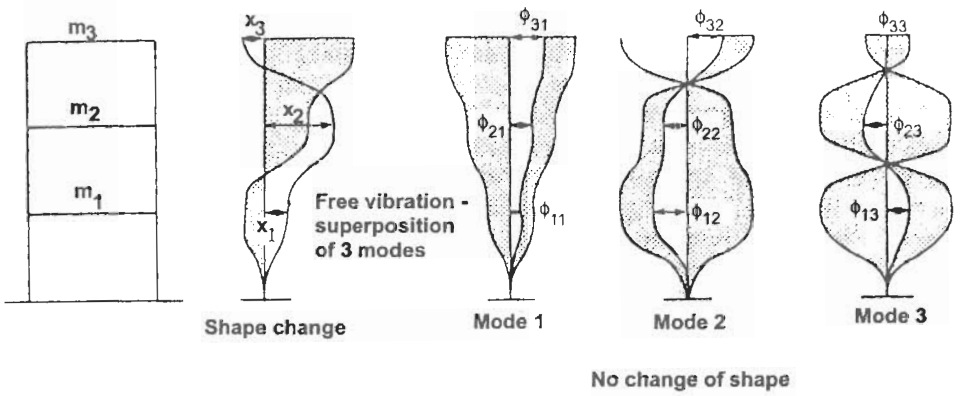


Figure 1-2 Mode Superposition Analysis of Earthquake Response

Modal periods are functions of the dynamic properties of mass, stiffness, and beyond the stage of elastic response, also strength. The seismic response of short, stiff buildings is dominated by the first or fundamental mode of response; most of the building mass vibrates in that mode. The higher modes of vibration contribute significantly to seismic response only when a building is relatively tall and flexible.

1.2.1.2 Design Philosophy

When a structure responds elastically to ground motion during a severe earthquake, the maximum response accelerations may be several times the maximum ground acceleration. It is generally uneconomical and unnecessary to design a structure to respond in the elastic range to the maximum earthquake-induced inertia forces. Thus, the design seismic horizontal forces prescribed in the 2000 IBC, as well as forces prescribed in prior model building codes and resource documents, are generally less than the elastic response inertia forces induced by the design earthquake.

It is expected that structures would undergo fairly large deformations when subjected to a major earthquake. The intent is that these large deformations will be accompanied by yielding in some of the members of the structure. It should be evident that the use of code-prescribed seismic design forces implies that critical regions of certain members should have sufficient inelastic deformability to enable the structure to survive without collapse when subjected to several cycles of loading well into the inelastic range. This means avoiding all forms of brittle failure and achieving adequate inelastic deformation via yielding of certain localized regions of certain members or connections in the structure. This is precisely why the material chapters of the IBC contain detailing and other requirements that go hand-in-hand with the code-prescribed seismic forces. For concrete structures, satisfying the design and detailing requirements in Chapter 19 of the 2000 IBC, which is essentially the requirements in Chapter 21 of ACI 318-99, provides the required levels of inelastic deformability that are inherent in the code.

Experience from recent earthquakes has shown that structures designed to the level of seismic forces prescribed by codes can survive major earthquake shaking. This is mainly due to the ability of well-designed structures to dissipate seismic energy by inelastic deformations in particular regions of certain members in the structure. Decrease in structural stiffness caused by accumulating damage and soil-structure interaction also helps at times.

Figure 1-3 shows the idealized force-displacement relationship of a structure subjected to the design earthquake of the 2000 IBC, as defined in IBC 1613. On the horizontal axis are the earthquake-induced displacements. The quantity V along the vertical axis is the code notation for design base shear, a global force quantity. The curve in the figure may be thought of as the envelope or the backbone curve of hysteretic force-displacement loops that describe the response of a structure subjected to reversed cyclic displacement histories of the type imposed by earthquake ground motion.

The bases shear V is to be distributed along the height of the structure as required by IBC 1617.4.3 (discussed below). The distribution results in a series of lateral forces concentrated at the various floor levels. Next, a mathematical model of the structure is to be elastically analyzed under these lateral forces. The quantity δ_{xe} represents the lateral displacement at floor level x obtained from this analysis, and Q_E represents the member forces (bending moments, shear forces, axial forces, etc.).

As the structure responds inelastically to the design earthquake defined in IBC 1613, the lateral displacement at floor level x increases from δ_{xe} to $C_d\delta_{xe}$, and the member forces increase from Q_E to $\Omega_o Q_E$. Both the deflection amplification factor C_d and the system overstrength coefficient Ω_o depend on the structural system used for earthquake resistance, and are given in IBC Table 1617.6. Quantities V_E and δ_{xE} are the base shear and the lateral displacement at floor level x, respectively, corresponding to the hypothetical elastic response of the structure to the design earthquake defined in IBC 1613. Figure 1-3 suggests that a response modification factor R of 2 used in design would result in an essentially elastic response of a structure to the design earthquake. The basis for this is explained in Reference 1.13.

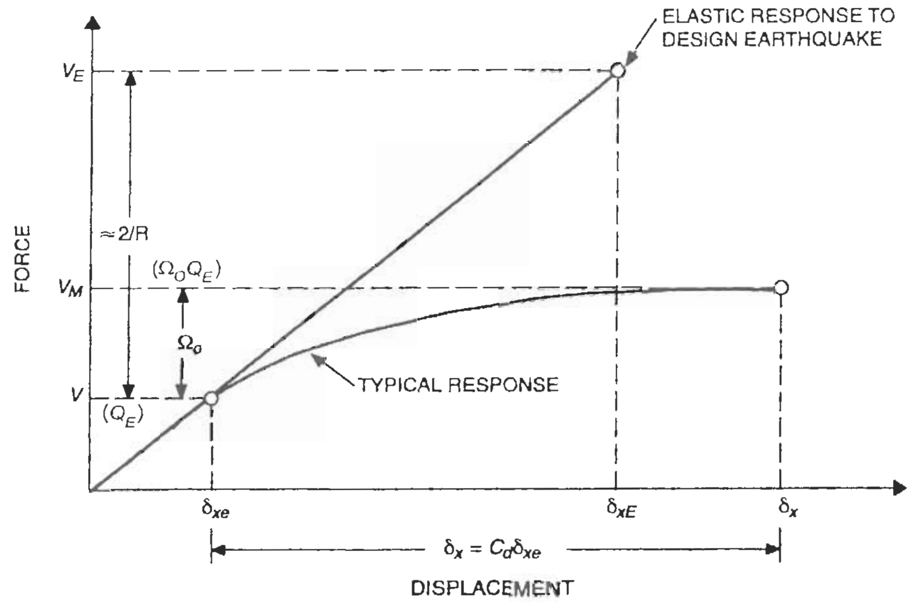


Figure 1-3 Idealized Force-Displacement Relationship of a Structure Subjected to the Design Earthquake of the 2000 IBC

1.2.1.3 Evolution of U.S. Seismic Codes

Until recently, seismic design provisions in U.S. building codes followed a certain pattern: provisions were first proposed by the Structural Engineers Association of California (SEAOC) in its *Recommended Lateral Force Requirements* [1.7], which is commonly referred to as the Blue Book, then adopted by the International Conference of Building Officials (ICBO) in the UBC [1.5]. The UBC provisions were then adopted, often with modifications, by the American National Standards Institute (ANSI) Standard A58.1 [1.8], which in turn, was adopted by the other two model codes: *The BOCA National Building Code* (BOCA/NBC) published by the Building Officials and Code Administrators International [1.9] and the *Standard Building Code* (SBC) published by the Southern Building Code Congress International [1.10]. Thus, the seismic design provisions of all three model codes were based on the provisions in the SEAOC Blue Book; a lag usually occurred in the time it took the most current Blue Book provisions to reach the model codes.

A departure from the above pattern was initiated in 1972 when the National Science Foundation and the National Bureau of Standards (now the National Institute of Standards and Technology) decided to jointly sponsor a Cooperative Program in Building

Practices for Disaster Mitigation. Under that program, the Applied Technology Council (ATC) developed a document entitled *Tentative Provisions for the Development of Seismic Regulations for Buildings* [1.11]. This document, published in 1978 and commonly referred to as ATC 3-06, underwent a thorough review by the building community in ensuing years. Trial designs were conducted to establish the technical validity of the new provisions and to assess their impact. A new entity, the Building Seismic Safety Council (BSSC), was created under the auspices of the National Institute of Building Sciences (NIBS) to administer and oversee the trial design effort. The trial designs indicated the need for certain modifications to the original ATC 3-06 document. The modifications were made, and the resulting document was the first edition, dated 1985, of the NEHRP (National Earthquake Hazards Reduction Program) *Recommended Provisions for the Development of Seismic Regulations for New Buildings* [1.12]. Under continued federal funding, this document has been updated every three years; the 1988, 1991, 1994, 1997, and 2000 editions of the NEHRP Provisions have been issued by the Building Seismic Safety Council. The 2003 edition of the NEHRP Provisions is currently under development.

In 1980, the SEAOC Seismology Committee undertook the task of developing an ATC-based revision of their Blue Book. This extensive effort resulted in the 1988 edition of the SEAOC Blue Book, which was then adopted into the 1988 edition of the UBC. Changes from the 1988 to the 1991 and from the 1991 to the 1994 editions of the UBC were minor. Major revisions to the seismic code provisions occurred between the 1994 and 1997 editions of the UBC, which were due to a code change under development since 1993 by the SEAOC Seismology Committee and its ad hoc subcommittees on strength design and strong ground motions. Numerous significant revisions were incorporated into their code change proposal to reflect the lessons learned from the Northridge and Kobe earthquakes. Also, recognizing that the IBC would be based on existing model codes, SEAOC evaluated the basis of the seismic provisions in the BOCA/NBC and the SBC to determine if convergence of the seismic provisions was possible. As a result, significant concepts consistent with the seismic provisions in these other two model codes were incorporated into SEAOC's code change proposal. These changes, sometimes in modified forms, were included in Appendix C to the 1996 SEAOC Blue Book and in the 1999 SEAOC Blue Book.

The BOCA/NBC and the SBC, which traditionally adopted the seismic provisions of the ANSI A58.1 (now ASCE 7) Standard, adopted seismic design provisions based on the 1991 NEHRP Provisions in 1993 and 1994, respectively. This is because the seismic provisions of the national loading standard had fallen behind the times. The 1988 edition of the ASCE/ANSI Standard, published in 1990, still had seismic design provisions based on the 1979 UBC. Requirements based on the 1991 NEHRP Provisions were retained in the 1996 BOCA/NBC and the 1997 SBC. The 1994 NEHRP Provisions did not become the basis of the seismic design provisions of the 1996 BOCA/NBC or the 1997 SBC. However, seismic design provisions based on the 1994 NEHRP Provisions were adopted into the 1995 edition of the ASCE 7 Standard. Both the 1996 edition of the BOCA/NBC and the 1997 edition of the SBC permit seismic design by ASCE 7-95. Seismic design provisions of the 1999 BOCA/NBC remained the same as those of the 1996 edition.

Similarly, the seismic design provisions of the 1999 SBC remained the same as those of the 1997 edition. Table 1-1 summarizes seismic design provisions of the model codes.

Table 1-1 Seismic Design Provisions of the Model Codes

Resource Document	Edition	Model Code
SEAOC Blue Book*	1988	UBC 1994
	1996 (App. C), 1999	UBC 1997
NEHRP	1991	BOCA/NBC 1993, 1996, 1999
		SBC 1994, 1997, 1999
	1994	See ASCE 7 1995 below
	1997	IBC 2000
ASCE 7	1995 (NEHRP 1994 adopted)	BOCA/NBC 1996[†], 1999[†]
		SBC 1997[†], 1999[†]

* Recommended Lateral Force Requirements

† The 1991 NEHRP Provisions form the basis of the seismic design requirements of the 1996 and 1999 editions of the BOCA/NBC and the 1997 and 1999 editions of the SBC. However, both codes allow the use of ASCE 7-95, which has adopted seismic design requirements based on the 1994 NEHRP Provisions. ASCE 7-98 and ASCE 7-02 have adopted seismic design provisions based on the 1997 and 2000 NEHRP Provisions, respectively.

The seismic design provisions of the IBC were treated separately from the rest of the structural provisions in the code development process. In 1996, the IBC Code Development Committee agreed in concept for the IBC to be based on the 1997 edition of the NEHRP Provisions, which was being developed at the time the last edition of the UBC (1997) was published. A Code Resource Development Committee (CRDC), funded by the Federal Emergency Management Agency (FEMA), was formed under the auspices of the Building Seismic Safety Council (BSSC) to generate seismic code provisions based on the 1997 edition of the NEHRP Provisions, for incorporation into the 2000 IBC. This effort was successful and the CRDC submittal was accepted by the IBC Code Development Committee for inclusion in the IBC. The seismic design provisions of the Working Draft were based on the 1997 NEHRP Provisions, but also included a number of features of the 1997 UBC that were not included in the 1997 NEHRP Provisions. Many changes were made to the provisions of the Working Draft through a public forum and two sets of public hearings. BSSC's Code Resource Support Committee (CRSC), a successor group to the CRDC, played an active role in this development by sponsoring changes of their own and by taking positions on other submitted changes—positions that were carefully considered by the IBC Structural Subcommittee at the public forum and at the public hearings. The seismic design provisions of the First Edition of the IBC remain based on the 1997 NEHRP Provisions, with some of the features of the 1997 UBC also included.

The changes from the 1994 to the 1997 edition of the UBC took it in the direction of the 1997 NEHRP and the IBC. However, there are still substantial differences between the seismic design provisions of the 1997 UBC and those of the 2000 IBC. The treatment of ground motion parameters in seismic design is entirely different in the two documents. Another major difference is discussed below.

The 2000 IBC uses Seismic Design Categories to determine permissible structural systems, limitations on height and irregularity, the type of lateral force analysis that must be performed, the level of detailing for structural members and joints that are part of the lateral-force-resisting system and for the components that are not. The 1997 UBC, as in prior editions of the code, uses the seismic zone in which a structure is located for all these purposes. Seismic Design Category (SDC) is a function of occupancy (called Seismic Use Group in the 2000 IBC and the 1997 NEHRP Provisions) and of soil-modified seismic risk at the site of the structure.

In 1978, when ATC 3-06, the predecessor document to the NEHRP Provisions, made the level of detailing (and other restrictions concerning permissible structural systems, height, irregularity, and analysis procedure) a function of occupancy, that was a major departure from UBC practice, which was continued in all the NEHRP Provisions through the 1994 edition. Now, in the 2000 IBC and the 1997 NEHRP Provisions, the level of detailing and the other restrictions have been made a function of the soil characteristics at the site of the structure. This is a further major departure from UBC practice and indeed from current seismic design practice across the U.S.

1.2.1.4 Evolution of Seismic Design Criteria

Seismic Zones.

Until relatively recently, seismic design criteria in building codes depended solely upon the seismic zone in which a structure is located. This is still the situation with the last edition, dated 1997, of the UBC. Zones are regions in which seismic ground motion on rock, corresponding to a certain probability of occurrence, is within certain ranges. Under the UBC, the United States is divided into Seismic Zones 0 through 4, with 0 indicating the weakest earthquake ground motion, and 4 indicating the strongest. The level of seismic detailing (including the amount of reinforcement) for concrete structures is then indexed to the Seismic Zone. Also indexed to the seismic zone are height limits on structural systems, minimum requirements concerning analytical procedure that must form the basis of seismic design, and other restrictions/limitations/requirements.

Seismic Performance Categories.

Given that public safety is a primary code objective, and that not all buildings in a given seismic zone are equally crucial to public safety, a new mechanism for triggering restrictions and requirements, called the Seismic Performance Category (SPC), was developed. The SPC classification includes not only the seismicity at the site, but also the occupancy of the structure. The SPC, rather than the Seismic Zone, is the determinant of seismic design and detailing requirements in the last three editions of the BOCA/NBC and the SBC, thereby dictating that the seismic design requirements for a hospital be more restrictive than those for an office building constructed on the same site. The detailing requirements under Seismic Performance Categories A & B, C, and D & E are roughly equivalent to those for Seismic Zones 0 & 1, 2, and 3 & 4, respectively.

Seismic Design Categories.

The most recent development in the mechanism for triggering seismic design requirements and restrictions has been the establishment of Seismic Design Categories. Recognizing that building performance during a seismic event depends not only on the severity of sub-surface rock motion, but also on the type of soil upon which a structure is founded, the SDC is a function of location, building occupancy, and soil type.

Table 1-2 contains a summary of the seismic risk levels, Seismic Performance Categories, and Seismic Design Categories specified in the model building codes and other resource documents.

Table 1-2 Seismic Risk Terminology

<table>
<tr><th rowspan="2">Code, Standard, or Resource Document</th><th colspan="3">Level of Seismic Risk or Assigned Seismic Performance Category (SPC) or Seismic Design Category (SDC)</th></tr>
<tr><th>Low</th><th>Moderate</th><th>High</th></tr>
<tr><td>BOCA National Building Code (1993, 1996, 1999)</td><td rowspan="4">SPC A, B</td><td rowspan="4">SPC C</td><td rowspan="4">SPC D, E</td></tr>
<tr><td>Standard Building Code (1994, 1997, 1999)</td></tr>
<tr><td>ASCE 7-93, 7-95</td></tr>
<tr><td>NEHRP (1991, 1994)</td></tr>
<tr><td>Uniform Building Code (1991, 1994, 1997)</td><td>Seismic Zone 0, 1</td><td>Seismic Zone 2</td><td>Seismic Zone 3, 4</td></tr>
<tr><td>International Building Code (2000)</td><td rowspan="4">SDC A, B</td><td rowspan="4">SDC C</td><td rowspan="4">SDC D, E, F</td></tr>
<tr><td>ASCE 7-98, 7-02</td></tr>
<tr><td>NEHRP (1997)</td></tr>
<tr><td>NFPA 5000 (2003)</td></tr>
</table>

1.2.2 Response of Concrete Buildings to Seismic Forces

A well-designed and well-constructed building has a reliable load path that transfers lateral forces through the structure to the foundation where the soil can resist them. Horizontal earthquake forces are usually resisted by either walls or frame elements. At the base of wall and frame elements, foundation components transfer the earthquake forces to the earth.

The diaphragms, walls, frames, and foundations of a building are the key elements in the load path through the structure. The connections between these elements are also important components of the chain that makes up the horizontal and vertical load paths that transfer the horizontal forces. The earthquake resistance of a building is only as strong as the weakest link in the load path.

The following discussion of structural elements along the load path is largely based on a very lucid treatment of the same topic in the ATC-48 seminar manual [1.6].

1.2.2.1 Diaphragm Response

Floor and roof diaphragms in a concrete building typically span between shear walls of cast-in-place or precast concrete. When subjected to lateral forces caused by earthquakes, diaphragms respond like deep beams bending in their own plane. Figure 1-4 illustrates the two basic types of forces produced at the diaphragm edges: shear and tension or compression.

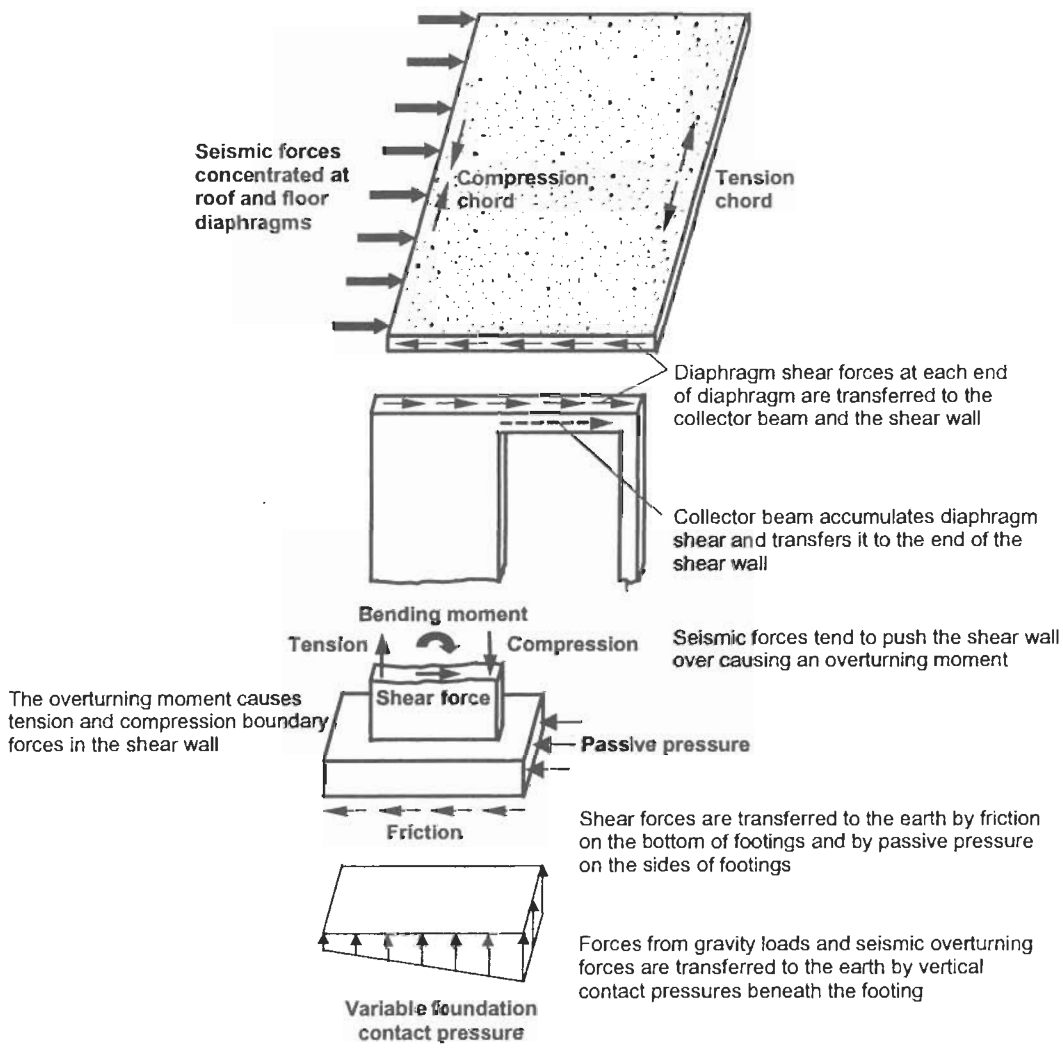


Figure 1-4 Load Path of Seismic Forces in a Concrete Building

Seismic forces acting in the direction perpendicular to the long side of the diaphragm, as shown in Figure 1-4, produce shear forces acting in the opposite direction at each end of the diaphragm. These shear forces are transferred to shear walls alone or to collector elements (also called "drag struts") and shear walls (or to vertical frames) at those locations. On the side of the diaphragm opposite the seismic forces, tension develops in the chord, and compression develops on the side on which the seismic forces are acting. Reverse situations apply upon reversal of the seismic forces. When seismic forces act in the direction perpendicular to the short side of the diaphragm, chord forces (tension and compression) develop along the short sides of the diaphragms, and shear forces act along the long sides.

Diaphragms are typically classified into two broad groups for design purposes. When the in-plane deflection of a diaphragm at a particular floor level is more than twice the average lateral deflections of the supporting vertical elements (shear walls or frames) at the same level with respect to the next lower level, it is classified as flexible by seismic codes of the present and the recent past. If that is not the case, a diaphragm is classified as rigid.

Floors and roofs constructed of reinforced concrete or concrete fill on metal deck are almost invariably rigid diaphragms (unless very large openings are made in such diaphragms). The concrete is effective in resisting shear, as long as it is reinforced. The reinforcement may be in the form of reinforcing bars laid out in both directions at uniform spacing. Welded wire fabric is a much more common alternative. In many cases, a sufficient amount of longitudinal reinforcement may be present along the edges of concrete diaphragms, with or without the presence of beams, to resist the chord forces. If that is not the case, extra chord reinforcement must be added.

Forces similar to chord forces also can develop around openings in both rigid and flexible diaphragms. The edges of these openings may need to be reinforced with additional longitudinal steel in concrete diaphragms.

If a diaphragm is flexible, lateral forces transmitted by the diaphragm are distributed to vertical lateral-force-resisting elements (shear walls, frames) in proportion to their tributary areas. If a diaphragm is rigid, lateral forces are distributed according to the relative lateral stiffnesses of the vertical lateral-force-resisting elements. Stiffness is the force required to cause a unit displacement.

Diaphragm Force Transfer.

Shear forces at diaphragm edges must be transferred to the supporting walls or frame elements. One mechanism for such shear transfer is known as shear-friction. It is assumed that the diaphragm cannot slide with respect to the wall or frame element without yielding of the reinforcing steel that crosses the interface between the two. The shear strength is a function of the amount of steel crossing the interface and the roughness of the interface. Another mechanism for transferring shear is the so-called dowel action. This assumes that shear is transferred from the diaphragm into the wall or frame element through the reinforcement acting in shear like an anchor bolt. In some cases, forces can be large enough to require that the construction joint itself have deformations to provide dowel action.

Out-of-Plane Forces.

In an earthquake, out-of-plane forces act perpendicularly to the plane of a wall or frame (which itself is perpendicular to the direction of ground motion), causing direct tension or compression across its connections to diaphragms. Out-of-plane forces are usually not a problem for rigid diaphragms, but they have caused serious problems in past earthquakes for flexible diaphragms. This makes it important to pay close attention to the connections between diaphragms and wall or frame elements. No matter how strong or stiff a diaphragm and its supporting elements may be, the lateral-force-resisting elements cannot be mobilized if the connection between the two fails.

Collector Elements.

A supporting shear wall may not extend along the entire edge of a diaphragm. In such cases, the shear forces from the unsupported portion of the diaphragm must be transmitted to the shear walls. A collector element or drag strut, which is usually a beam, "collects" these forces and transfers them to the end of the shear wall. Collector elements must be designed for the tension and compression forces they collect. Additional reinforcement may have to be placed in collectors supporting concrete diaphragms, particularly where they frame into the wall.

Collector elements are often the same members that are used as the chords for seismic forces in the perpendicular direction. The required amount of drag reinforcement needs to be compared to the required amount of chord reinforcement, and the larger of the two should be supplied along that edge.

1.2.2.2 Seismic Response of Shear Walls

Shear walls are elements that resist, in addition to gravity loads, in-plane (as distinct from out-of-plane) lateral forces. They are like vertical cantilever beams and are typically quite deep. Once the shear force along the edge of a diaphragm is transferred into a shear wall (see Figure 1-4), it causes bending moment as well as shear force in the plane of the wall. The tendency of a shear wall to overturn and slide under such bending moment and shear force is resisted by the foundation to which these internal forces must be transmitted. The bending moment, often called overturning moment, increases from the top to the bottom of a building and causes tension and compression forces in the plane of the wall, which are the largest along the vertical edges at the ends of the wall.

The seismic response of short stocky shear walls (height to horizontal length ratio of one or less) is typically governed by shear (bending strength being typically in excess of that required). The response of taller, more flexible shear walls (height to horizontal length ratio of two or more) is, on the other hand, typically governed by flexure (shear strength being in excess of that required). In the range of height to horizontal length ratios between one and two, whether response would be shear- or flexure-governed depends on a number of factors, including the amount of horizontal shear reinforcement. Shear-dominated response is characterized by inclined, often x-shaped, cracking patterns. In the absence of adequate reinforcement, shear-controlled walls can lose strength quite rapidly, leading to failure with little warning. Flexure-dominated walls exhibit horizontal cracking, starting at or near the base. These typically do not fail abruptly.

1.2.2.3 Seismic Response of Frames

Frames respond differently from shear walls to lateral forces. Frames consist of vertical elements (columns) and horizontal elements (beams or slabs). A frame resists being deformed by lateral forces due to the rigidity of the beam-column joints. Because of this rigidity (the tendency of a joint to retain the initial 90-degree angles between beams and columns meeting at the joint), the beams and the columns bend as shown in Figure 1-5. The tension stresses caused by the bending must be resisted by reinforcement. The bending also causes vertical shear forces in beams and horizontal shear forces in columns. Vertical shear reinforcement (hoops or stirrups) is therefore needed in beams, and horizontal shear reinforcement in columns.

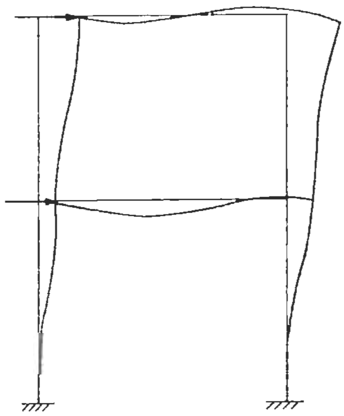


Figure 1-5 Response of Frame to Seismic Forces

1.2.2.4 Foundation Response

This section largely reproduces the excellent, concise text on this topic in ATC-48 Briefing Paper 4, Part C [1.6].

Foundations can be shallow or deep. Shallow foundations are supported by the vertical pressure of the earth directly below. They can be square or rectangular conventional spread footings supporting individual columns, or continuous, relatively narrow, rectangular (strip) footings supporting walls or frames. Deep foundations consist of wood, steel, or concrete piles driven into the ground, or concrete piers that have been drilled and poured in place. These components are supported vertically by end bearing and by skin friction along their length. Foundation elements are often connected together by ties, grade beams, or slabs on grade.

Shear forces are transferred to the foundation from walls and frames (see Figure 1-4). Dowels from footings or pile caps must match in total cross-section both boundary reinforcement in walls and flexural reinforcement in columns, to resist overturning. In fact, all vertical reinforcement in walls and frames must be matched by foundation dowels.

1.2.3 Seismic Design Requirements of the 2000 IBC

The seismic design provisions in the 2000 IBC are contained in sections 1613 through 1623. The following discussion focuses on IBC 1613 through 1617. IBC 1617 contains the provisions of the Equivalent Lateral Force Procedure. Requirements of seismic design using structural dynamics are contained in IBC 1618 and are not covered in this publication. Readers are referred to [1.14] for a comprehensive coverage of this topic.

1.2.3.1 General Requirements

IBC 1614.1 requires that every structure and it components be designed for the effects of earthquake motions and be assigned a Seismic Design Category (SDC). However, it does permit a number of important exceptions. Structures in SDC A need only comply with IBC 1616.4 (note: a discussion on how to determine the SDC of a structure is given below). For structures exposed to such a low seismic risk, that section requires only that a complete lateral-force-resisting system be provided and that all elements of the structure be tied together. The lateral-force-resisting system must be proportioned to resist a lateral force at every floor level equal to 1 percent of the weight at that floor level as depicted in Figure 1-6 (IBC 1616.4.1).

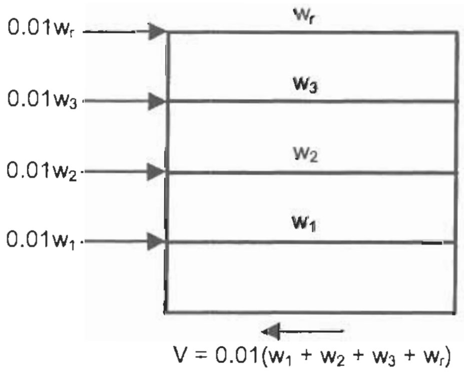


Figure 1-6 Design Seismic Force Distribution for Structures Assigned to SDC A

The other exceptions in IBC 1614.1 are:

1. Detached one- and two-family dwellings in SDC A, B, and C are totally exempt from all seismic design requirements.
2. The seismic-force-resisting system of wood frame buildings that meet the limitations of IBC 2308 (conventional light frame construction) need only conform to that section. Although some control is necessary to ensure the integrity of these structures, it is felt that the requirements of IBC 2308 are

adequate to provide the safety required based on the history of such light-frame construction – invariably low-rise structures – in earthquakes.

3. Agricultural storage structures intended only for incidental human occupancy are exempt from all seismic design requirements because of the exceptionally low risk to life involved.
4. Structures located where the mapped spectral response acceleration at short periods (discussed below), $S_S \leq 0.15g$ and the mapped spectral response acceleration at 1 second period (also discussed below), $S_1 \leq 0.04g$ need only comply with IBC 1616.4.
5. Structures located where the short period design spectral response acceleration (discussed below), $S_{DS} \leq 0.167g$ and the design spectral response acceleration at 1 second period (also discussed below), $S_{D1} \leq 0.067g$ need only comply with IBC 1616.4. In view of the definitions of Seismic Design Categories in IBC Tables 1616.3(1) and 1616.3(2), this is a repeat of the SDC A exemption.

Exception 4 is a particularly important one. Areas with $S_S \leq 0.15g$ and $S_1 \leq 0.04g$ are darkened on the map of the United States in Figure 1-7. The darkened areas are, in a sense, analogous to Zone 0 of the UBC.

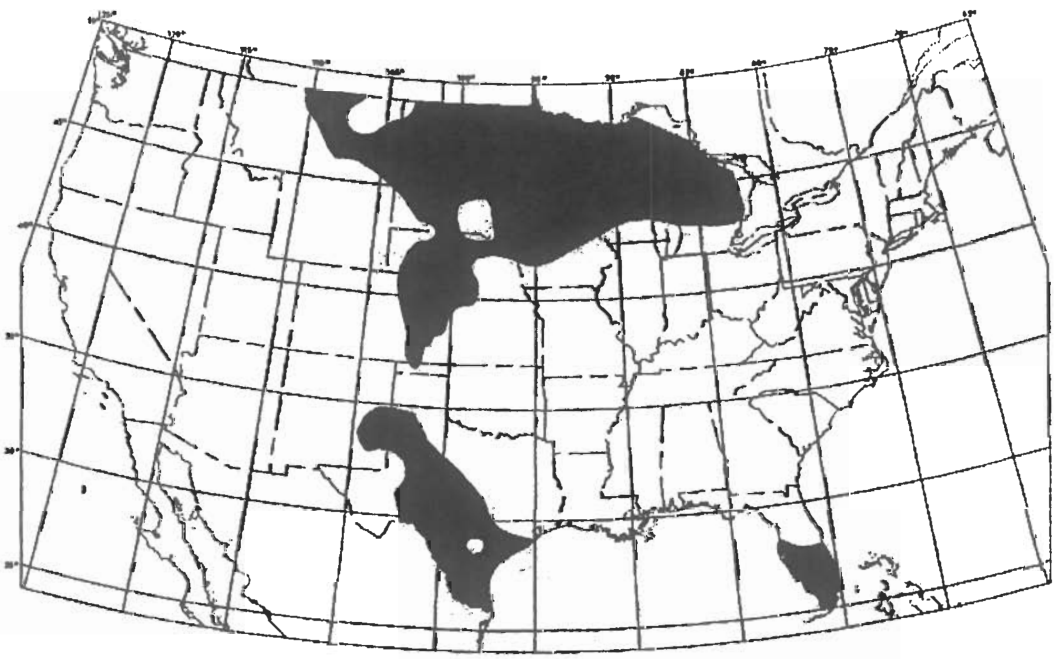


Figure 1-7 Areas of the U.S. with $S_S \leq 0.15g$ and $S_1 \leq 0.04g$

Also contained in this section are requirements for additions, change of occupancy, and alterations to existing buildings (IBC 1614.1.1, 1614.2, and 1614.3). IBC 1614.5 is also important: this section requires that appropriate seismic detailing requirements and limitations are to be satisfied even though wind forces govern the design.

1.2.3.2 Site Ground Motion

IBC 1615.1 defines a Maximum Considered Earthquake (MCE) which, for most of the United States, is an earthquake with an approximately 2500-year return period (2% probability of exceedance in 50 years). However, in coastal California, the MCE is the largest (deterministic) earthquake that can be generated by the known seismic sources.

IBC Figures 1615(1) and 1615 (2) contain contour maps of the conterminous U.S. giving S_S and S_1, respectively. The quantities S_S and S_1 are the mapped MCE spectral response accelerations at periods of 0.2 sec and 1.0 sec, respectively, on Site Class B. For the concept of the spectral acceleration, refer to [1.14]. Spectral acceleration is directly related to base shear (base shear equals spectral acceleration times mass or spectral acceleration divided by g times weight, where g is the acceleration due to gravity). Figures 1615(3) through 1615(10) contain similar contour maps for specific regions of the conterminous U.S., Alaska, Hawaii, Puerto Rico, and U.S. commonwealths and territories. Straight-line interpolation or the value of the higher contour may be used where a site is between contours (IBC 1615.1).

Site Class B is one of six Site Classes defined by IBC Table 1615.1.1, which have replaced the old four-tier soil classification (S_1 through S_4); the old classifications are still in the BOCA/NBC and the SBC and were in the 1994 UBC. Site classification is now based on one of three measured properties of the soils in the top 100 ft of the site: shear wave velocity $\bar{v}_s$, standard penetration resistance (or blow count) $\bar{N}$ (ASTM D 1586-84), or undrained shear strength $\bar{s}_u$ (ASTM D 2166-91 or D 2850-87). Where site-specific data are not available to a depth of 100 feet, appropriate soil properties are permitted to be estimated by the registered design professional preparing the soils report based on known geologic conditions.

When the soil properties are not known in sufficient detail to determine the site class, Site Class D must be used unless the building official determines that Site Class E or F soil is likely to be present at the site.

At the beginning of seismic design, S_S and S_1 must be determined for the site—either by interpolating between contours on the maps included in the IBC, or by using a CD-ROM, prepared by the United States Geological Survey (USGS) and distributed with the IBC [1.15]. The program on the CD-ROM accepts as input either the zip code of the location or the latitude and the longitude (the latter is obviously more precise) and provides S_S and S_1 as output.

Once S_S, S_1, and the Site Class at a site have been determined, a short-period or acceleration-dependent site coefficient, F_a and a long-period or velocity-dependent site coefficient, F_v must be determined from IBC Tables 1615.1.2(1) and 1615.1.2(2), respectively. These coefficients replace the earlier soil factor S (UBC through 1994, BOCA/NBC through 1999, SBC through 1999), which was a function solely of the soil at the site. By contrast, F_a is a function of the Site Class and S_S, while F_v is a function of the Site Class and S_1. This dependence of the site coefficients on the seismicity at the site has created a much more elaborate scheme than before: now there is a row of values of each site coefficient for a particular Site Class. It should be noted that the values of F_a and F_v are higher in areas of low seismicity, and lower in areas of high seismicity. This is in line with observations that weak sub-surface rock motion is amplified to a much larger extent by overlying soft soil deposits than is strong sub-surface rock motion. In areas of high seismicity, short-period sub-surface rock motion may even be de-amplified by overlying soils when they are very soft, as illustrated by a value of 0.9 for F_a where $S_S = 1.0$ for site Class E. Finally, whereas the old S-factor had a maximum value of 2 for Type S_4 (very soft) soil, the new site coefficient F_a has a maximum value of 2.5 and F_v has a maximum value of 3.5. Thus, for taller buildings on softer soils in areas of low seismicity, the design base shear has gone up by up to 75% from the prior model codes (not including the 1997 UBC) to the 2000 IBC.

Having determined F_a and F_v, S_S must be modified by F_a, resulting in $S_{MS} = F_a S_S$, the soil-modified MCE spectral response acceleration at short periods (IBC Eq. 16-16). Likewise, S_1 must be modified by F_v, resulting in $S_{M1} = F_v S_1$, the soil-modified MCE spectral response acceleration at 1 second period (IBC Eq. 16-17).

For design purposes, two-thirds of these soil-modified MCE values are used. The five-percent damped design spectral response acceleration at short periods, S_{DS}, and at 1 second period S_{D1}, are determined from IBC Eqs. 16-18 and 16-19, respectively:

$$S_{DS} = \frac{2}{3} S_{MS}$$

$$S_{D1} = \frac{2}{3} S_{M1}$$

Note that 2/3 is the reciprocal of 1.5, the lower-bound margin of safety that is acknowledged to have been built into seismic design by model codes of the recent past. In other words, a code-designed structure is thought highly unlikely to collapse under ground motion that is one-and-one-half times as strong as the design earthquake ground motion. The design earthquake of all codes of the recent past was the 500-year return period earthquake (10% probability of non-exceedance in 50 years). In coastal California where MCE ground motion is no more than 150% as strong as ground motion in the 500-

year return period earthquake, collapse in the MCE was prevented by code-based design. In the Midwest and the East, where MCE ground motion can be four to five times as strong as ground motion in the 500-year return period earthquake, safety against collapse in the MCE was not ensured by code-based design. The design approach of the IBC ensures prevention of collapse of structures across the United States, because all structures must be designed for two-thirds of the MCE ground motion and because a margin of safety of at least one and a half is still built in.

A general procedure for determining a response spectrum (to be more exact, a design spectrum) is contained in IBC 1615.1.4. This section provides a method of obtaining a 5-percent damped response spectrum from S_{DS} and S_{D1}. The response spectrum consists of a series of two curves representing the short-period range (a region of constant spectral response acceleration) and the long-period range (a region of constant spectral response velocity). A very long period range, which is a region of constant spectral response displacement, is not included, since relatively few structures have a period long enough to fall into this range. IBC Figure 1615.1.4 depicts the design response spectrum based on the provisions in IBC 1615.1.4.

A step-by-step procedure for classifying a site is contained in IBC 1615.1.5. The definitions of average soil properties in IBC 1615.1.5 and the steps for classifying a site in IBC 1615.1.5.1 first appeared in the 1994 NEHRP Provisions, were adopted virtually unmodified into the 1997 UBC, were retained in the 1997 NEHRP Provisions, and were adopted into the 2000 IBC. Table 1615.1.5, which was part of the 1994 NEHRP Provisions, is really a part of IBC Table 1615.1.1, except that Table 1615.1.1 does not include the note below Table 1615.1.5.

IBC 1615.2 contains a site-specific procedure for determining ground motion accelerations. Five specific aspects must be accounted for in the investigation: (1) regional seismicity and geology, (2) expected recurrence rates and maximum magnitudes of events on known source zones, (3) location of the site with respect to these source zones, (4) near source effects, if any, and (5) subsurface site characteristics and conditions. IBC 1615.2.1 defines a probabilistic, site-specific MCE acceleration-response spectrum, and IBC 1615.2.3 defines a deterministic, site-specific MCE response spectrum. The site-specific MCE ground motion spectrum is required to be the lesser of these two, subject to the minimum of the deterministic limit ground motion of IBC 1615.2.2 (see IBC Figure 1615.2.2).

1.2.3.3 Seismic Design Category

According to IBC 1616.1, all structures shall be assigned to a Seismic Design Category (SDC) in accordance with IBC 1616.3. As mentioned previously, an SDC is used in the 2000 IBC to determine permissible structural systems, limitations on structure height and irregularity, the components of the structure that must be designed for seismic resistance, and the types of lateral force analysis that must be performed. Note that an SDC need not be determined for those structures for which earthquakes effects need not be considered (see discussion above and the exceptions in IBC 1614.1).

In general, the SDC is a function of occupancy or use, which is related to the Seismic Use Group (SUG) defined in IBC 1616.2, and the soil-modified risk at the site of the structure in the form of S_{DS} and S_{D1}. Both the SUG and the design spectral-response accelerations must be determined before an SDC can be established.

Descriptions of the four SUGs are contained in IBC Table 1604.5. Note that for purposes of IBC 1616.2, "Category," as it appears in this table, is equivalent to "Seismic Use Group." SUG I is comprised of Categories I an IV of Table 1604.5. Category I contains all occupancies other than those in Categories II, III, and IV; these occupancies are sometime referred to as ordinary for the purpose of risk exposure. Category IV contains buildings and other structures that represent a low hazard to human life in the event of failure.

SUG II is the same as Category II of Table 1604.5, except that the building official is permitted to assign a structure to this group at his or her discretion. This group pertains to buildings and other structures that represent a substantial hazard in the event of failure. Buildings and structures with large occupancies (assembly halls, schools, jails, etc.) fall into this SUG.

SUG III is the same as Category III of Table 1604.5; like SUG II, a structure may be assigned to this group at the discretion of the building official. Buildings and other structures that are required for post-earthquake response and recovery (essential facilities) such as hospitals, fire and police stations, and power-generating stations, as well as structures containing substantial amounts of hazardous substances as indicated in Table 1604.5, fall into this group.

For structures having two or more occupancies not included in the same SUG, the highest SUG corresponding to the various occupancies is to be assigned to the structure (IBC 1616.2.4).

IBC Table 1604.5 also contains the seismic importance factor I_E for each SUG. An importance factor has long been required in seismic design by the UBC. The magnitude of I_E is larger for high occupancy structures and essential facilities. As shown below, design seismic forces are directly proportional to I_E. The increased forces were intended to increase the likelihood that structures belonging to high occupancy and essential categories would continue to function during and following an earthquake.

Once the SUG and design spectral response accelerations have been determined, the SDC is determined twice: first as a function of S_{DS} by Table 1616.3(1) and second as a function S_{D1} by Table 1616.3(2). According to IBC 1616.3, the more severe category of the two governs.

Table 1-3 contains information on the applicability of the six SDCs of the 2000 IBC. Reference 1.16 contains a lucid step-by-step procedure to determine the SDC for any structure.

Table 1-3 Seismic Design Categories of the 2000 IBC

Seismic Design Category (SDC)	Seismic Use Group (SUG)	Description
A	All	Structures of any occupancy where anticipated ground motions are minor, even for very long return periods.
B	I, II	Structures in regions where moderately destructive ground shaking is anticipated.
C	III	Structures in regions where moderately destructive ground shaking may occur.
	I, II	Structures in regions with somewhat more severe ground shaking potential.
D	I, II, III	Structures in regions expected to experience destructive ground shaking, but not located close to major active faults.
E	I, II	Structures in regions located close to major active faults.
F	III	Structures in regions located close to major active faults.

Except for one- and two-family dwellings of light frame construction, structures assigned to SDC E or F are not allowed to be sited over an identified active fault. It is not possible to reliably design a structure to resist the very large forces it may be subjected to if an active fault were to cause rupture of the ground surface at its location.

1.2.3.4 Building Configuration

Past earthquakes have repeatedly shown that buildings having irregular configurations in plan and/or elevation suffer greater damage than those having regular configurations. In irregular structures, inelastic behavior can concentrate in certain localized regions, resulting in rapid deterioration of structural elements in these areas. In contrast, inelastic demands tend to be more well distributed throughout a regular structure. Furthermore, elastic analysis methods typically used to analyze a structure are not capable of accurately predicting the distribution of earthquake demands in an irregular structure. The requirements in IBC 1616.5 encourage the use of buildings with regular configurations and prohibit the use of highly irregular buildings located on sites close to active faults.

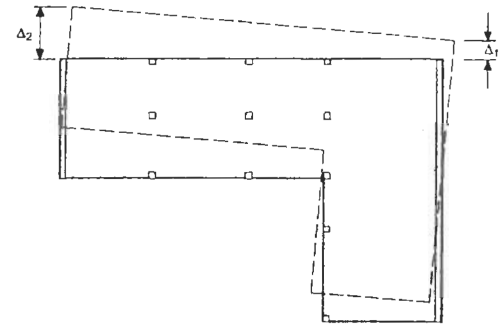
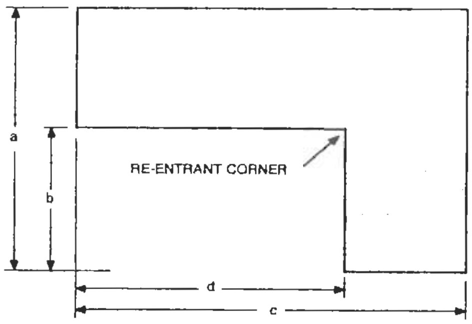
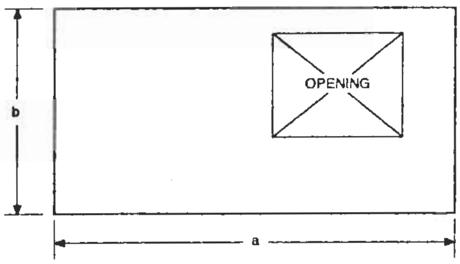
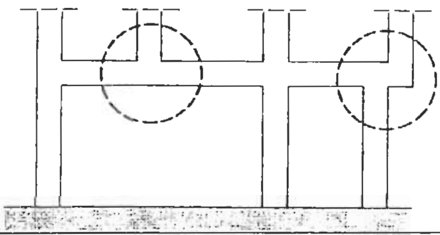
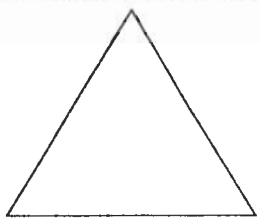
Plan Irregularities.

Five different plan structural irregularities are contained in IBC Table 1616.5.1 (see Table 1-4):

- Torsional Irregularity
- Re-entrant Corners
- Diaphragm Discontinuity
- Out-of-plane Offsets
- Nonparallel Systems

Structures having plan irregularity as defined in Table 1616.5.1 must comply with the requirements of the applicable sections referenced in the table.

Table 1-4 Plan Structural Irregularities

	Irregularity Type		Description
1a	Torsional Irregularity		$\Delta_2 > 1.2\left(\frac{\Delta_1 + \Delta_2}{2}\right)$
1b	Extreme Torsional Irregularity		$\Delta_2 > 1.4\left(\frac{\Delta_1 + \Delta_2}{2}\right)$
2	Re-entrant Corners		• Projection b > 0.15a and • Projection d > 0.15c
3	Diaphragm Discontinuity		• Area of opening > 0.5ab or • Changes in effective diaphragm stiffness > 50% from one story to the next
4	Out-of-Plane Offsets		Discontinuities in lateral-force-resisting path, such as out-of-plane offsets of vertical elements
5	Nonparallel Systems		Vertical lateral-force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral-force-resisting system

Vertical Irregularities.

Five different types of vertical structural irregularities are contained in IBC Table 1616.5.2 (see Table 1-5):

- Stiffness Irregularity—Soft Story
- Weight (Mass) Irregularity
- Vertical Geometric Irregularity
- In-Plane Discontinuity in Vertical Lateral-Force-Resisting Elements
- Discontinuity in Capacity—Weak Story

Vertical configuration irregularities affect seismic response at various floor levels and induce forces at these levels that depart significantly from the distribution assumed in the equivalent lateral force procedure in IBC 1617.4, which is discussed below.

Structures having vertical irregularity as defined in Table 1616.5.2 must comply with the requirements of the applicable sections referenced in the table.

1.2.3.5 Analysis Procedures

Except for structures assigned to SDC A, a structural analysis must be carried out for all structures subjected to seismic forces in accordance with IBC 1616.6. This analysis forms the basis for determining the internal forces in the members of the structure due to seismic effects that are used in the load combinations given in IBC 1605. It also forms the basis for determining the design drift required by IBC 1617.3. The following is a summary of the analysis procedures contained in the 2000 IBC.

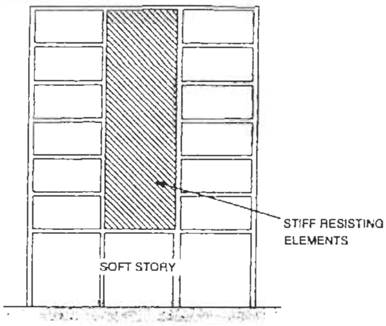
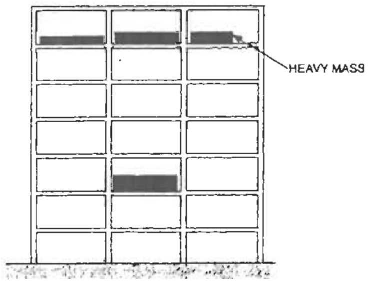
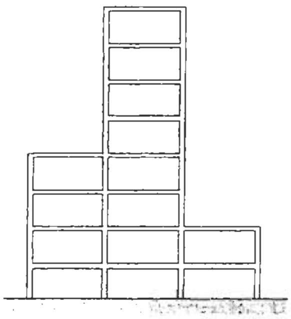
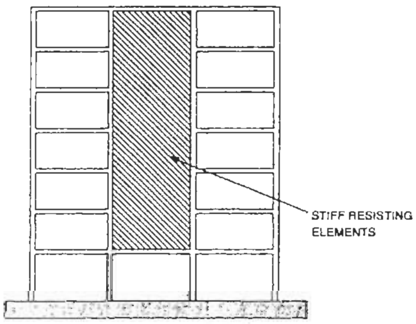
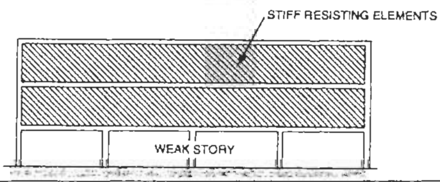
Simplified Analysis.

The simplified method in IBC 1617.5 is applicable to SUG I structures assigned to SDC B through F that are up to 2 stories in height when constructed of any material and up to 3 stories in height when light frame construction is utilized. The method is thus applicable only to short-period structures. The seismic base shear V is determined in accordance with IBC Eq. 16-49:

$$V = \frac{1.2S_{DS}}{R}W$$

where R is the response modification factor contained in IBC Table 1617.6 for seismic-force-resisting systems and W is the effective seismic weight of the structure, which is defined in IBC 1617.5.1. A complete discussion on these 2 quantities is given in the Equivalent Lateral Force Procedure section below.

Table 1-5 Vertical Structural Irregularities

	Irregularity Type		Description
1a	Stiffness Irregularity—Soft Story		Soft story stiffness < 70%(story stiffness above) or < 80%(average stiffness of 3 stories above)
1b	Stiffness Irregularity—Extreme Soft Story		Soft story stiffness < 60%(story stiffness above) or < 70%(average stiffness of 3 stories above)
2	Weight (Mass) Irregularity		Story mass > 150%(adjacent story mass) (a roof that is lighter than the floor below need not be considered)
3	Vertical Geometric Irregularity		Horizontal dimension of lateral-force-resisting system in story > 130% of that in adjacent story
4	In-Plane Discontinuity in Vertical Lateral-Force-Resisting Elements		In-plane offset of lateral-force-resisting elements > lengths of those elements, or reduction in stiffness of resisting elements in story below
5	Discontinuity in Capacity—Weak Story		• Weak story strength < 80%(story strength above) • Story strength = total strength of seismic-force-resisting elements sharing story shear for direction under consideration

The vertical distribution of the base shear is determined from IBC Eq. 16-50:

$$F_x = \frac{1.2S_{DS}}{R} w_x$$

where F_x is the seismic force applied at level x and w_x is the portion of the effective seismic weight of the structure W at level x. The vertical force distribution of the base shear V is depicted in Figure 1-8.

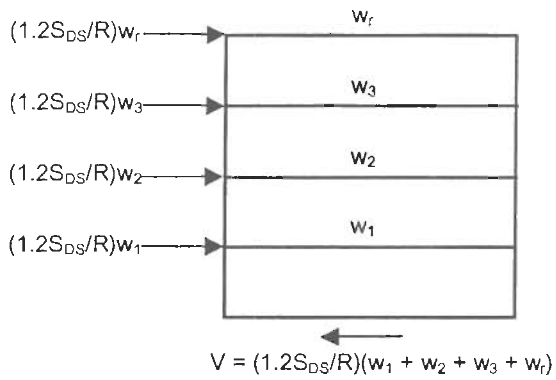


Figure 1-8 Design Seismic Force Distribution According to the Simplified Method in IBC 1617.5

According to the second exception in IBC 1616.6, design drift need not be evaluated in accordance with IBC 1617.3 when the simplified analysis method is used. Instead, unless a more exact analysis is utilized, the design story drift may be taken as 1 percent of the story height (IBC 1617.5.3).

Equivalent Lateral Force Procedure.

The Equivalent Lateral Force Procedure (ELFP) is contained in IBC 1617.4. This analysis procedure can be used for all structures assigned to SDC B and C (IBC 1616.6.2) as well as for some types of structure assigned to SDC D, E, and F (see IBC Table 1616.6.3 for the analysis procedures that are to be utilized for SDC D, E, or F).

Design Base Shear, *V*. In a given direction, the design base shear V is determined from IBC Eq. 16-34:

$$V = C_s W$$

where C_s is the seismic response coefficient determined in accordance with IBC 1617.4.1.1 and W is the effective weight of the structure, which includes the total dead load and the following other loads (IBC 1617.4.1):

- In areas used for storage, a minimum of 25 percent of the reduced floor live load (floor live load in public garages and open parking garages need not be included).
- When an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 10 psf of floor area, whichever is greater.
- Total operating weight of permanent equipment.
- Twenty percent of flat roof snow load where flat roof snow load exceeds 30 psf.

Seismic Response Coefficient, C_s. The seismic response coefficient C_s is determined from IBC Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T}$$

where S_{D1} = design spectral response acceleration at 1 second period
R = response modification factor determined from IBC Table 1617.6
I_E = occupancy importance factor determined from IBC Table 1604.5
T = elastic fundamental period of the structure determined in accordance with IBC 1617.4.2

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)}$$

where S_{DS} = design spectral response acceleration at short period (0.2 sec)

Also, C_s must not be less than that determined from IBC Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E$$

For structures assigned to SDC E or F, and for structures located where $S_1 \geq 0.6g$, C_s shall not taken less than that determined from IBC Eq. 16-38:

$$C_s = \frac{0.5 S_1}{R / I_E}$$

The design spectrum defined by IBC Eqs. 16-34 through 16-38 is depicted in Figure 1-9. Equation 16-35 represents the constant acceleration portion of the spectrum, while Eq. 16-36 represents the constant velocity portion. The design force level defined by Eqs. 16-34 through 16-36 is based on the assumption that a structure will undergo several cycles of inelastic deformation during major earthquake ground motion; therefore, the force level is related to the type of structural system and the structure's estimated ability to sustain these deformations and dissipate energy without collapse. It is important to note that the force level close to that defined by Eqs. 16-34 through 16-36 is also used as the lower bound for the dynamic lateral force procedure of IBC 1618.

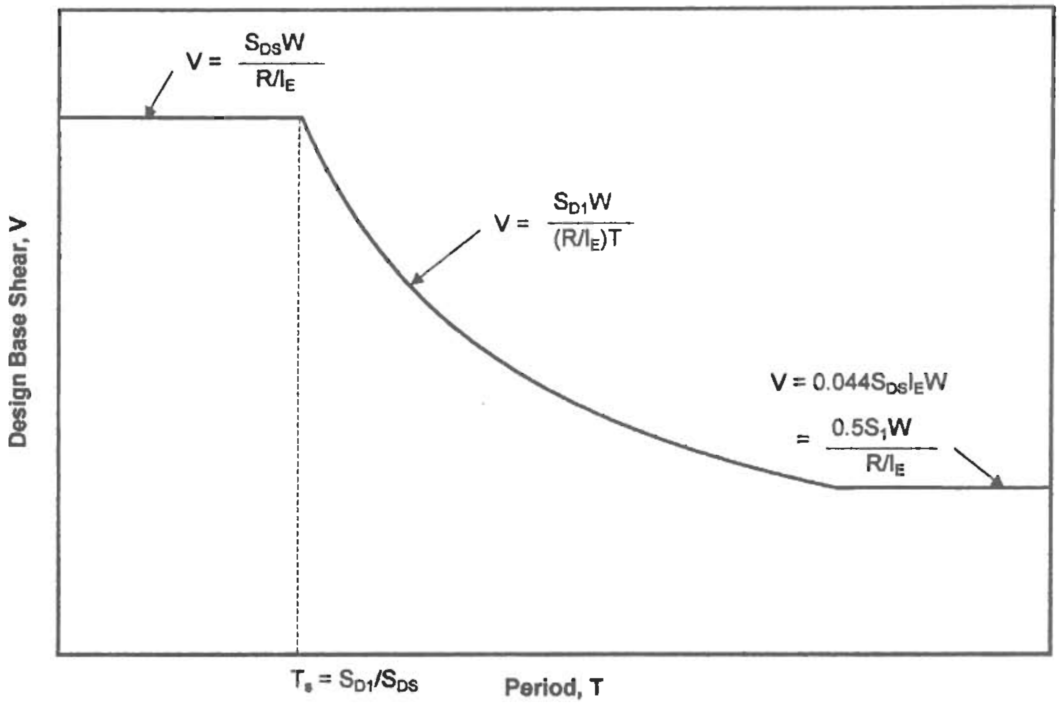


Figure 1-9 Design Response Spectrum According to the Equivalent Lateral Force Procedure in IBC 1617.4

The minimum seismic base shear from Eqs. 16-34 and 16-37 is included in view of the uncertainty and lack of knowledge of actual structural response of long-period buildings subjected to earthquake motions. The lower bound represented by Eq. 16-38 was adopted into the IBC from the 1997 NEHRP Provisions. A form of this lower bound equation originally appeared in the 1997 UBC; it was adopted in response to the 1994 Northridge earthquake. Originally applicable to structures assigned to SDC E and F only, the applicability was later expanded to all structures located where $S_1 \geq 0.6g$.

Elastic Fundamental Period, *T*. The design base shear is dependent on the elastic fundamental period T for buildings in the intermediate height range. However, T, which is a function of the mass and the stiffness of a structure, cannot be determined until a structure has been designed, since until then, the stiffness and the mass cannot be evaluated. Basically, seismic design cannot be started without a period, and a period cannot be determined until seismic design has been done. This being the situation, building codes include approximate period formulas, the purpose of which is to get the design process started. The approximate period formulas deliberately produce estimated periods that are shorter than the "real" periods of actual structures, the idea being that if the initial period estimate is later not refined, the design should still be safe. Since design base shear is inversely proportional to period (Eqs. 16-34 and 16-36), a shorter period means a higher base shear used in design.

IBC 1617.4.2.1 provides Eq. 16-39 to determine an approximate fundamental period T_a:

$$T_a = C_T (h_n)^{3/4}$$

where C_T = building period coefficient
= 0.030 for moment-resisting frames of concrete
= 0.020 for all other concrete structural systems
h_n = height above the base to the highest level of the building

The base is the level at which the horizontal seismic ground motions are considered to be imparted to a structure. Having started design based on the approximate period and having gone part of the way through, it is then possible to refine the initial period estimate if desired. The code permits this to be done.

Period may be estimated by any rational procedure as long as it is in conformance with the principles of mechanics (IBC 1617.4.2). There is, however, scope for potential abuse here. The rationally computed period of a concrete building is very much dependent upon what stiffness assumption is made in the period computation. Gross section stiffness versus cracked section stiffness makes a big difference; how low the cracked section stiffness is taken to be obviously has a major impact. In order to ensure that an unreasonably low design base shear is not taken by calculating an unduly long period based on unrealistic stiffness assumptions, the code imposes a limit on rationally computed period. According to IBC 1617.4.2, the rationally computed T may not be taken any longer than a multiplier C_u, which is obtained from IBC Table 1617.4.2, times the approximate period T_a (this restriction typically does not apply to drift computations). In the 2000 IBC, C_u depends on the long-period design spectral response acceleration, S_{D1}, and varies from 1.2 for $S_{D1} \geq 0.4$ to 1.7 for $S_{D1} \leq 0.1$.

The period $T_s = S_{D1} / S_{DS}$ in Figure 1-9 is the dividing line between "short-period" and "long-period" response. If the period of a structure T is less than or equal to the transition period, T_s, its response is governed by the "flat top" or period-independent part of the spectrum, making it a short-period structure. If, on the other hand, T is larger than or equal to T_S, its response is governed by the period-dependent part of the spectrum, making it a long-period structure.

Equation 16-40 may also be used to determine the approximate fundamental period:

$$T_a = 0.1N$$

where N is the number of stories in a building. This equation is applicable to concrete moment resisting frame buildings not exceeding 12 stories in height and having a minimum story height of 10 ft. This approximate equation has long been in use for low- to moderate-height frames.

Response Modification Factor, *R*. The response modification factor R is intended to account for differences in the inelastic deformability or energy dissipation capacity of various structural systems. It reflects the reduction in structural response caused by damping, overstrength, and inelasticity.

It has been suggested in Section 1.2.1.2 above that an R-value of 2 used in design would result in essentially elastic response of a structure to the design earthquake of the IBC. By contrast, the R-values assigned by the 2000 IBC to structural systems of concrete range from 1-1/2 to 8, as can be seen from IBC Table 1617.6, reproduced here in part as Table 1-6. An R-value of 8 (for special reinforced concrete moment frames or dual systems combining special moment frames with special reinforced concrete shear walls) represents one quarter (2/8) of the strength level that would have been needed for elastic response to the design earthquake of the IBC. An R-value of 1-1/2 (for a bearing wall system consisting of ordinary plain concrete shear walls) represents elastic response to the design earthquake of the IBC, with a margin of safety built in.

The R-values contained in IBC Table 1617.6 are largely based on engineering judgment of the performance of various materials and systems in past earthquakes. For ATC 3-06 [1.11], where R was first introduced, certain agreed-upon reference structures were selected. Two systems having high and low expected levels of performance were chosen to be "a steel ductile frame and a box type masonry or concrete building, respectively." In today's terminology, these would be the special moment frame of steel and a bearing wall system consisting of masonry or concrete shear walls, respectively. The R-values for these two systems were chosen considering the seismic design forces assigned to them by older editions of the UBC. No compelling arguments were offered to change the design basis loads for these systems or to change their interrelationship. The expected performances of other systems were then evaluated relative to these reference systems in order to determine the other R-values. Considerations focused on the following issues:

1. The degree to which the system can be allowed to go beyond the elastic range, its degree of energy dissipation in so doing, and the stability of the vertical load-carrying system during inelastic response due to maximum expected ground motion.

Table 1-6 Seismic-Force-Resisting Systems of Concrete

Basic Seismic-Force-Resisting System	Detailing Reference Section	R	Ω_o	C_d	System Limitations and Building Height Limitations (ft) by SDC*				
					A or B	C	D	E	F
Bearing Wall Systems									
Special reinforced concrete shear walls	1910.2.4	5½	2½	5	NL	NL	160	160	160
Ordinary reinforced concrete shear walls	1910.2.3	4½	2½	4	NL	NL	NP	NP	NP
Detailed plain concrete shear walls	1910.2.2	2½	2½	2	NL	NP	NP	NP	NP
Ordinary plain concrete shear walls	1910.2.1	1½	2½	1½	NL	NP	NP	NP	NP
Building Frame Systems									
Special reinforced concrete shear walls	1910.2.4	6	2½	5	NL	NL	160	160	100
Ordinary reinforced concrete shear walls	1910.2.3	5	2½	4½	NL	NL	NP	NP	NP
Detailed plain concrete shear walls	1910.2.2	3	2½	2½	NL	NP	NP	NP	NP
Ordinary plain concrete shear walls	1910.2.1	2	2½	2	NL	NP	NP	NP	NP
Moment-resisting Frame Systems									
Special reinforced concrete moment frames	ACI 21.1	8	3	5½	NL	NL	NL	NL	NL
Intermediate reinforced concrete moment frames	ACI 21.1	5	3	4½	NL	NL	NP	NP	NP
Ordinary reinforced concrete moment frames	ACI 21.1	3	3	2½	NL	NP	NP	NP	NP
Dual Systems with Special Moment Frames									
Special reinforced concrete shear walls	1910.2.4	8	2½	6½	NL	NL	NL	NL	NL
Ordinary reinforced concrete shear walls	1910.2.3	7	2½	6	NL	NL	NP	NP	NP
Dual Systems with Intermediate Moment Frames									
Special reinforced concrete shear walls	1910.2.4	6	2½	5	NL	NL	160	100	100
Ordinary reinforced concrete shear walls	1910.2.3	5½	2½	4½	NL	NL	NP	NP	NP
Shear wall-frame interactive system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls	ACI 21.1 1910.2.3	5½	2½	5	NL	NP	NP	NP	NP
Inverted Pendulum Systems									
Special reinforced concrete moment frames	ACI 21.1	2½	2	1¼	NL	NL	NL	NL	NL

*NL = not limited, NP = not permitted

2. The consequence of failure or partial failure of vertical elements of the seismic-force-resisting system on the vertical load-carrying capacity and stability of the total building system.

3. The inherent redundancy of the system that would allow some progressive inelastic excursions without overall failure. One localized failure of a part must not lead to failure of the system.

4. Where dual systems are employed, important performance characteristics include the ability of the secondary (back-up) system to maintain vertical support when the primary system suffers significant damage at the maximum deformation response. The back-up system can serve to redistribute lateral loads when the primary system undergoes degradation and should stabilize the building in the event that the primary system is badly damaged.

Vertical Distribution of Seismic Forces. Once the design base shear V has been determined, the lateral force F_x to be applied at level x of the structure is determined from Eqs. 16-41 and 16-42:

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

where C_{vx} = vertical distribution factor

k = distribution exponent related to the building period
= 1 for $T \leq 0.5$ sec
= 2 for $T \geq 2.5$ sec
= 2, or is to be determined by linear interpolation between 1 and 2 for $0.5 \text{ sec} < T < 2.5 \text{ sec}$

h_i, h_x = height from the base to level i and x

w_i, w_x = portion of W located or assigned to level i and x

For structures with $T \leq$ 0.5 sec, V is distributed linearly over the height, varying from zero at the base to a maximum value at the top. For 0.5 sec < T < 2.5 sec, a linear interpolation between a linear and a parabolic distribution is permitted, or a parabolic distribution is also allowed (see Figure 4-10). When $T \geq 2.5$ sec, a parabolic distribution is to be used. The larger the value of k, the higher the proportion of V distributed to the upper portions of a structure. This produces more overturning moment for the same base shear, which is characteristic of flexible building response.

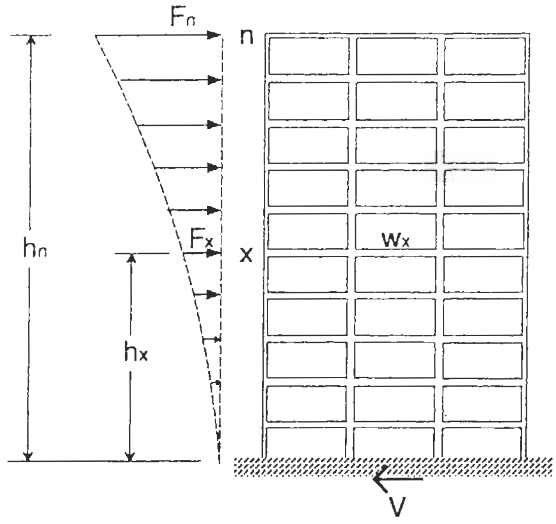


Figure 1-10 Vertical Distribution of Seismic Forces According to IBC 1617.4.3

Horizontal Distribution of Seismic Forces. The seismic design story shear V_x in any story x is the sum of the lateral forces acting at the floor or roof level supported by that story and all the floor levels above, including the roof (IBC Eq. 16-43):

$$V_x = \sum_{i=x}^{n} F_i$$

The distribution of V_x to the vertical elements of the lateral-force-resisting system (shear walls and frames) in story x is determined by the flexibility of the supported diaphragm.

A diaphragm is flexible for the purpose of distribution of the story shear and torsional moment when the lateral deformation of the diaphragm is more than two times the average story drift of the associated story (the story supporting the diaphragm), determined by comparing the computed maximum in-plane deflection of the diaphragm itself under lateral load with the story drift of adjoining vertical resisting elements under equivalent tributary lateral load (IBC 1602.1). A diaphragm that is not flexible by the above definition is rigid for the purposes of the code.

For flexible diaphragms, the seismic design story shear V_x is distributed to various vertical elements based on the area of the diaphragm tributary to each line of resistance. The vertical elements of the seismic-force-resisting system may be considered to be in the same line of resistance if the maximum out-of-plane offset between such elements is less than 5 percent of the building dimension perpendicular to the direction of the lateral force (IBC 1617.4.4.2).

For rigid diaphragms, V_x is distributed to the various vertical elements of the seismic-force-resisting system in the story under consideration based on the relative lateral stiffnesses of the vertical resisting elements and the diaphragm (IBC 1617.4.4.1). Virtually all computer programs utilized for structural analysis assume diaphragms to be rigid (by default), unless otherwise specified.

Torsion, Including Accidental Torsion. Where diaphragms are rigid, provisions must be made for the increased horizontal forces induced on vertical elements of the lateral-force-resisting system resulting from torsion due to eccentricity between the center of application of the lateral forces (center of mass) and the center of rigidity of the seismic-force-resisting system (through which the resultant of the resistances to the lateral forces act). Forces are not to be decreased due to torsional effects.

IBC 1617.4.4.4 clearly states that the torsional design moment at a given story must be the moment resulting from eccentricities between applied design lateral forces at levels above that story and the center of rigidity of the vertical resisting elements in that story (IBC 1617.4.4.3) *plus* an accidental torsion. To compute the accidental torsion, the mass at each level must be assumed to be displaced from the calculated center of mass in each direction a distance equal to 5 percent of the building plan dimension at that level perpendicular to the direction of the force under consideration.

IBC 1617.4.4.5 further requires that where a torsional irregularity or an extreme torsional irregularity exists, as defined in Table 1616.5.1, the effects must be accounted for by increasing the accidental torsion at each level by an amplification factor A_x (≤ 3.0), given by IBC Eq. 16-45. This last requirement applies to buildings in Seismic Design Categories C, D, E, and F only.

Overturning. According to IBC 1617.4.5, structures are to be designed for the effects of overturning caused by the seismic forces determined from IBC 1617.4.3. The overturning moment M_x at level x is determined from Eq. 16-45:

$$M_x = \tau \sum_{i=x}^{n} F_i (h_i - h_x)$$

where F_i = portion of V induced at level i

h_i, h_x = height from the base to level i and x

τ = overturning moment reduction factor
= 1.0 for the top 10 stories
= 0.8 for the 20th story from the top and below
= value between 1.0 and 0.8 determined by a straight-line interpolation for stories between the 20th and 10th stories below the top

Note that τ is permitted to be taken as 1.0 for the full height of the structure.

The reduction, represented by τ, below overturning moments that are statically consistent with the forces F_i, is justified in terms of higher mode response. All the masses at the various floor levels move in the same direction only when response is in the fundamental mode.

Drift Limitation. Drift computation starts with δ_{xe}, the elastically computed lateral deflection at floor level x under code-prescribed seismic forces (the design base shear V, distributed along the height of the structure in the manner prescribed by the code). Recognizing that the deflections δ_{xe} are much lower than the actual lateral deflections the various floor levels would undergo if the structure were to be subjected to the design earthquake of the IBC (two-thirds of the maximum considered earthquake), the deflections δ_{xe} are multiplied by the deflection amplification factor C_d producing estimated design earthquake displacements at the various floor levels. At the same time, the δ_{xe} values are divided by the occupancy importance factor I_E by which the code-prescribed seismic forces (under which the deflections δ_{xe} were calculated) were increased for structures belonging to the higher occupancy categories. This is necessary because the limiting values of interstory drift in the IBC are more stringent for structures in the higher occupancy categories. Without the division by I_E, structures with I_E larger than 1.0 would be doubly penalized. Thus, the deflection δ_x at level x is determined from IBC Eq. 16-46:

$$\delta_x = \frac{C_d \delta_{xe}}{I_E}$$

The design story drift Δ is computed as the difference of the deflections δ_x at the centers of mass of the diaphragms at the top and bottom of the story under consideration (see Figure 1-11). For structures assigned to Seismic Design Category C or higher, with torsional or extreme torsional irregularity (Table 1616.5.1), Δ is computed as the largest difference of the deflections along any of the edges of the diaphragms at the top and bottom of the story under consideration.

Deflections δ_{xe} must be determined based on the elastic properties of all elements of the lateral-force-resisting system, including the spatial distribution of the mass and the stiffness of the structure. For concrete elements, stiffness properties must include the effects of cracked sections.

For the purposes of drift analysis, the upper bound limitation on the computed fundamental period T ($T \le C_u T_a$) does not apply (IBC 1617.4.6.1).

The design story drift Δ must be increased by the incremental factor $1.0/(1-\theta)$ when P-delta effects are determined to be significant (see following section on that topic). When calculating drift, the redundancy coefficient ρ (see later discussion) is to be taken as 1.0.

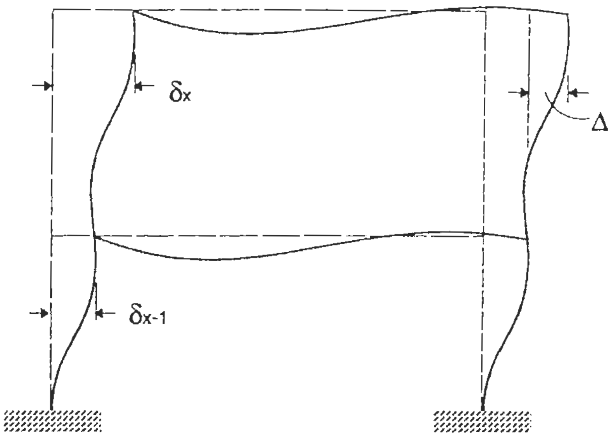


Figure 1-11 Interstory Drift, Δ

Once the design story drifts are computed, they are to be compared to the allowable story drift Δ_a contained in Table 1617.3 (IBC 1617.3). Buildings subjected to earthquakes need drift control to restrict damage to partitions, shaft and stair enclosures, glass and other fragile nonstructural elements, and more important, to minimize differential movement demand on the seismic safety elements. The 2000 IBC limitations on story drift depend on the SUG, and generally become more restrictive for the higher use groups, to provide a higher level of performance. The limits also depend on the type of structure. The design story drifts must not exceed the allowable values.

P-delta Effects. IBC 1617.4.6.2 specifically requires that member forces and story drifts induced by P-delta effects must be considered in the evaluation of overall structural stability. P-delta need not be considered when the ratio of secondary moment to primary moment does not exceed 0.10. The ratio may be evaluated for any story as the product of the dead, floor live, and snow load above the story (P_x), times the unamplified drift (Δ / C_d) in the story, divided by the product of the seismic shear in the story (V_x) and the story height (h_{sx}) (see IBC Eq. 16-47 and Figure 1-12):

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d}$$

If θ is greater than θ_{max}, the structure is potentially unstable and must be redesigned. θ_{max} is given by IBC Eq. 16-48 as follows:

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25$$

where β is the ratio of shear demand to shear capacity for the story between levels x and $x-1$. If β is not calculated, β is to be taken equal to 1.0.

For $0.10 < \theta \leq \theta_{max}$, interstory drift and element forces must be computed including P-delta effects. To obtain the story drift for determining P-delta effects, the design story drift Δ is to be multiplied by $[1.0/(1-\theta)]$ (see preceding section).

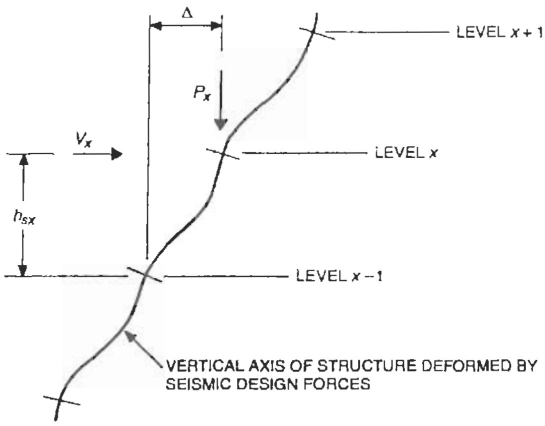


Figure 1-12 P-delta Effect

1.2.3.6 Seismic-Force-Resisting Systems

The basic structural systems that may be used to resist earthquake forces are listed in IBC Table 1617.6. A summary of the seismic-force-resisting systems of reinforced concrete is given in Table 1-6. Included in the table are the response modification factors R to be used in determining the base shear V, the system overstrength factor Ω_o to be used in determining element design forces, and the deflection amplification factor C_d to be used in determining design story drift. A general description of each of the seismic-force-resisting systems is given below.

Moment-Resisting Frame Systems.

Figure 1-13(a) depicts a moment-resisting frame system. This is a structural system with an essentially complete space frame providing support for gravity loads. Lateral forces are resisted primarily by flexural action of the frame members. The entire space frame or selected portions of the space frame may be designated as the seismic-force-resisting system; the members of the seismic-force-resisting system must be designed and detailed accordingly, based on the SDC.

For structures assigned to SDC A or B, an ordinary reinforced concrete moment frame is permitted (IBC 1910.3). No special detailing of the frame members in accordance with

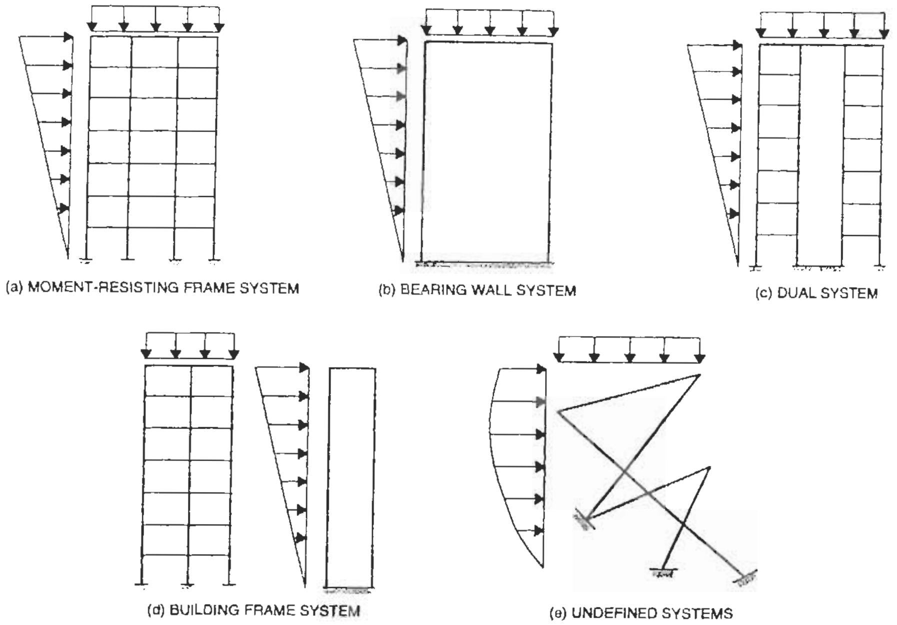


Figure 1-13 Seismic-Force-Resisting Structural Systems

ACI 318 Chapter 21 is required for the effects of seismic forces.

Intermediate moment frames or special moment frames are to be utilized for structures assigned to SDC C (IBC 1910.4). Members of the seismic-force-resisting system of intermediate moment frames are to be detailed in accordance with ACI 21.10.

Special moment frames are required for structures assigned to SDC D, E, or F (IBC 1910.5). The special design and detailing requirements in ACI 21.2 through 21.5 must be satisfied for members of the seismic-force resisting system. For members that are not proportioned to resist forces induced by earthquake motions, the deformation compatibility requirements of ACI 21.9, as modified by IBC 1908.1.11, must be satisfied (IBC 1910.5.2). In short, every structural component not included in the seismic-force-resisting system in the direction under consideration must be designed to be adequate for vertical load-carrying capacity and the induced bending moments and shear forces resulting from the design story drift Δ.

Bearing Wall Systems.

The bearing wall system depicted in Figure 1-13(b) is a structural system without an essentially complete space frame that provides support for the gravity loads. Bearing walls provide support for all or most of the gravity loads. Resistance to lateral forces is provided by the same bearing walls acting as shear walls.

Ordinary reinforced concrete shear walls are permitted to be used in structures assigned to SDC A and B. Ordinary reinforced concrete shear walls are walls conforming to the requirements of ACI 318 Chapter 14 for ordinary reinforced concrete structural walls (IBC 1910.2.3). Note that ordinary plain and detailed plain concrete shear walls may also be used. Ordinary plain concrete shear walls are walls conforming to the requirements of ACI Chapter 22 (IBC 1910.2.1). According to IBC 1910.2.2, detailed plain concrete shear walls are walls conforming to the requirements for ordinary plain concrete shear walls, which contain additional reinforcement per IBC 1910.2.2.

For structures assigned to SDC C, ordinary or special reinforced concrete shear walls are to be utilized. Special reinforced concrete shear walls are walls conforming to the requirements of ACI 21.6 for special reinforced concrete structural walls (IBC 1910.2.4).

Special reinforced concrete shear walls are required for structures with bearing wall systems assigned to SDC D, E, or F (IBC 1910.5.1). In such cases, the height of the building is limited to 160 ft (IBC Table 1617.6).

Dual Systems.

A dual system, which is depicted in Figure 1-13(c), is a structural system with the following essential features:

1. An essentially complete space frame provides support for gravity loads.
2. Resistance to lateral forces is provided by moment-resisting frames capable of resisting at least 25 percent of the design base shear and by shear walls (IBC 1617.6.1).
3. The two subsystems (moment-resisting frames and shear walls) are designed to resist the design base shear in proportion to their relative rigidities (IBC 1617.6.1).

The 2000 IBC separately recognizes dual systems in which the moment-resisting frame consists of special moment frames and dual systems in which the moment-resisting frame consists of intermediate moment frames.

For buildings assigned to SDC D, E, or F, a dual system with special moment frames and special reinforced concrete shear walls can be utilized without any height limitations. A dual system with an intermediate moment frame and special reinforced concrete shear walls may also be used; however, in SDC D, a building is then limited in height to 160 ft and in SDC E or F, the height limit is 100 ft.

For buildings assigned to SDC C, dual systems with special or intermediate moment frames with special or ordinary reinforced concrete shear walls can be used without any limitations.

The concept of the dual system loses its validity in buildings assigned to SDC A and B, since it is questionable whether the moment frames, which are required to have only ordinary detailing, can act as a backup to the ordinary reinforced concrete shear walls (the inelastic deformability of both systems are comparable). In areas of low seismicity, utilizing a shear wall-frame interactive system is more logical. In this system, which is also found in IBC Table 1617.6, the shear walls and frames resist the lateral forces in proportion to their rigidities, considering interaction between the two subsystems at all levels. It is important to note that a shear wall-frame interactive system is only allowed in structures assigned to SDC A or B.

Building Frame Systems.

A building frame system is depicted in Figure 1-13(d). This a structural system with an essentially complete space frame that supports the gravity loads. Resistance to lateral forces is provided by shear walls. No interaction between the shear walls and frames is considered in the lateral load analysis; all of the lateral forces are allocated to the walls.

For structures assigned to SDC A or B, the following types of concrete shear walls may be utilized without any limitations: ordinary plain, detailed plain, and ordinary reinforced. Ordinary reinforced shear walls are allowed to be used in SDC C.

Special reinforced concrete shear walls must be used in SDC D, E, or F. In such cases, the building is limited in height to 160 ft, and the deformational compatibility requirements in IBC 1617.6.4.3 must be satisfied. Although the walls are designed to carry all of the seismic forces, the beam-column frames must be designed to resist the effects caused by the lateral deflections, since they are connected to the walls at every level. Members not designated to be part of the seismic-force-resisting system must be capable of maintaining support of the gravity loads when subjected to the expected deformations caused by the seismic forces.

Similar to dual systems, the concept of the building frame system loses its appeal for structures assigned to SDC A or B, since there is little to be gained from assigning the entire lateral resistance to the shear walls in the absence of any special detailing requirements for the frames. As noted above, a shear wall-frame interactive system is more practical and economical in such cases.

Inverted Pendulum Systems.

Inverted pendulum type structures are defined in IBC 1613 as structures that have a large portion of their mass concentrated near the top; thus, they have essentially one degree of freedom in horizontal translation. These structures have little redundancy and overstrength, and inelastic behavior is concentrated at their bases. As a result, they have substantially less energy dissipation capacity than other systems.

Undefined Structural Systems.

Undefined structural systems are any systems not listed in IBC Table 1617.6. The coefficients R, Ω_o, and C_d are to be substantiated based on approved cyclic test data and analysis (IBC 1617.6).

1.2.3.7 Seismic Force Effects

Once the seismic-force-resisting system has been chosen and the design base shear V has been determined and distributed over the height of the building, an analysis of the building is performed, and the structural members are designed for the combined effects of gravity and seismic forces. The effects of wind, which are covered in the next section, are to be considered as well.

Load Combinations.

Basic load combinations for strength design are given in IBC 1605.2.1. The first exception in this section requires that the non-seismic load combinations of ACI 9.2 be used for concrete structures. Thus, the following load combinations are utilized in the design of the structural members:

1. $1.4D + 1.7L$ (ACI Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (ACI Eq. 9-2)
3. $0.9D + 1.3W$ (ACI Eq. 9-3)
4. $1.2D + f_1 L + 1.0E$ (IBC Formula 16-5)
5. $0.9D + 1.0E$ (IBC Formula 16-6)

where D, L, W, and E are the effects due to dead loads, live loads, wind, and seismic forces, respectively, and f_1 is equal to either 1.0 for places of public assembly, for live loads in excess of 100 psf, and for parking garage live load, or is equal to 0.5 for other live loads.

Seismic Force Effect, *E*.

According to IBC 1617.1.1, the seismic force effect E for use in load combination Formula 16-5, which is the combined effect of horizontal and vertical earthquake-induced forces, is computed from Eq. 16-28 when the effects of gravity and seismic ground motion are additive:

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of horizontal seismic forces

ρ = redundancy coefficient determined in accordance with IBC 1617.2.2 for structures assigned to SDC D, E, or F

= 1.0 for structures assigned to SDC A, B, or C

Similarly, E for use in Formula 16-6 is computed from Eq. 16-29 where the effects of gravity and seismic ground motion are counteractive:

$$E = \rho Q_E - 0.2 S_{DS} D$$

Redundancy.

The redundancy coefficient ρ is a measure of the redundancy inherent in the lateral-force-resisting system and depends on three items:

1. The number of vertical lateral-force resisting elements.
2. Floor areas of the building at different story levels.
3. Distribution of lateral forces to the lateral-force-resisting elements.

According to IBC 1617.2.2, ρ is the largest of the values of ρ_i at each story *i* computed in accordance with Eq. 16-32:

$$\rho_i = 2 - \frac{20}{r_{max_i}\sqrt{A_i}}$$

where, in general, for a given direction of loading, r_{max_i} is the ratio of the design story shear resisted by the most heavily loaded single element in the story to the total story shear, and A_i is the floor area in square feet of the diaphragm level immediately above story *i*.

Determination of r_{max_i} depends on the type of lateral-force-resisting system:

- For moment frames:

  r_{max_i} = maximum sum of shears in any 2 adjacent columns/total story shear

  In the column shear summation, use 70% of shear in columns common to 2 adjacent bays, with moment-resisting connections on opposite sides.

- For shear walls:

  r_{max_i} = (maximum wall shear × $10/\ell_w$)/total story shear

  ℓ_w = length of the wall in feet

- For dual systems:

$$r_{max_i} = \text{max. of} \begin{cases} r_{max_i} \text{ calculated for portion of story shear carried by columns} \\ r_{max_i} \text{ calculated for portion of story shear carried by walls} \end{cases}$$

ρ based on the above may be reduced by 20 percent in recognition of frames backing up shear walls.

The value of ρ must not be less than 1.0 and need not be greater than 1.5 (i.e., the upper limit on the penalty for nonredundancy is 1.5). For structures with seismic-force-resisting systems comprised solely of special moment frames, the upper limit on ρ is equal to 1.25 for structures assigned to SDC D, and is 1.1 for structures assigned to SDC E or F. In other words, if ρ is determined to be greater than the appropriate upper limit, the physical configuration of the system must be changed until ρ comes within the target range.

Seismic Force Effect, E_m.

IBC 1620.1.9 requires that elements supporting discontinuous walls or frames of structures having certain plan or vertical irregularities in SDC B or higher must have the design strength to resist the special seismic load combinations in IBC 1605.4:

1. $1.2D + f_1 L + E_m$
2. $0.9D + E_m$

where E_m is the maximum effect of horizontal and vertical forces determined from Eqs. 16-30 and 16-32 in IBC 1617.1.2:

$$E_m = \Omega_o Q_E + 0.2 S_{DS} D$$ (when effects of gravity and seismic forces are additive)

$$E_m = \Omega_o Q_E - 0.2 S_{DS} D$$ (when effects of gravity and seismic forces counteract)

The overstrength factor Ω_o, which is given in Table 1617.6 for the various seismic-force-resisting systems, increases the design level effects to represent the actual forces that may be experienced in a structural member as a result of the design ground motion (see Figure 1-3, which also shows the corresponding displacement that is expected when the effects of the horizontal ground motion Q_E are amplified by Ω_o).

The term $\Omega_o Q_E$ need not exceed the maximum force that can be transferred to the element by the other elements of the seismic-force-resisting system.

1.2.3.8 Structural Component Load Effects

Except for structures assigned to SDC A, the design and detailing of the components of the seismic-force-resisting system must comply with the requirements of IBC 1620.1 in addition to the nonseismic requirements of the IBC. The discussion that follows focuses on those provisions in IBC 1620 that are required for the types of structures addressed in this publication.

Diaphragms.

For structure assigned to SDC B and C, floor and roof diaphragms are to be designed to resist the seismic force F_p determined from IBC Eq. 16-62:

$$F_p = 0.2 I_E S_{DS} w_p + V_{px}$$

where w_p is the weight of the diaphragm and other elements of the structure attached to it and V_{px} is the portion of the seismic shear force at the level of the diaphragm that is required to be transferred to the components of the vertical seismic-force-resisting system due to offsets or changes in the stiffness of the vertical components above or below the diaphragm. An example of an offset in the vertical components of the seismic-force-resisting system is illustrated in Figure 1-14. The diaphragm must be designed for the additional force due to this offset.

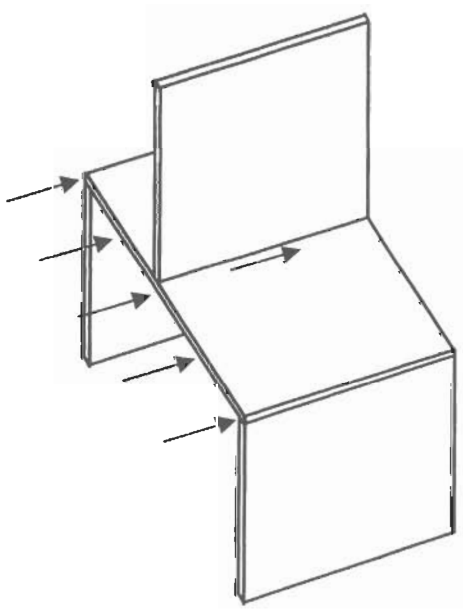


Figure 1-14 Example of Offset of Vertical Components of Seismic-force-resisting System

For structures assigned to SDC D or above, floor and roof diaphragms must be designed to resist the design seismic force F_{px} given in IBC Eq. 16-65:

$$0.3S_{DS}I_E w_{px} \geq F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \geq 0.15S_{DS}I_E w_{px}$$

where F_i is the design level force applied to level i, w_i is the weight tributary to level i, and w_{px} the weight tributary to the diaphragm at level x. Similar to the case described above for SDC B and C, any forces due to offsets in the vertical seismic-force-resisting system or changes in lateral stiffness of the vertical elements must be added to the force determined from Eq. 16-65.

Collector Elements.

In general, collector elements transfer seismic forces from the diaphragm to the vertical elements of the seismic-force-resisting system. Collectors (or drag struts), are required, for example, when shear walls do not extend the full length of the diaphragm in the direction of loading. It is essential that the seismic forces are transferred to the shear walls in order to guarantee a continuous load path. Therefore, IBC 1620.1.6 requires that for structures assigned to SDC B or C, collector elements, splices, and their connections be designed to resist the special load combinations in IBC 1605.4.

In addition to the requirements for SDC B or C, collector elements, splices, and their connections must be designed to resist the forces determined in accordance with IBC Eq. 16-65 for structures assigned to SDC D and higher (IBC 1620.3.4).

Bearing Walls and Shear Walls.

In addition to seismic forces in the plane of the wall, bearing walls, shear walls, and their anchorages in structures assigned to SDC B and higher are to be designed to resist an out-of-plane force F_p given by IBC Eq. 16-63:

$$F_p = 0.40 I_E S_{DS} w_w \geq 0.10 w_w$$

where w_w is the weight of the wall.

Additionally, concrete walls are to be anchored to the roof and floor members that provide lateral support for the wall. The anchorage shall be capable of resisting the greater of F_p or $400 S_{DS} I_E$ pounds per linear foot of wall.

For structures assigned to SDC C and higher with diaphragms that are not flexible, which is typical for reinforced concrete systems, anchorage of concrete walls shall be designed for the forces specified in IBC 1621.1.4 (IBC 1620.2.1).

Direction of Seismic Load.

For structures assigned to SDC B or C, IBC 1620.1.10 requires that the seismic forces be applied in a direction that will produce the most critical load effects in the members. This requirement is satisfied if the design seismic forces are applied separately and independently in each of the two orthogonal directions.

For SDC C, structures that have plan irregularity Type 5 in Table 1616.5.1 are required to satisfy the requirements in IBC 1620.2.2. The critical direction requirements of IBC 1620.1.10 are deemed to be satisfied if the members are designed for either of the following orthogonal combinations of the applied loads: (1) one hundred percent of the forces for one direction plus 30 percent of the forces for the perpendicular direction (the member is designed for the combination producing the critical load effects) (2) the effects of the two orthogonal directions are permitted to be combined on a square root of the sum of squares basis (SRSS). When the SRSS method is utilized, each sum must be assigned the sign that will produce the most conservative result.

The independent orthogonal procedure is not satisfactory for members that are part of a structure assigned to SDC D or higher. In such cases, members are to be designed using the orthogonal combination procedure in IBC 16202.2 that is described above (IBC 1620.3.5). In the 2002 supplement to the 2000 IBC [1.17], the orthogonal combination procedure is required only for columns or walls that form part of two or more intersecting seismic-force-resisting systems and are subjected to axial load due to seismic forces greater than or equal to 20% of the axial load design strength.

1.2.3.9 Detailing Requirements

As was discussed previously, the design seismic forces determined by the analysis procedures described in Section 1.2.3.5 of this publication are only a fraction of the actual forces that an elastic structure may experience during an earthquake. For design purposes, the seismic forces are reduced, since it is impractical as well as uneconomical to design a structure to remain elastic in such situations. In order to prevent collapse, structures must be properly detailed so that they are able to dissipate the earthquake energy through inelastic deformations.

The 2000 IBC references the requirements of ACI 318-99 for the design of reinforced concrete structures, with a few modifications. Chapter 21 of ACI 318-99 contains the design and detailing requirements for structures assigned to regions of low, moderate, and high seismic risk. Table 1-7 contains a summary of the sections of Chapter 21 to be satisfied as a function of the SDC for various types of components resisting earthquake effects.

Reference 1.18 contains a comprehensive summary of the design and detailing requirements in Chapter 21 of ACI 318-99. Included are tables and figures that summarize the provisions for all of the components listed in Table 1-7.

Table 1-7 Sections of ACI 318 Chapter 21 to be Satisfied†

Component Resisting Earthquake Effect	Seismic Design Category (SDC)	
	SDC C (21.2.1.3)	**SDC D, E, or F (21.2.1.4)**
Frame members	21.10	21.2–21.5
Structural walls and coupling beams	None	21.2, 21.6
Structural diaphragms and trusses	None	21.2, 21.7
Foundations	None	21.2, 21.8
Frame members not proportioned to resist forces induced by earthquake motions	None	21.2, 21.9

†In addition to requirements of Chapters 1–18 for structures assigned to SDC C and Chapters 1–17 for structures assigned to SDC D and higher.

1.2.4 Impact of the 2000 IBC Seismic Design Provisions

As is evident from the discussion above, the seismic design provisions of the 2000 IBC represent revolutionary changes from those of the three model building codes.

The procedure for establishing the trigger that determines detailing requirements and other applicable restrictions has become more complex in the 2000 IBC. In earlier model codes, determining the seismic zone of a structure only required establishing the location of the structure on the seismic zone maps contained in the model codes.

In the more recent editions of the BOCA/NBC and SBC, determining the Seismic Performance Category (SPC) of a structure requires interpolating the ground motion parameter A_v on a contour map, based on the location of the structure, and determining the use classification of the structure. The SPC is subsequently determined by using a table.

When determining a structure's Seismic Design Category (SDC), site-specific soil data must be gathered to establish the site class; otherwise, the default Site Class D must be used. The IBC procedure requires evaluation of SDC for short-period and long-period ground motion parameters. After working through the requirements, the more severe SDC is selected on the basis of these parameters.

Tables 1-8, 1-9, and 1-10 below provide snapshots of the potential impact of the difference in the triggering mechanism for invoking higher levels of detailing and other restrictions under the 2000 IBC criteria as compared to the latest editions of the BOCA/NBC, the SBC, and the UBC. These tables assume a Seismic Use Group I (standard occupancy) structure.

The default Site Class D is of particular interest. If the average shear wave velocity, standard penetration resistance, or undrained shear strength has not been determined for the top 100 ft (30 m) of soil at a site (as indicated in Table 1615.1 of the 2000 IBC), the engineer must design for an assumed Site Class D.

A comprehensive comparison of the seismic design provisions in the 2000 IBC and the three model codes is given in Reference 1.19. Based on the information in that reference, the potential impact of the 2000 IBC on seismic design and construction across the U.S. appears to be quite substantive. The following section summarize some of the findings from that report.

1.2.4.1 Comparison: 2000 IBC and 1999 BOCA/NBC

It is very instructive to compare the soil-independent Seismic Performance Categories of the 1999 BOCA/NBC and the soil-dependent seismic design categories of the 2000 IBC for the various locations listed in Table 1-8. For instance, structures in Cincinnati, Ohio, that are assigned to SPC B under the BOCA/NBC may be assigned to SDC A, B, C, or D under the IBC, depending upon soil conditions at the building site.

Table 1-8 Seismic Design Categories of 2000 IBC vs. Seismic Performance Categories of 1999 BOCA/NBC

Location	BOCA/NBC	2000 IBC				
		Site Class				
		A	B	C	D	E
	SPC	SDC				
Washington, D.C.	A	A	A	B	B	C
Chicago, IL	A	A	A	B	B	C
Baltimore, MD	A	A	A	B	B	C
Boston, MA	C	B	B	B	C	D
New York, NY	C	B	B	C	C	D
Cincinnati, OH	B	A	A	B	C	D
Philadelphia, PA	B	B	B	B	C	C
Richmond, VA	B	A	B	B	B	C

Table 1-8 shows that a standard-occupancy structure in Cincinnati on Site Class D would be assigned to IBC Seismic Design Category C, thus triggering the equivalent of UBC Seismic Zone 2 detailing and other restrictions. A standard-occupancy structure at the same location on Site Class E would be assigned to IBC SDC D, thus triggering the equivalent of UBC Seismic Zone 3 and 4 detailing and other restrictions. The comments here, however, are made without specific knowledge of existing soil types at the various locations. For instance, Site Class E may or may not exist in and around Cincinnati.

1.2.4.2 Comparison: 2000 IBC and 1997 UBC

As shown in Table 1-9, many of the Zone 3 or 4 locations on Site Class E require a site-specific geotechnical investigation. This is currently not required by the 1997 UBC. On firmer soils in Sacramento, CA, only the equivalent of Zone 2 detailing will be required under the 2000 IBC. This is because the seismicity at the location has been judged to be lower by the IBC. In many of the low-seismicity locations examined, such as Denver (Seismic Zone 1), the equivalent of Zone 2 detailing will be required for Site Class E. These comments are once again made without specific knowledge of existing soil types at the various locations. For instance, while it is known that Site Class A does not exist in California, Site Class B may or may not exist in and around Sacramento.

Table 1-9 Seismic Design Categories of 2000 IBC vs. Seismic Zone of 1997 UBC

Location	UBC	2000 IBC				
		Site Class				
		A	B	C	D	E
	Zone	SDC				
West L.A. ($N_a = 1.3, N_v = 1.6$)	4	E	E	E	E	*
San Francisco, CA	4	D	D	D	D	*
Berkeley, CA ($N_a = 1.5, N_v = 2.0$)	4	E	E	E	E	*
Denver, CO	1	A	A	A	B	C
Sacramento, CA	3	B	C	D	D	D
St. Paul, MN	0	A	A	A	A	B
Seattle, WA	3	D	D	D	D	*
Portland, OR	3	D	D	D	D	D
Houston, TX	0	A	A	A	B	B

* Site specific geotechnical investigation and dynamic site response analysis must be performed

1.2.4.3 Comparison: 2000 IBC and 1999 SBC

Comparison of soil-independent Seismic Performance Categories of the 1999 SBC and soil-dependent Seismic Design Categories of the 2000 IBC for the locations listed in Table 1-10 appears to indicate that the IBC may have its greatest impact on seismic design and construction in cities in the SBC territory. Standard-occupancy structures in Atlanta, GA, currently are in SPC B, which requires no special seismic detailing. Such structures under the IBC will be assigned to SDC C, if founded on the default Site Class D. This would require these structures to conform to the equivalent of UBC Seismic Zone 2 requirements. If founded on Site Class E, the same structures will be assigned to SDC D, triggering the equivalent of UBC Seismic Zone 3 and 4 detailing and other restrictions.

In Charlotte, NC, standard-occupancy structures are now in SPC C, requiring the equivalent of Seismic Zone 2 detailing. The same structures under the IBC would be assigned to SDC D, if founded on the default Site Class D, thus triggering the equivalent of UBC Seismic Zone 3 and 4 detailing and other restrictions.

In Charleston, SC, as in Charlotte, standard-occupancy structures are now assigned to SPC C, requiring the equivalent of UBC Seismic Zone 2 detailing. The same structures under the IBC would be assigned to SDC D, irrespective of the site class they are founded on, thus triggering the equivalent of UBC Seismic Zone 3 and 4 detailing and other restrictions. This is because the IBC judges the seismicity at the location to be higher.

Table 1-10 Seismic Design Categories of 2000 IBC vs. Seismic Performance Categories of 1999 SBC

Location	SBC	2000 IBC				
		Site Class				
		A	B	C	D	E
	SPC	SDC				
Birmingham, AL	B	B	B	B	C	D
Atlanta, GA	B	A	B	B	C	D
Orlando, FL	A	A	A	A	B	B
Little Rock, AK	B	B	B	C	D	D
New Orleans, LA	A	A	A	A	B	B
Nashville, TN	B	B	B	C	D	D
Charlotte, NC	C	B	B	C	D	D
Charleston, SC	C	D	D	D	D	D

Once again, the above comments are made without specific knowledge of existing soil types at the various locations. For instance, Site Class E may or may not exist in and around Atlanta.

1.2.4.4 Overall Observations

In addition to Reference 1.19, the various aspects of impact of the more recent seismic regulations, as embodied in the 1997 NEHRP Provisions, the 2000 IBC, and ASCE 7-98, have been examined in detail elsewhere [1.20]. S.K. Ghosh Associates Inc. carried out a limited comparative design study using the 2000 IBC, the 1997 UBC, and the 1991 NEHRP Provisions (as contained in the last three editions of the BOCA/NBC and the SBC) for reinforced concrete structures. J.R. Harris & Co. conducted a similar study for steel, masonry, and wood structures. Both efforts were funded by FEMA under a contract administered by the BSSC, and limited results of this study have been published [1.21]. Reference 1.21 makes the following significant points:

1. Required level of seismic detailing under 1997 NEHRP/2000 IBC/ASCE 7-98 is a function of the soil characteristics at the site of a structure. The level additionally depends on the occupancy or use of the structure and on seismic risk at the site of the structure.
2. The design ground motion parameters in the 1997 and 2000 NEHRP Provisions, the 2000 IBC, and ASCE 7-98 differ significantly from those in any previous code or code-like document. Seismicity assessment at many locations across the U.S. has also changed rather significantly.

3. In the 2000 IBC, response modification factors R for several concrete structural systems (the higher the R, the lower the seismic design force) differ significantly from those in any previous code.

4. The occupancy importance factor I of the UBC and other older seismic codes (the higher the I, the higher the seismic design force) has been introduced into the 1997 and 2000 NEHRP Provisions, the 2000 IBC, and ASCE 7-98.

5. Differences in soil-modified design ground motion parameters, R-values and I-values between existing model codes and the 1997 and 2000 NEHRP Provisions, the 2000 IBC, or ASCE 7-98 often add up to differences in seismic design forces for the same structural systems. These are significant in many locations.

6. The treatment of soils at the site in the seismic design of a structure is profoundly different in the 1997 and 2000 NEHRP Provisions, the 2000 IBC, and ASCE 7-98 than in any previous code or code-like document, with the sole exception of the 1997 UBC.

7. Seismic design to withstand moderate to severe ground motion is no longer only a regional concern. The equivalent of moderate or even high seismic zone design may be required in unlikely places, particularly on softer soils.

1.3 WIND-RESISTANT DESIGN

1.3.1 Wind Forces

In general, the application of wind forces to a building is in the form of pressures that act normal to the surface of the building (see Figure 1-15). Positive (or above ambient) wind pressure (commonly referred to as just pressure) acts towards the surface of the building, while negative (or below ambient) pressure (or, suction) acts away from the surface. For the simple structure shown in Figure 1-15, it can be seen that positive pressure acts on the windward wall and negative pressure acts on the leeward wall, the walls parallel to the direction of wind, and the leeward portion of the roof. Either positive or negative pressure acts on the windward portion of the roof, depending on its slope.

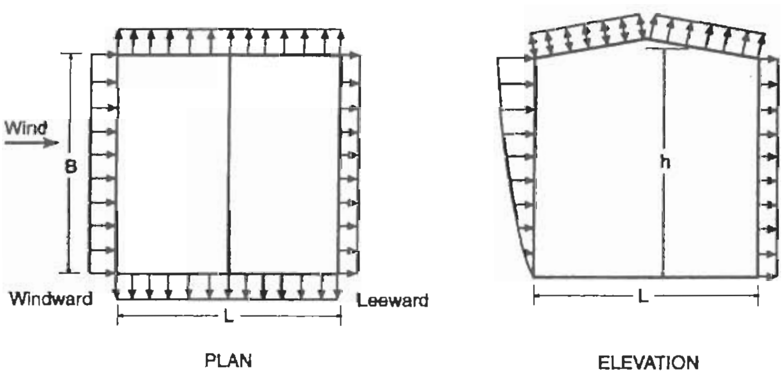


Figure 1-15 Wind Pressures on a Building

Building codes typically contain static methods for estimating the magnitude of wind pressure, which, in general, depends on the importance, size, geometry, openness, and location (exposure) of the structure, and on the height above the ground level (as illustrated on the elevation of the building in Figure 1-15). These methods also account for gusting and local extreme pressures at various locations on the building.

Static methods cannot be used for most tall structures that respond dynamically to wind forces; in such cases, inertial forces are generated. Dynamic analysis or a wind tunnel test to determine the design pressures is warranted in these cases.

1.3.2 Response of Concrete Buildings to Wind Forces

Figure 1-16 shows the path of wind forces as they propagate through a simple, idealized concrete building. The windward wall receives the wind pressures and transfers the resulting forces to the roof and floor diaphragms. The diaphragms, in turn, transfer these loads to the elements of the lateral-force-resisting system, which in this case, are the shear walls parallel to the wind forces. The shear walls then transfer these loads to the building foundation. A similar scenario would occur for frame buildings. In any case, the design of structural members for wind forces is based on linear behavior; the structure is assumed to remain elastic under the design wind forces.

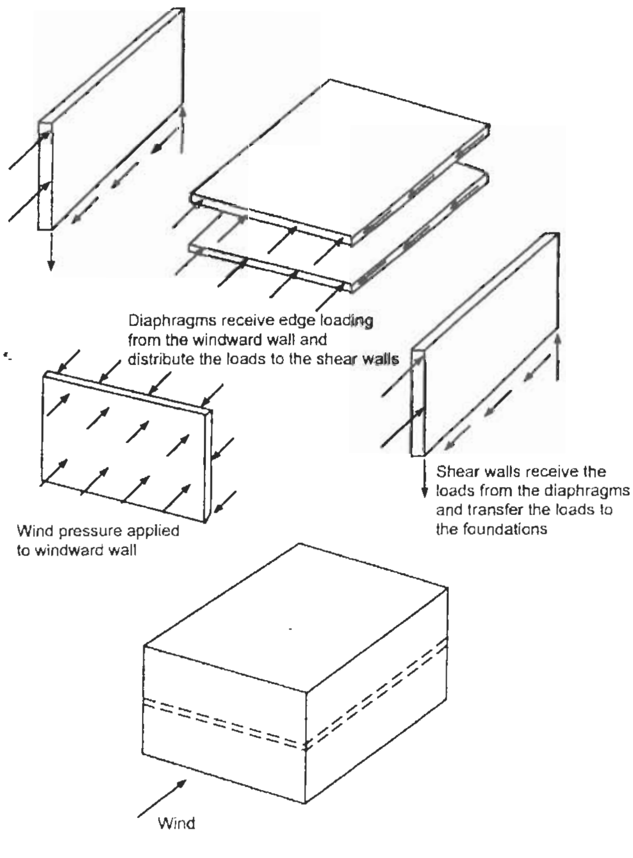


Figure 1-16 Propagation of Wind Forces in a Concrete Building

1.3.3 Wind Design Requirements of the 2000 IBC

The wind load provisions of the 2000 IBC are contained in IBC 1609. According to IBC 1609.1.1, wind loads on buildings and structures are to be determined in accordance with Section 6 of ASCE 7-98 [1.3]. Exceptions to this provision are also contained in that section, and are applicable mainly to residential buildings. One exception allows the use of the simplified provisions for low-rise buildings that are contained in IBC 1609.6, which is covered in the next section.

IBC 1609.1.2 specifies a minimum wind pressure of 10 psf for the design of the main wind-force-resisting system (MWFRS) acting on the area of an enclosed or partially enclosed structure projected on a vertical plane normal to the direction of the wind. A minimum wind pressure of 10 psf is also required for components and cladding acting in either direction normal to the surface.

In wind-borne debris regions, as defined in IBC 1609.2, glazing in the lower 60 ft is assumed to be openings unless impact-resistant glazing is used or the glazing is protected with an impact-resistant covering meeting the requirements given in IBC 1609.1.4. Whether this type of protection is provided or not has an important effect on the design wind pressures.

It is important to note that appropriate levels of seismic detailing must be provided in any structure, even when the effects from wind forces govern the design (IBC 1609.1.5).

Reference 1.22 contains a lucid summary of the wind load provisions contained in the 2000 IBC.

1.3.3.1 Simplified Provisions for Low-rise Buildings

The simplified provisions of IBC 1609.6 are allowed to be used for determining wind forces on simple diaphragm buildings with flat, gabled, or hipped roofs that have a mean roof height less than or equal to the least horizontal dimension of the building or 60 ft, whichever is less. Additionally, such buildings must not be located on the upper half of an isolated hill or escarpment that meets the conditions in IBC 1609.6.1. These provisions are based on Method 1, the simplified procedure in ASCE 6.4, and are briefly described below.

The seven characteristics that define a simple diaphragm building are contained in IBC 1609.2. Based on this definition, a concrete-framed building can be classified as a simple diaphragm building.

As its name implies, determining wind forces by the simplified procedure is simple. The basic wind speed and wind importance factor are obtained from Figure 1609 and Table 1604.5, respectively. Pressures for various wind speeds are tabulated for Exposure B buildings up to 30 ft in (mean roof) height:

- Table 1609.6.2.1(1): main wind-force-resisting systems

- Table 1609.6.2.1(2): components and cladding
- Table 1609.6.2.1(3): roof overhang component and cladding

Table 1609.6.2.1(4) contains the adjustment coefficients that are to be used to modify the pressures from the above tables for different exposures and mean roof heights.

Figures 1609.6(1), 1609.6(2), and 1609.6(3) contain the loading diagrams that are to be used for the MWFRS and components and cladding.

Since the wind forces are delivered to the MWFRS through the roof and floor slabs and the building, by definition of a simple diaphragm building, is enclosed, internal pressures are not required to be computed for the walls, since they cancel out. Internal pressures must be considered for the roof.

1.3.3.2 Wind Loads According to ASCE 7-98

As noted above, wind loads are to be determined in accordance with Section 6 of ASCE 7-98 when the simplified method in IBC 1609.6 cannot be used. What follows is a summary of the provisions contained in that section.

Method 1 – Simplified Procedure

ASCE 6.4 contains a simplified method that can be used to determine design wind loads for enclosed or partially enclosed structures (note: definitions for these types of structures are given in ASCE 6.2). Although the provisions contained in this section are very similar to those in IBC 1609.6, their applicability is not as broad: Method 1 applies to essentially flat roof buildings (roof slope less than 10 degrees) and only buildings that are less than or equal to 30 ft in height. Note that ASCE 6.4 has not been adopted by the 2000 IBC; IBC 1609.6 has taken its place instead.

Conditions when method 1 can be applied are contained in ASCE 6.4.1, and the design procedure is given in ASCE 6.4.2, which closely follows the procedure given in IBC 1609.6.

Method 2 – Analytical Procedure

The analytical procedure in ASCE 6.5 can be used to determine design wind forces for any building or structure that meets the following conditions:

- The building or structure is regular-shaped (has no unusual geometrical irregularity in spatial form), and
- The building or structure does not have response characteristics that make it subject to across wind loading, vortex shedding, instability due to gallop or flutter; or is not located on a site where channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.

Commentary Section C6.5 provides examples of structure types and site locations that do not satisfy the conditions listed above. In such cases, the wind tunnel procedure in ASCE 6.6 or methods in recognized literature are to be used to determine the wind effects on the structure.

The 10-step design procedure of the analytical method is contained in ASCE 6.5.3. Each of these steps is examined below.

1. Basic wind speed, *V*, and wind directionality factor, K_d. The basic wind speed V is determined in accordance with ASCE 6.5.4. ASCE Figure 6-1 contains the wind speed map for the contiguous United States and Alaska. Wind speeds correspond to 3-sec gust speeds at 33 ft above the ground for Exposure C (exposure categories are described below). The mean recurrence interval for these wind speeds is 50 years (annual probability of occurrence equal to 0.02).

The wind speed map in ASCE 7-98 has been updated to include hurricane wind speeds based on more complete data analyses. Also, the hurricane importance factor that was used to modify wind speeds obtained from maps in earlier editions of the standard is now included in the map contours in ASCE 7-98. In the case of hurricane winds, ultimate loads are obtained after the loads calculated from the standard are multiplied by the appropriate load factors; these ultimate loads have approximately the same return period as loads that are determined from the standard for non-hurricane winds.

The wind directionality factor, which has been essentially hidden in the wind load factor in previous editions of the standard, has been separated out in ASCE 7-98. This factor accounts for the reduced probability of maximum winds coming from any given direction and the reduced probability of the maximum pressure coefficient occurring for any given wind direction.

Values of the wind directionality factor K_d are contained in ASCE Table 6-6 for various types of structures. The footnote to this table is important: K_d has been calibrated with the load combinations specified in Sections 2.3 and 2.4. Note that these load combinations are essentially the same as those in Sections 1605.2 and 1605.3 of the 2000 IBC. Exception 1 in IBC 1605.2.1 is important for concrete structures: load combinations of ACI 9.2 are to be used for concrete structures where combinations do not include seismic forces. The load factors in the ACI 318 combinations are different than those in ASCE 7 and the IBC. The exception goes on to state that for concrete structures designed for wind in accordance with ASCE 7-98 using the ACI load combinations, wind forces are to be divided by K_d.

2. Importance factor, I_W. Similar to the seismic importance factor I_E, the wind importance factor I_W is used to adjust the level of structural reliability of a building or structure. This is accomplished by adjusting the velocity pressure to different return periods. For non-hurricane winds, importance factors equal to 0.87 and 1.15 correspond to mean recurrence intervals of 25 and 100 years, respectively. For hurricane winds, mean recurrence levels vary along the coast; risk levels, though, are approximately consistent with those applied to non-hurricane winds.

ASCE Table 6-1 contains I_W as a function of the occupancy classification and location of the building (non-hurricane or hurricane prone region). In ASCE 7, occupancy categories are contained in Table 1-1. The wind load importance factors are the same in ASCE 7 Table 6-1 and IBC Table 1604.5. However, what is important to note is that the numbers corresponding to the occupancy category are different in the two tables for a given occupancy: Occupancy Categories I, II, III, and IV in the IBC are equivalent to Occupancy Categories II, III, IV, and I, respectively, in ASCE 7.

3. Exposure categories and Velocity pressure exposure coefficient, K_z. An exposure category that adequately reflects the characteristics of ground surface irregularities must be determined for each wind direction that is considered (IBC 1609.4 and ASCE 6.5.6). The exposure categories in both codes are essentially the same, except the IBC has made modifications to the definitions of Exposure B and Exposure C. In view of new research, revisions were made to the definitions for Exposure C and Exposure D in ASCE 7-98. An improvement introduced in ASCE 7-98 is that wind forces for both the MWFRS and components and cladding are based on the actual exposure. Previously, using only Exposure C or using a modofied exposure factor for Exposure B in such cases was required.

According to ASCE 6.5.6.4, values of the velocity pressure exposure coefficient K_z are to be determined from ASCE Table 6-5. To account for increased wind loading caused by local turbulence and from increased wind speed near the surface associated with openings in surface roughness, the wind profiles have been truncated in the bottom 100 ft and 30 ft for Exposures A and B, respectively, in the case of low-rise buildings and components and cladding.

In lieu of linear interpolation, K_z may be calculated at any height z above ground level from the equations given at the bottom of Table 6-5:

$$K_z = \begin{cases} 2.01\left(\dfrac{15}{z_g}\right)^{2/\alpha} & \text{for } z < 15\text{ ft} \\ 2.01\left(\dfrac{z}{z_g}\right)^{2/\alpha} & \text{for } 15\text{ ft} \le z \le z_g \end{cases}$$

where α = 3-second gust speed power law exponent from ASCE Table 6-4
z_g = nominal height of the atmospheric boundary layer from ASCE Table 6-4

4. Topographic factor, K_{zt}. Buildings or structures located on the upper half of an isolated hill or escarpment may experience significantly higher wind speeds than buildings located on level ground. The topographic factor accounts for this effect by multiplying the velocity pressure exposure coefficients in Table 6-5 by K_{zt}, which is determined from Eq. 6-1 in ASCE 6.5.7. ASCE Figure 6-2 contains the three multipliers that are to be used to determine K_{zt} in Eq. 6-1.

5. Gust effect factors, *G* and G_f. Gust effect factors account for the loading effects in the along-wind direction due to wind turbulence-structure interaction. Effects due to wind gust depend on whether a building is rigid or flexible (ASCE 6.5.8). A rigid building has a fundamental natural frequency greater than or equal to 1 Hz (a fundamental period if 1 sec or less), while a flexible building has a fundamental natural frequency less than 1 Hz (a fundamental period exceeding 1 sec) (ASCE 6.2). Fundamental period may be calculated by using the approximate formulas in ASCE 9.5.3.3 or IBC 1617.4.2.

In the case of rigid structures, the gust effect factor *G* may be taken as 0.85 or may be calculated from Eq. 6-2. Equation 6-2 will give a more accurate value for *G*, since it incorporates specific features of the wind environment and building size.

For flexible or dynamically sensitive structures, the gust effect factor G_f is to be determined from Eq. 6-6. This equation accounts for along-wind loading effects due to dynamic amplification. Across-wind loading effects, vortex shedding, instability due to gallop or flutter, or dynamic torsional effects are not considered. As mentioned above, wind tunnel tests or information from recognized literature should be used to determine wind loads on structures susceptible to those effects not covered by G_f.

6. Enclosure classification. According to ASCE 6.5.9, all buildings are to be classified as enclosed, partially enclosed, or open according to the definitions in ASCE 6.2. Such a classification is required in order to determine internal pressure coefficients.

An open building has each wall at least 80 percent open:

$$A_o \geq 0.8 A_g$$

where A_o is the total area of openings in a wall that receives positive external pressure and A_g is the gross area of that wall with total area of openings A_o (see Figure 1-17). By definition, an opening is an aperture or hole in the building envelope that allows air to flow through the building envelope. Glazing in the lower 60 ft of ASCE Category II, III, or IV buildings located in wind borne debris regions (i.e., portions of the hurricane prone regions as defined in ASCE 6.2) shall be assumed to be openings when impact resistant glazing or impact resistant covering is not used (ASCE 6.5.9.3).

A partially enclosed building complies with both of the following conditions (see Figure 1-17):

1. the total area of openings in a wall that receives external positive pressure exceeds the sum of the areas of openings in the balance of the building envelope (walls and roof) by more than 10 percent:

$$A_o > 1.1A_{oi}$$

2. the total area of openings in a wall that receives external positive pressure exceeds 4 sq ft or 1 percent of the area of that wall, whichever is smaller, and the percentage of openings in the balance of the building envelope does not exceed 20 percent:

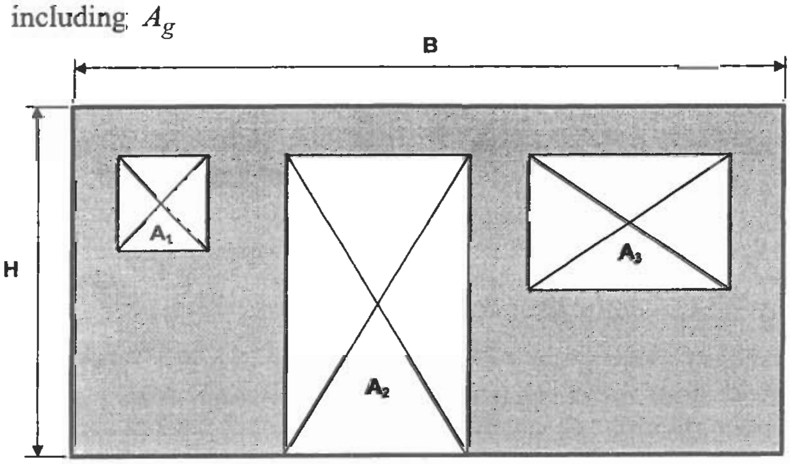
$$A_o \geq \text{smaller of} \begin{cases} 4 \text{ ft}^2 \\ 0.01A_g \end{cases}$$

and

$$\frac{A_{oi}}{A_{gi}} \leq 0.20$$

where A_{oi} = sum of the areas of openings in building envelope (walls and roof) not including A_o

A_{gi} = sum of gross surface areas of the building envelope (walls and roof) not including A_g



$A_g = B \times H$

$A_o = A_1 + A_2 + A_3$

Figure 1-17 Wall Openings in Windward Wall

Buildings that comply with both the open and partially enclosed definitions given above are to be classified as open buildings (ASCE 6.5.9.4). An enclosed building is one that does not comply with the requirements for open or partially enclosed buildings. A building may have large openings which would allow pressure to enter, but also have enough background porosity that the pressure escapes as fast as it enters. In this case, the building is still classified as enclosed. By this definition, many porosity structures with significant (but less than 80 percent) openings on the four faces become classified as enclosed buildings.

7. Internal pressure coefficient, GC_{pi}. According to ASCE 6.5.11.1, internal pressure coefficients are to be determined from ASCE Table 6-7 based on building enclosure classification. Values in Table 6-7 were obtained from wind tunnel tests and other full-scale test data.

8. External pressure coefficients, C_p, GC_{pf}, GC_p. External pressure coefficients are provided for MWFRS, components and cladding, and other structures (ASCE 6.5.11.2).

The pressure coefficients for MWFRS are separated into two groups: one for buildings of all heights (C_p in ASCE Figure 6-3) and one for low-rise buildings with a mean roof height less than or equal to 60 ft (GC_{pf} in ASCE Figure 6-4). A comprehensive discussion on how these coefficients were determined is contained in ASCE Commentary Section C6.5.11.

Combined gust effect factor and external pressure coefficients for components and cladding GC_p are contained in ASCE Figures 6-5 through 6-8.

9. Velocity pressure, q_z. The velocity pressure at height z is determined from Eq. 6-13 in ASCE 6.5.10; this equation essentially converts the basic wind speed V to a velocity pressure:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I$$

where all terms have been defined previously. The constant 0.00256 is the mass density of air for standard atmosphere (temperature of 59 degrees Fahrenheit and sea level pressure of 29.92 in. of mercury). A conversion factor from miles per hour wind speed to feet per second wind speed is also built in, so that wind speed in miles per hour can be directly plugged into the above equation. This constant should be used except where sufficient weather data are available to justify a different value. Average and extreme values of air density are given in ASCE Commentary Table C6-1.

10. Design wind pressure, p. Design wind pressures on the main wind-force-resisting systems of enclosed and partially enclosed buildings are determined in accordance with ASCE 6.5.12.

For rigid buildings of all heights, design wind pressures are calculated from Eq. 6-15:

$$p = qGC_p - q_i(GC_{pi})$$

For flexible buildings, Eq. (6-17) is to be used:

$$p = qG_fC_p - q_i(GC_{pi})$$

where $q = q_z$ for windward walls at height z above ground
$q = q_h$ for leeward walls, side walls, and roof, evaluated at mean roof height h
$q_i = q_h$ for all walls and roofs of enclosed buildings

and all other terms have been defined previously.

Once the design wind pressures have been computed, design wind forces are determined by multiplying the pressures by the appropriate tributary area. For buildings, design pressures are typically determined at floor levels; the corresponding design forces are obtained by multiplying the pressures by the tributary area at that floor level.

ASCE 6.5.12.3 requires that the MWFRS of buildings with mean roof heights greater than 60 ft be designed for the torsional moments resulting from the design wind loads acting in the combinations illustrated in ASCE Figure 6-9. The load combinations shown in the figure reflect surface pressure patterns that have been observed on tall buildings in turbulent wind. Additional information on torsional response due to full and partial loading can be found in ASCE Commentary Section C6.5.12.3.

Design wind pressures on components and cladding are given in ASCE 6.5.12.4 for low-rise buildings with mean roof heights less than or equal to 60 ft and in ASCE 6.5.12.4.2 for buildings with mean roof heights greater than 60 ft. Design wind forces for open buildings and other structures are contained in ASCE 6.5.13.

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CHAPTER 2

OFFICE BUILDING WITH DUAL AND MOMENT-RESISTING FRAME SYSTEMS

2.1 INTRODUCTION

A typical plan and elevation of a 12-story office building is shown in Figure 2-1. The computation of wind and seismic forces according to the 2000 IBC is illustrated below. Typical beams, columns, and walls are designed and detailed for combined effects of gravity, wind, and seismic forces for Seismic Design Categories (SDC) A, C, D, and E.

In the N-S direction, resistance to lateral forces is provided by a combination of shear walls and frames acting together. A shear wall-frame interactive system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls is permitted for SDC A according to IBC Table 1617.6. For SDC C, a dual system with intermediate moment frames and ordinary reinforced concrete shear walls is required, while for SDC D and E, a dual system with special moment frames and special reinforced concrete shear walls must be used.

Resistance to lateral forces in the E-W direction is provided by the flexural action of the beams and columns. An ordinary reinforced concrete moment frame is permitted for SDC A, while intermediate and special reinforced concrete moment frames must be used for SDC C and for SDC D and E, respectively.

It is assumed mainly for simplicity that slabs, beams, columns, and walls have constant cross-sections throughout the height of the building, and that the bases of the lowest story segments are fixed. Although the member dimensions in the following sections are within the practical range, the structure itself is a hypothetical one, and has been chosen mainly for illustrative purposes.

2.2 DESIGN FOR SDC A

2.2.1 Design Data

- Building Location: Miami, FL (zip code 33122)

- Material Properties

  Concrete: $f'_c = 4{,}000$ psi, $w_c = 150$ pcf
  Reinforcement: $f_y = 60{,}000$ psi

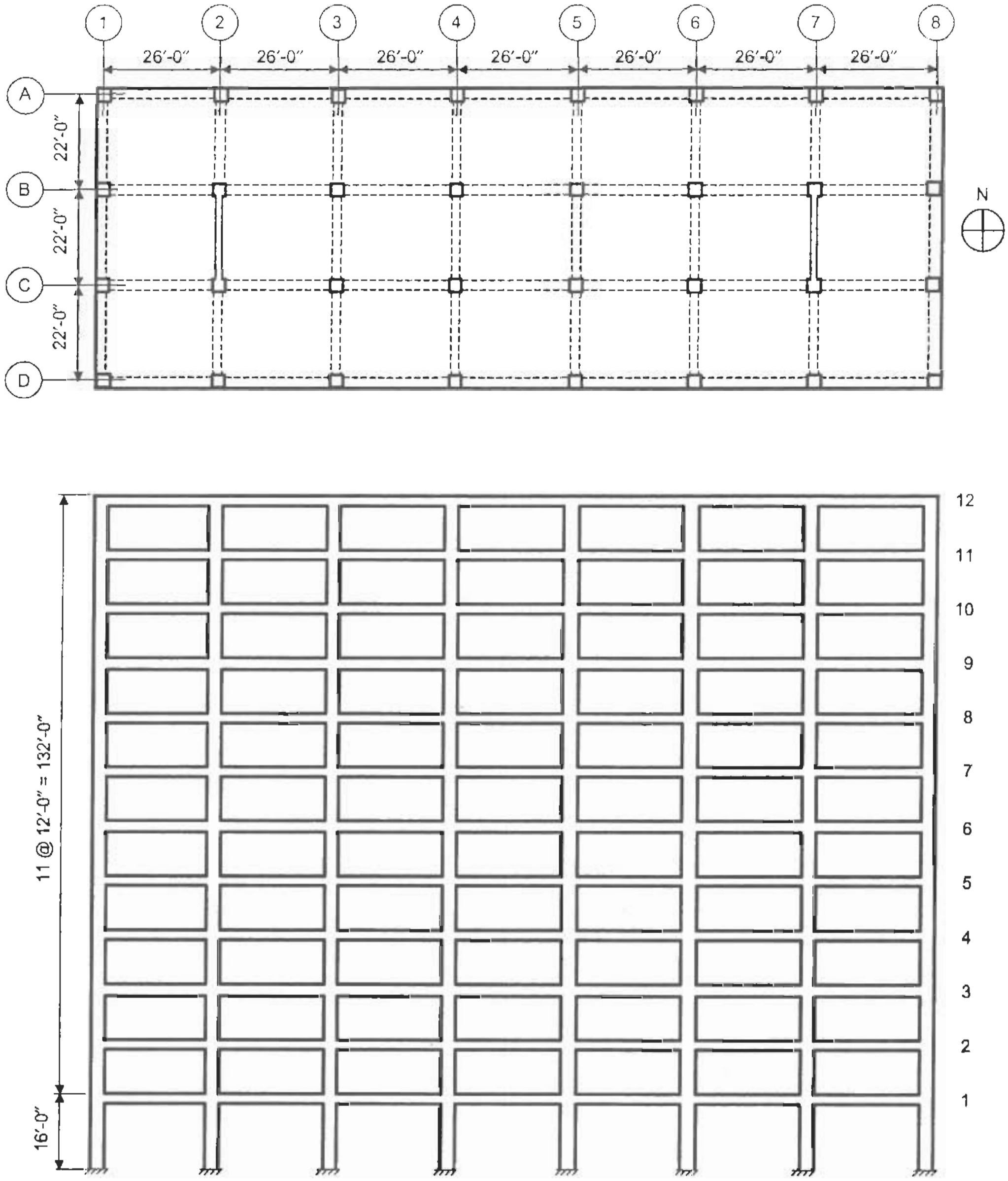


Figure 2-1 Typical Plan and Elevation of Example Building

- Service Loads

Live loads: roof = 20 psf
floor = 50 psf

Superimposed dead loads: **roof** = 10 psf + 200 kips for penthouse
floor = **30** psf (20 psf permanent partitions + 10 psf ceiling, etc.)

- Seismic Design Data

  For zip code 33122: $S_S = 0.065g, S_1 = 0.024g$ [2.1]
  Site Class D (stiff soil profile; IBC Table 1615.1.1)
  For Category (Seismic Use Group) I occupancy, $I_E = 1.0$ (IBC Table 1604.5)

- Wind Design Data

  Basic wind speed = 145 mph (IBC Figure 1609)
  Exposure B (IBC 1609.4)
  For Category I occupancy, $I_W = 1.0$ (IBC Table 1604.5)

- Member Dimensions

  Slab: 8 in.
  Beams: 24 × 24 in.
  Columns: 28 × 28 in.
  Wall thickness: 12 in.

2.2.2 Seismic Load Analysis

Exception 4 of IBC 1614.1 states that structures located where $S_S \le 0.15g$ and $S_1 \le 0.04g$ need only comply with IBC 1616.4, which are design requirements for SDC A. Thus, the minimum lateral force F_x applied simultaneously at each floor level is computed according to IBC Eq. 16-27:

$$F_x = 0.01 w_x$$

where w_x is the portion of the total gravity load W located or assigned to level x and W is defined in IBC 1616.4.1. The seismic forces F_x are summarized in Table 2-1.

Note that the seismic forces are the same in both the N-S and E-W directions, and may be applied separately in each of the two orthogonal directions of the building, i.e., orthogonal effects may be neglected (IBC 1616.4.1).

2.2.3 Wind Load Analysis

According to IBC 1609.1.1, wind forces shall be determined in accordance with Section 6 of ASCE 7 [2.2]. Since the building has a mean roof height greater than 30 ft, the simplified procedure (Method 1) given in ASCE 6.4 cannot be used to determine the wind forces. Similarly, the simplified procedure of IBC 1609.6 must not be used, since the building is taller than 60 ft.

Table 2-1 Seismic Forces for SDC A

Level	Story weight, w_x (kips)	Lateral force, $F_x = 0.01w_x$ (kips)
12	2,205	22.1
11	2,439	24.4
10	2,439	24.4
9	2,439	24.4
8	2,439	24.4
7	2,439	24.4
6	2,439	24.4
5	2,439	24.4
4	2,439	24.4
3	2,439	24.4
2	2,439	24.4
1	2,503	25.0
	Σ	291.1

The example building is regular shaped by the definition in ASCE 6.2, i.e., it has no unusual geometrical irregularity in spatial form. Also, the building does not have response characteristics making it subject to across wind loading, vortex shedding, or instability due to galloping or flutter. It is assumed that the site location is such that channeling effects or buffeting in the wake of upwind obstructions need not be considered. Thus, the analytical procedure (Method 2) of ASCE 6.5 may be used to determine the wind forces.

Design Procedure.

The design procedure outlined in Section 6.5.3 of ASCE 7 is used to determine the wind forces on the building in both the N-S and E-W directions.

1. Basic wind speed, *V*, and wind directionality factor, K_d. Both quantities are determined in accordance with ASCE 6.5.4. As noted above, *V* is equal to 145 mph for Miami according to IBC Figure 1609 or ASCE Figure 6-1.

The wind directionality factor K_d is equal to 0.85 for main wind-force-resisting systems per ASCE Table 6-6 when load combinations specified in Sections 2.3 and 2.4 are used. Note that these load combinations are essentially the same as those in Sections 1605.2 and 1605.3 of the 2000 IBC. It is important to note exception 1 to IBC 1605.2.1, Basic load combinations: load combinations of ACI 9.2 shall be used for concrete structures where combinations do not include seismic forces. The load factors in the ACI 318 combinations are different than those in ASCE 7 and the IBC. The exception goes on to state that for concrete structures designed for wind in accordance with ASCE 7, wind forces are to be divided by the directionality factor. Thus, in the following computations, instead of multiplying and then subsequently dividing the external wind pressures/forces by 0.85, K_d is taken equal to 1.0.

2. Importance factor, I_W. As noted above, I_W is equal to 1.0 for Category I occupancy according to IBC Table 1604.5 and Category II occupancy according to ASCE Table 1-1 (note that IBC Category I and ASCE 7 Category II are the same).

3. Velocity pressure exposure coefficient, K_z. According to ASCE 6.5.6.4, values of K_z are to be determined from ASCE Table 6-5. In lieu of linear interpolation, K_z may be calculated at any height z above ground level from the equations given at the bottom of Table 6-5:

$$K_z = \begin{cases} 2.01\left(\dfrac{15}{z_g}\right)^{2/\alpha} & \text{for } z < 15\text{ ft} \\ 2.01\left(\dfrac{z}{z_g}\right)^{2/\alpha} & \text{for } 15\text{ ft} \le z \le z_g \end{cases}$$

where α = 3-second gust speed power law exponent from ASCE Table 6-4
= 7.0 for Exposure B
z_g = nominal height of the atmospheric boundary layer from ASCE Table 6-4
= 1,200 ft for Exposure B

Values of K_z are summarized in Table 2-2 at the various story heights for the example building.

Table 2-2 Velocity Pressure Exposure Coefficient K_z

Level	Height above ground level, z (ft)	K_z
12	148	1.105
11	136	1.079
10	124	1.051
9	112	1.021
8	100	0.988
7	88	0.953
6	76	0.914
5	64	0.870
4	52	0.820
3	40	0.761
2	28	0.687
1	16	0.585

4. Topographic factor, K_{zt}. The topographic factor is to be determined in accordance with ASCE 6.5.7, Eq. 6-1. Assuming the example building is situated on level ground and not on a hill, ridge, or escarpment, K_{zt} is equal to 1.

5. Gust effect factors, G and G_f. Effects due to wind gust depend on whether a building is rigid or flexible (ASCE 6.5.8). A rigid building has a fundamental natural frequency n_1 greater than or equal to 1 Hz, while a flexible building has a fundamental natural frequency less than 1 Hz (ASCE 6.2).

In lieu of a more exact method, an approximate fundamental period T_a is determined in the N-S and E-W directions using Eq. 16-39 in IBC 1617.4.2.1. The natural frequency is computed by taking the inverse of the period.

$$T_a = C_T (h_n)^{3/4}$$

where C_T = building period coefficient
= 0.030 for moment-resisting frame systems of concrete
= 0.020 for other types of building systems

Thus, in the N-S direction,

$$T_a = 0.020 \times (148)^{3/4} = 0.85 \text{ sec} \quad \text{or} \quad n_1 = 1/0.85 = 1.2 \text{ Hz}$$

Since $n_1 > 1.0$ Hz, the building is considered rigid, and G may be taken equal to 0.85 or may be calculated by Eq. 6-2 (ASCE 6.5.8.1). For simplicity, G is taken as 0.85.

In the E-W direction,

$$T_a = 0.030 \times (148)^{3/4} = 1.27 \text{ sec} \quad \text{or} \quad n_1 = 1/1.27 = 0.79 \text{ Hz} < 1.0 \text{ Hz}$$

Therefore, the gust effect factor G_f must be computed in accordance with Eq. 6-6:

$$G_f = 0.925\left[\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right]$$

$$= 0.925\left[\frac{1 + (1.7 \times 0.25)\sqrt{(3.4^2 \times 0.85^2) + (4.14^2 \times 0.54^2)}}{1 + (1.7 \times 3.4 \times 0.25)}\right] = 0.97$$

where $g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3{,}600 n_1)} + \frac{0.577}{\sqrt{2\ln(3{,}600 n_1)}}$$

$$= \sqrt{2\ln(3{,}600 \times 0.79)} + \frac{0.577}{\sqrt{2\ln(3{,}600\times 0.79)}} = 3.99 + 0.15 = 4.14$$

$I_{\bar{z}}$ = intensity of turbulence at height $\bar{z}$

$$= c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{88.8}\right)^{1/6} = 0.25 \quad \text{(Eq. 6-3 and Table 6-4 for Exposure B)}$$

$\bar{z} = 0.6h \geq z_{min} = 0.6 \times 148 = 88.8 \text{ ft} > z_{min} = 30 \text{ ft}$ (Table 6-4 for Exposure B)

Q = background response

$$= \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{68.33+148}{445.1}\right)^{0.63}}} = 0.85 \quad \text{(Eq. 6-4)}$$

$L_{\bar{z}}$ = integral length scale of turbulence at equivalent height

$$= \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\varepsilon}} = 320\left(\frac{88.8}{33}\right)^{1/3} = 445.1 \text{ ft} \quad \text{(Eq. 6-5 and Table 6-4 for Exposure B)}$$

The resonant response factor R is computed from:

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$$

$$= \sqrt{\frac{1}{0.01} \times 0.07 \times 0.20 \times 0.37[0.53 + (0.47 \times 0.05)]} = 0.54 \quad \text{(Eq. 6-8)}$$

where β = damping ratio (assumed to be 0.01)

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47 \times 2.87}{[1+(10.3\times 2.87)]^{5/3}} = 0.07 \quad \text{(Eq. 6-9)}$$

N_1 = reduced frequency

$$= \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.79 \times 445.1}{122.6} = 2.87 \quad \text{(Eq. 6-10)}$$

$\bar{V}_{\bar{z}}$ = mean hourly wind speed at height $\bar{z}$

$$= \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 0.45\left(\frac{88.8}{33}\right)^{1/4} \times 145\left(\frac{88}{60}\right)$$

$= 122.6$ ft/sec (Eq. 6-12 and Table 6-4 for Exposure B)

$$R_h = \frac{1}{\eta_h} - \frac{1}{2\eta_h^2}(1 - e^{-2\eta_h})$$

$$= \frac{1}{4.39} - \frac{1}{2 \times 4.39^2}\left(1 - e^{-2\times 4.39}\right)$$

$= 0.20$ (Eq. 6-11a)

$$\eta_h = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{4.6 \times 0.79 \times 148}{122.6} = 4.39$$

$$R_B = \frac{1}{\eta_B} - \frac{1}{2\eta_B^2}(1 - e^{-2\eta_B})$$

$$= \frac{1}{2.03} - \frac{1}{2 \times 2.03^2}\left(1 - e^{-2\times 2.03}\right)$$

$= 0.37$ (Eq. 6-11a)

$$\eta_B = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}} = \frac{4.6 \times 0.79 \times 68.33}{122.6} = 2.03$$

$$R_L = \frac{1}{\eta_L} - \frac{1}{2\eta_L^2}(1 - e^{-2\eta_L})$$

$$= \frac{1}{18.29} - \frac{1}{2 \times 18.29^2}\left(1 - e^{-2\times 18.29}\right)$$

$= 0.05$ (Eq. 6-11a)

$$\eta_L = \frac{15.4 n_1 L}{\bar{V}_{\bar{z}}} = \frac{15.4 \times 0.79 \times 184.33}{122.6} = 18.29$$

6. Enclosure classification. It is assumed in this example that the building is enclosed per ASCE 6.5.9 or IBC 1609.2.

7. Internal pressure coefficient, GC_{pi}. According to ASCE 6.5.11.1, internal pressure coefficients are to be determined from Table 6-7 based on building enclosure classification. The building is in a hurricane-prone region. Glazing in the bottom 60 ft of the building is assumed to be debris resistant (ASCE 6.5.9.3). Therefore, for an enclosed building, $GC_{pi} = \pm 0.18$.

8. External pressure coefficients, C_p. External pressure coefficients for main wind-force-resisting systems are given in ASCE 7 Figure 6-3 for this example building. For wind in the N-S direction:

Windward wall: $C_p = 0.8$

Leeward wall (L/B = 68.33/184.33 = 0.37): $C_p = -0.5$

Side wall: $C_p = -0.7$

Roof (h/L = 148/68.33 = 2.17):

$C_p = -1.3$ over entire roof (68.33 ft < h/2 = 74 ft). May be reduced to 0.80 × −1.3 = −1.04 for area greater than 1,000 sq ft per Figure 6-3.

For wind in the E-W direction:

Windward wall: $C_p = 0.8$

Leeward wall (L/B = 184.33/68.33 = 2.70): $C_p = -0.26$

Side wall: $C_p = -0.7$

Roof (h/L = 148/184.33 = 0.80):

$C_p = -1.14$ from windward edge to h/2 = 74 ft

$C_p = -0.78$ from 74 ft to h = 148 ft

$C_p = -0.62$ from 148 ft to 184.33 ft

9. Velocity pressure, q_z. The velocity pressure at height z is determined from Eq. 6-13 in ASCE 6.5.10:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I$$

where all terms have been defined previously. Table 2-3 contains a summary of the velocity pressures for the example building.

10. Design wind pressure, p. Design wind pressures on the main wind-force-resisting systems of enclosed and partially enclosed buildings are determined in accordance with ASCE 6.5.12. For rigid buildings of all heights, design wind pressures are calculated from Eq. 6-15:

$$p = qGC_p - q_i(GC_{pi})$$

Table 2-3 Velocity Pressure q_z (V = 145 mph)

Level	Height above ground level, *z* (ft)	K_z	q_z (psf)
12	148	1.105	59.5
11	136	1.079	58.1
10	124	1.051	56.6
9	112	1.021	54.9
8	100	0.988	53.2
7	88	0.953	51.3
6	76	0.914	49.2
5	64	0.870	46.8
4	52	0.820	44.1
3	40	0.761	40.9
2	28	0.687	37.0
1	16	0.585	31.5

For flexible buildings, Eq. (6-17) is to be used:

$$p = qG_f C_p - q_i(GC_{pi})$$

where $q = q_z$ for windward walls at height *z* above ground

$q = q_h$ for leeward walls, side walls, and roof, evaluated at height *h*

$q_i = q_h$ for all walls and roofs of enclosed buildings

and all other terms have been defined previously.

Table 2-4 contains a summary of the design pressures for wind in the N-S direction (rigid building). The design wind forces at each floor level are given in Table 2-5. It has been assumed in the calculation of the wind forces that the design wind pressure is constant over the tributary height of the floor level. Design wind pressures and forces are contained in Tables 2-6 and 2-7, respectively, for wind acting in the E-W direction (flexible building).

2.2.4 Design for Combined Load Effects

It is evident from Tables 2-1, 2-5, and 2-7 that wind forces will govern the design of the structural members for SDC A. It is important to note, however, that IBC 1609.1.5 requires that seismic detailing requirements of the code be satisfied, even when wind load effects are greater than seismic load effects.

Table 2-4 Design Wind Pressures in N-S Direction (V = 145 mph)

Location	Level	Height above ground level, z (ft)	External Pressure				Internal Pressure		
			q (psf)	G	C_p	qGC_p (psf)	q_i (psf)	GC_{pi}	q_iGC_{pi} (psf)
Windward	12	148	59.5	0.85	0.80	40.5	59.5	± 0.18	± 10.7
	11	136	58.1	0.85	0.80	39.5	59.5	± 0.18	± 10.7
	10	124	56.6	0.85	0.80	38.5	59.5	± 0.18	± 10.7
	9	112	54.9	0.85	0.80	37.4	59.5	± 0.18	± 10.7
	8	100	53.2	0.85	0.80	36.2	59.5	± 0.18	± 10.7
	7	88	51.3	0.85	0.80	34.9	59.5	± 0.18	± 10.7
	6	76	49.2	0.85	0.80	33.4	59.5	± 0.18	± 10.7
	5	64	46.8	0.85	0.80	31.8	59.5	± 0.18	± 10.7
	4	52	44.1	0.85	0.80	30.0	59.5	± 0.18	± 10.7
	3	40	40.9	0.85	0.80	27.8	59.5	± 0.18	± 10.7
	2	28	37.0	0.85	0.80	25.1	59.5	± 0.18	± 10.7
	1	16	31.5	0.85	0.80	21.4	59.5	± 0.18	± 10.7
Leeward	---	All	59.5	0.85	-0.50	-25.3	59.5	± 0.18	± 10.7
Side	---	All	59.5	0.85	-0.70	-35.4	59.5	± 0.18	± 10.7
Roof	---	148	59.5	0.85	-1.04	-52.6	59.5	± 0.18	± 10.7

Table 2-5 Design Wind Forces in N-S Direction (V = 145 mph)

Level	Height above ground level, z (ft)	Tributary Height (ft)	Windward		Leeward		Total Design Wind Force (kips)
			External Design Wind Pressure, q_zGC_p (psf)	Design Wind Force, P* (kips)	External Design Wind Pressure, q_hGC_p (psf)	Design Wind Force, P* (kips)	
12	148	6	40.5	44.7	-25.3	28.0	72.7
11	136	12	39.5	87.4	-25.3	56.0	143.4
10	124	12	38.5	85.1	-25.3	56.0	141.1
9	112	12	37.4	82.6	-25.3	56.0	138.6
8	100	12	36.2	80.0	-25.3	56.0	136.0
7	88	12	34.9	77.1	-25.3	56.0	133.1
6	76	12	33.4	74.0	-25.3	56.0	130.0
5	64	12	31.8	70.4	-25.3	56.0	126.4
4	52	12	30.0	66.4	-25.3	56.0	122.4
3	40	12	27.8	61.6	-25.3	56.0	117.6
2	28	12	25.1	55.6	-25.3	56.0	111.6
1	16	14	21.4	55.3	-25.3	65.3	120.6
						Σ	1,493.5

*$P = qGC_p \times$ Tributary height × 184.33

Table 2-6 Design Wind Pressures in E-W Direction (V = 145 mph)

Location	Level	Height above ground level, z (ft)	External Pressure				Internal Pressure		
			q (psf)	G_f	C_p	qG_fC_p (psf)	q_i (psf)	GC_{pi}	q_iGC_{pi} (psf)
Windward	12	148	59.5	0.97	0.80	46.2	59.5	± 0.18	± 10.7
	11	136	58.1	0.97	0.80	45.1	59.5	± 0.18	± 10.7
	10	124	56.6	0.97	0.80	43.9	59.5	± 0.18	± 10.7
	9	112	54.9	0.97	0.80	42.6	59.5	± 0.18	± 10.7
	8	100	53.2	0.97	0.80	41.3	59.5	± 0.18	± 10.7
	7	88	51.3	0.97	0.80	39.8	59.5	± 0.18	± 10.7
	6	76	49.2	0.97	0.80	38.2	59.5	± 0.18	± 10.7
	5	64	46.8	0.97	0.80	36.3	59.5	± 0.18	± 10.7
	4	52	44.1	0.97	0.80	34.2	59.5	± 0.18	± 10.7
	3	40	40.9	0.97	0.80	31.8	59.5	± 0.18	± 10.7
	2	28	37.0	0.97	0.80	28.7	59.5	± 0.18	± 10.7
	1	16	31.5	0.97	0.80	24.5	59.5	± 0.18	± 10.7
Leeward	---	All	59.5	0.97	-0.26	-15.0	59.5	± 0.18	± 10.7
Side	---	All	59.5	0.97	-0.70	-40.4	59.5	± 0.18	± 10.7
Roof	---	148*	59.5	0.97	-1.14	-65.8	59.5	± 0.18	± 10.7
	---	148†	59.5	0.97	-0.78	-45.0	59.5	± 0.18	± 10.7
	---	148‡	59.5	0.97	-0.62	-35.8	59.5	± 0.18	± 10.7

*from windward edge to 74 ft
†from 74 ft to 148 ft
‡from 148 ft to 184.33 ft

Chapter 19 of the 2000 IBC references ACI 318-99 [2.3] for the design of reinforced concrete members; provisions that differ from ACI 318 are given in italicized text. According to ACI 21.2.1.2, design of members in regions of low seismic risk shall satisfy Chapters 1 through 18 and 22, and no special seismic detailing is required.

A three-dimensional analysis of the building was performed in the N-S and E-W directions for the wind loads contained in Tables 2-5 and 2-7 using SAP2000 [2.4]. In the model, rigid diaphragms were assigned at each floor level, and rigid-end offsets were defined at the ends of the horizontal members so that results were automatically obtained at the faces of the supports. The stiffness properties of the members were input assuming cracked sections. In lieu of a more accurate analysis, the following cracked section properties were used:

- Beams: $I_{eff} = 0.5I_g$
- Columns: $I_{eff} = 0.7I_g$
- Shear walls: $I_{eff} = 0.5I_g$

where I_g is the gross moment of inertia of the section.

According to ASCE 6.5.12.3, the main wind-force-resisting systems of buildings with mean roof height h greater than 60 ft must be designed for the full and partial wind load cases of Figure 6-9 (Cases 1 through 4). These four cases were considered in the three-dimensional analysis.

Table 2-7 Design Wind Forces in E-W Direction (V = 145 mph)

Level	Height above ground level, z (ft)	Tributary Height (ft)	Windward: External Design Wind Pressure, $q_z G_f C_p$ (psf)	Windward: Design Wind Force, P* (kips)	Leeward: External Design Wind Pressure, $q_h G_f C_p$ (psf)	Leeward: Design Wind Force, P* (kips)	Total Design Wind Force (kips)
12	148	6	46.2	18.9	-15.0	6.2	25.1
11	136	12	45.1	37.0	-15.0	12.3	49.3
10	124	12	43.9	36.0	-15.0	12.3	48.3
9	112	12	42.6	35.0	-15.0	12.3	47.3
8	100	12	41.3	33.8	-15.0	12.3	46.1
7	88	12	39.8	32.6	-15.0	12.3	44.9
6	76	12	38.2	31.3	-15.0	12.3	43.6
5	64	12	36.3	29.8	-15.0	12.3	42.1
4	52	12	34.2	28.1	-15.0	12.3	40.4
3	40	12	31.8	26.0	-15.0	12.3	38.3
2	28	12	28.7	23.5	-15.0	12.3	35.8
1	16	14	24.5	23.4	-15.0	14.3	37.7
						Σ	498.9

*$P = qGC_p \times$ Tributary height × 68.33

2.2.4.1 Load Combinations

As noted above, IBC 1605.2.1 requires that the non-seismic load combinations of ACI 9.2 be used for concrete structures. Thus, the following load combinations are utilized in the design of the structural members:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)

where D, L, and W are the effects due to dead, live, and wind loads, respectively. It is important to reiterate that the wind loads were computed utilizing a directionality factor equal to one (IBC 1605.2.1, exception 1).

2.2.4.2 Design of Beam C4-C5

Flexural design.

Table 2-8 contains a summary of the design bending moments and shear forces for beam C4-C5 at the second floor level. Included are the results for full and partial wind load cases of ASCE 7 Figure 6-9. Note that for wind load cases 1 and 2, wind in the E-W direction will cause appreciable reactions in this member.

The required flexural reinforcement is contained in Table 2-9. The provided areas of steel are within the limits prescribed in ACI 10.3.3 for maximum reinforcement and ACI 10.5 for minimum reinforcement. The selected reinforcement satisfies ACI 7.6.1 and 3.3.2 (minimum spacing for concrete placement), ACI 7.7.1 (minimum cover for protection of reinforcement), and ACI 10.6 (maximum spacing for control of flexural cracking).

Shear design.

The largest factored shear force V_u at the support is 50 kips (see Table 2-8). At critical section d = 21.5/12 = 1.8 ft from the face of the support (ACI 11.1.3.1), $V_u = 50 - (4.3 \times 1.8) = 42$ kips, where 4.3 kips/ft is the equivalent factored uniform load on the beam, which is determined as follows (see ACI Figure R13.6.8 for definition of tributary area on beam):

$$\text{Total trapezoidal area tributary to beam} = 2\left\{2\left[\frac{1}{2}\times 11\times 11\right]+(4\times 11)\right\} = 330 \text{ ft}^2$$

$$\text{Dead load} = \left(\frac{8}{12}\times 0.15\times 330\right)+\left(\frac{16\times 24}{144}\times 0.15\times 23.67\right)+\left(\frac{30\times 330}{1{,}000}\right) = 52.4 \text{ kips}$$

$$\text{Live load} = \frac{50\times 330}{1{,}000} = 16.5 \text{ kips}$$

$$\text{Total factored load} = (1.4\times 52.4)+(1.7\times 16.5) = 101.4 \text{ kips}$$

$$\text{Equivalent factored uniform load } w_u = \frac{101.4}{23.67} = 4.3 \text{ kips/ft}$$

Equivalent uniform loads are more convenient to use than the actual trapezoidal loads, especially when determining cutoff points for the flexural reinforcement as illustrated below.

Table 2-8 Summary of Design Bending Moments and Shear Forces for Beam C4-C5 at the 2nd Floor Level (SDC A)

Load Case	Full and Partial Wind Load Cases*	Location	Bending Moment (ft-kips)	Shear Force (kips)
Dead (*D*)		Support	-109	26
		Midspan	75	
Live (*L*)		Support	-16	8
		Midspan	11	
Wind (*W*)	1	Support	± 87	± 7
	2	Support	± 77	± 7
	3	Support	± 65	± 6
	4	Support	± 58	± 5
Load Combinations				
1.4*D* + 1.7*L*		Support	-180	50
		Midspan	124	
0.75(1.4*D* + 1.7*L* + 1.7*W*)	1	Support	-246	46
			-24	29
		Midspan	93	
	2	Support	-233	46
			-37	29
		Midspan	93	
	3	Support	-218	45
			-52	30
		Midspan	93	
	4	Support	-209	44
			-61	31
		Midspan	93	
0.9*D* + 1.3*W*	1	Support	-211	33
			15	14
		Midspan	68	
	2	Support	-198	33
			2	14
		Midspan	68	
	3	Support	-183	31
			-14	16
		Midspan	68	
	4	Support	-174	30
			-23	17
		Midspan	68	

*Cases defined in ASCE 7 Figure 6-9

Table 2-9 Required Flexural Reinforcement for Beam C4-C5 at the 2nd Floor Level (SDC A)

Location	M_u (ft-kips)	A_s^* (in.^{2})	Reinforcement*	ϕM_n (ft-kips)
Support	-246	2.66	5-No. 7	275
Midspan	93	1.72	3-No. 7	169

$$^*A_{s,min} = \frac{3\sqrt{f'_c}\,b_w d}{f_y} = \frac{3\sqrt{4{,}000} \times 24 \times 21.5}{60{,}000} = 1.63 \text{ in.}^2$$ ACI 10.5

$$= \frac{200 b_w d}{f_y} = \frac{200 \times 24 \times 21.5}{60{,}000} = 1.72 \text{ in.}^2 \text{ (governs)}$$

$$A_{s,max} = \rho_{max} b_w d = 0.0214 \times 24 \times 21.5 = 11.04 \text{ in.}^2$$ ACI 10.3.3

The nominal shear strength provided by concrete V_c is computed from ACI Eq. 11-3:

$$V_c = 2\sqrt{f'_c}\,b_w d = 2\sqrt{4{,}000} \times 24 \times 21.5/1{,}000 = 65.3 \text{ kips}$$

Since V_u is greater than $\phi V_c/2 = 0.85 \times 65.3/2 = 27.8$ kips and is less than $\phi V_c = 55.5$ kips, provide minimum shear reinforcement per ACI 11.5.5. Assuming No. 3 stirrups, the required spacing s is determined from Eq. (11-13):

$$s = \frac{A_v f_y}{50 b_w} = \frac{(2 \times 0.11) \times 60{,}000}{50 \times 24} = 11 \text{ in.}$$

According to ACI 11.5.4, the maximum spacing of shear reinforcement is $d/2 = 21.5/2 = 10.75$ in. (governs) or 24 in. Thus, provide No. 3 stirrups @ 10 in. at both ends of the beam. Stirrups can be discontinued at sections where $V_u < \phi V_c/2$. This occurs at 5.7 ft from the face of the support, based on Eq. (9-2) for wind load cases 1 or 2.

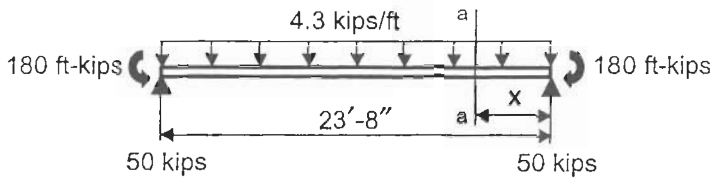
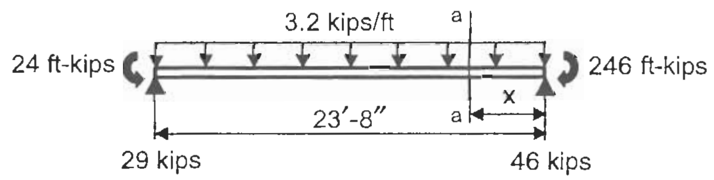
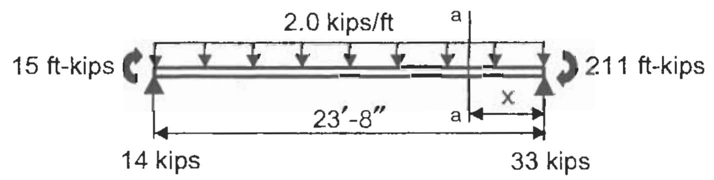
Use 8-No. 3 stirrups at each end of the beam spaced at 10 in. on center with the first stirrup located 2 in. from the face of the support.

Reinforcing bar cutoff points.

The negative reinforcement at the supports is 5-No. 7 bars. The location where 3 of the 5 bars can be terminated is determined. The design flexural strength ϕM_n provided by 2-No. 7 bars is 114 ft-kips.

The load combinations used to determine the cutoff points are shown in Table 2-10. The distance x from the face of the support to the section where the bending moment equals 114 ft-kips is obtained by summing moments about section *a-a* in the figure. For example, for the first load combination:

Table 2-10 Cutoff Location of Negative Bars (SDC A)

Load Combination	Load Diagram	x (ft)
1.4*D* + 1.7*L*		1.4
0.75(1.4*D* + 1.7*L* + 1.7*W*)		3.2
0.9*D* + 1.3*W*		3.3

$$\frac{4.3x^2}{2} - 50x + 180 = 114$$

Solving for *x* gives a distance of 1.4 ft from the face of the support. The cutoff locations for other load combinations are determined in a similar fashion. It is evident from the table that the third load combination governs.

The 3-No. 7 bars must extend a distance d = 21.5 in. (governs) or $12d_b = 12 \times 0.875 = 10.5$ in. beyond the distance *x* (ACI 12.10.3). Thus, from the face of the support, the total bar length must be at least equal to 3.3 + (21.5/12) = 5.1 ft. Also, the bar must extend a full development length ℓ_d beyond the face of the support (ACI 12.10.4), which is determined from ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.3 for top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 7 bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5 + 0.375 + \dfrac{0.875}{2} = 2.3 \text{ in. (governs)} \\ \dfrac{24 - 2(1.5 + 0.375) - 0.875}{2 \times 4} = 2.4 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index = 0 (conservative)

$$\frac{c + K_{tr}}{d_b} = \frac{2.3 + 0}{0.875} = 2.6 > 2.5, \quad \text{use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.3 \times 1.0 \times 1.0 \times 1.0}{2.5} = 37.0$$

$$\ell_d = 37.0 \times 0.875 = 32.3 \text{ in.} = 2.7 \text{ ft} < 5.1 \text{ ft}$$

Thus, the total required length of the 3-No. 5 bars must be at least 5.1 ft beyond the face of the support.

Note that flexural reinforcement shall not be terminated in a tension zone unless one or more of the conditions of ACI 12.10.5 are satisfied. In this case, the point of inflection is approximately 8.7 ft from the face of the support, which is greater than 5.1 ft. Thus, the No. 7 bars cannot be terminated here unless one of the conditions of ACI 12.10.5 is satisfied. In this case, check if the factored shear force V_u at the cutoff point does not exceed $2\phi V_n/3$ (ACI 12.10.5.1). With No. 3 stirrups at 10 in. on center that are provided in this region of the beam, ϕV_n is determined by ACI Eqs. 11-1 and 11-2:

$$\phi V_n = \phi(V_c + V_s) = 0.85 \times \left(65.3 + \frac{0.22 \times 60 \times 21.5}{10}\right) = 79.6 \text{ kips}$$

$$\frac{2}{3}\phi V_n = 53.1 \text{ kips}$$

At 5.1 ft from the face of the support, $V_u = 33 - (2.0 \times 5.1) = 22.8$ kips, which is less than 53.1 kips. Therefore, the 3-No. 7 bars can be terminated at 5.1 ft from the face of the support.

It is assumed in this example that all positive reinforcement is continuous with splices over the columns. Note that the structural integrity provisions of ACI 7.13.2.3 require that at least one-quarter of the positive reinforcement be continuous or spliced over the support with a Class A tension splice for other than perimeter beams that do not have closed stirrups.

Figure 2-2 shows the reinforcement details for beam C4-C5.

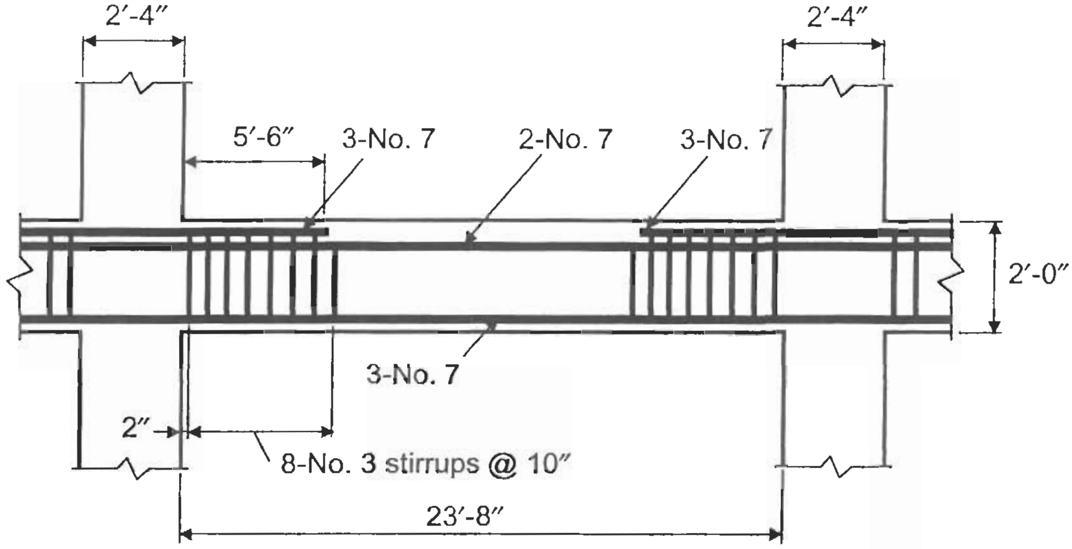


Figure 2-2 Reinforcement Details for Beam C4-C5 at the 2nd Floor Level (SDC A)

2.2.4.3 Design of Column C4

Table 2-11 contains a summary of the design axial forces, bending moments, and shear forces on column C2. It is subjected to biaxial bending due to wind load cases 3 and 4.

Table 2-11 Summary of Design Axial Forces, Bending Moments, and Shear Forces on Column C4 between the 1st and 2nd Floor Levels (SDC A)

Load Case	Full and Partial Wind Load Cases*		Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (*D*)			1,084	0	0
Live (*L*)**			134	0	0
Wind (*W*)	1	N-S	± 11	± 89	± 15
		E-W	0	± 87	± 17
	2	N-S	± 9	± 77	± 13
		E-W	0	± 77	± 15
	3		± 8	± 65	± 13
				± 67	± 12
	4		± 7	± 58	± 12
				± 57	± 11

*Cases defined in ASCE 7 Figure 6-9

** Live load reduced per IBC 1607.9

Table 2-11 (continued) Summary of Design Axial Forces, Bending Moments, and Shear Forces on Column C4 between the 1st and 2nd Floor Levels (SDC A)

Load Combinations	Full and Partial Wind Load Cases		Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
1.4*D* + 1.7*L*			1,745	0	0
0.75(1.4*D* + 1.7*L* + 1.7*W*)	1	N-S	1,323	114	19
			1,295	-114	-19
		E-W	1,309	111	22
	2	N-S	1,321	98	17
			1,298	-98	-17
		E-W	1,309	98	19
	3		1,319	83	17
			1,299	-83	-17
			1,319	85	15
			1,299	-85	-15
	4		1,318	74	15
			1,300	-74	-15
			1,318	73	14
			1,300	-73	-14
0.9*D* + 1.3*W*	1	N-S	990	116	20
			961	-116	-20
		E-W	976	113	22
	2	N-S	987	100	17
			964	-100	-17
		E-W	976	100	20
	3		986	85	17
			965	-85	-17
			986	87	16
			965	-87	-16
	4		985	75	16
			967	-75	-16
			985	74	14
			967	-74	-14

Design for axial force and bending.

Based on the load combinations in Table 2-11, a 28 × 28 in. column with 16-No. 10 bars ($\rho_g = 2.59\%$) is adequate for column C4 supporting the second floor level. The load combination for gravity loads governs the design and slenderness effects need not be considered since P-delta effects were included in the analysis. Also, the provided reinforcement ratio is within the allowable range of 1% and 8% (ACI 10.9.1).

Figure 2-3 contains the interaction diagram cut at an angle of 0 degrees (pure axial load) through the biaxial interaction surface with the governing load combination shown. Other interaction diagrams cut at 44, 45, and 46 degrees through the biaxial interaction surface would show the other load combination points well inside the surface. Note that the angles of the interaction diagrams are obtained by taking the inverse tangent of the ratio of N-S and E-W factored bending moments. For example, for wind load case 1 in the second load combination, the angle of the interaction diagram = $\tan^{-1}(114/111) =$ 46 degrees (see Table 2-11).

ACI 7.6.3 requires that the clear distance between longitudinal bars shall not be less than $1.5d_b = 1.5 \times 1.27 = 1.9$ in. nor 1.5 in. In this case, the clear distance is equal to the following:

$$\frac{28-2\left(1.5+0.375+\frac{1.27}{2}\right)}{4}-1.27=4.475\text{ in.}>1.9\text{ in. O.K.}$$

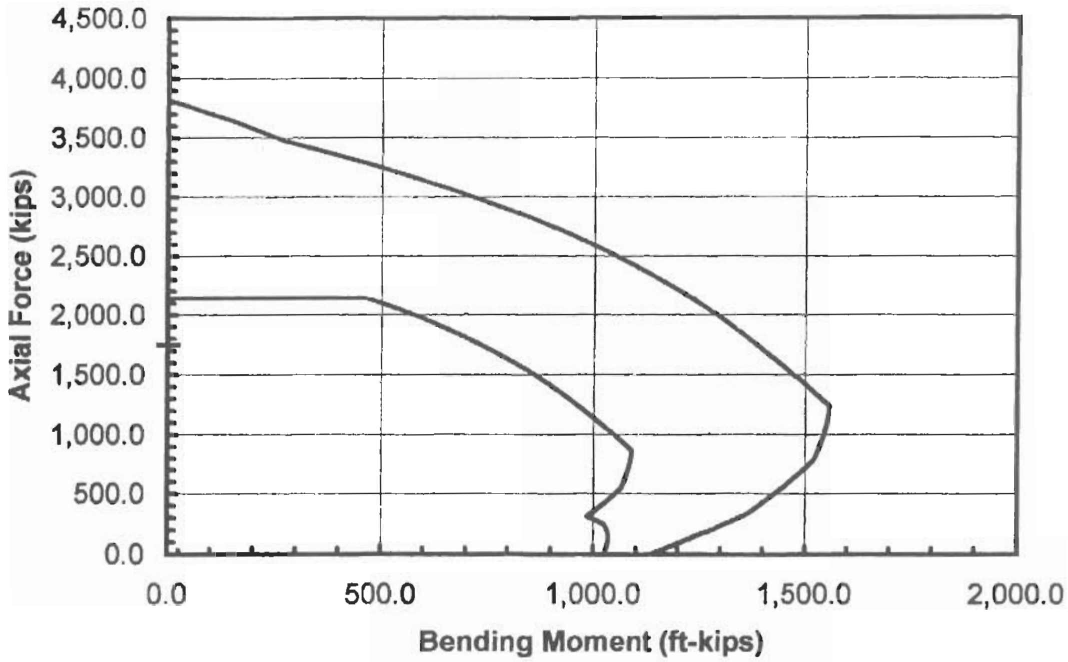


Figure 2-3 Design and Nominal Strength Interaction Diagrams for Column C4 Supporting the 2nd Floor Level (SDC A)

Design for shear.

The shear capacity of the column will be checked using ACI Eq. 11-4 for members subjected to axial compression:

$$V_c = 2\left(1+\frac{N_u}{2{,}000A_g}\right)\sqrt{f'_c}\,b_w d$$

$$= 2\left(1+\frac{976{,}000}{2{,}000\times 28^2}\right)\sqrt{4{,}000}\times 28\times 23.85/1{,}000 = 137.1 \text{ kips}$$

where N_u = 976 kips is the smallest axial force corresponding to the largest shear force V_u = 22 kips on the section (see Table 2-11) and d = 23.85 in. was obtained from a strain compatibility analysis.

Since $\phi V_c/2 = 0.85\times 137.1/2 =$ 58.3 kips > V_u = 22 kips, transverse reinforcement requirements must satisfy ACI 7.10.5. With No. 3 lateral ties, the vertical spacing of the ties must not exceed the least of the following:

- 16(smallest longitudinal bar diameter) = 16 × 1.27 = 20.3 in.
- 48(tie bar diameter) = 48 × 0.375 = 18.0 in. (governs)
- Least column dimension = 28 in.

Use No. 3 ties @ 18 in. with the first tie located vertically not more than 18/2 = 9 in. above the top of the slab and not more than 3 in. below the lowest horizontal reinforcement in the beams (ACI 7.10.5.4 and 7.10.5.5).

Splice length of longitudinal reinforcement.

ACI 12.17 contains special provisions for splices in columns. It can be shown for this example that the bar stress due to all factored load combinations is compressive. Thus, a compression lap splice computed in accordance with ACI 12.17.2.1 could be used. However, a Class A tension lap splice is used in this example.

From ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 7 and larger bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5 + 0.375 + \dfrac{1.27}{2} = 2.5 \text{ in.} \quad \text{(governs)} \\ \dfrac{28 - 2(1.5 + 0.375) - 1.27}{2 \times 4} = 2.9 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index = 0 (conservative)

$$\frac{c + K_{tr}}{d_b} = \frac{2.5 + 0}{1.27} = 2.0 < 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.0 \times 1.0 \times 1.0 \times 1.0}{2.0} = 35.6$$

$$\ell_d = 35.6 \times 1.27 = 45.2 \text{ in.} = 3.8 \text{ ft}$$

Class A splice length = $1.0\ell_d$ = 3.8 ft

Use a 3 ft-10 in. splice length with the splice located just above the floor level.

Reinforcement details for column C4 are shown in Figure 2-4.

2.2.4.4 Design of Shear Wall on Line 7

This section outlines the design of the shear wall on line 7. Table 2-12 contains a summary of the design axial forces, bending moments, and shear forces at the base of the wall. Note that for wind load cases 1 and 2, wind in the N-S direction will cause appreciable reactions in this member.

Design for shear.

The shear strength of the concrete is determined in accordance with ACI 11.10.5 for walls subjected to axial compression:

$$\begin{aligned} V_c &= 2\sqrt{f'_c}hd \\ &= 2\sqrt{4{,}000} \times 12 \times 233.6 / 1{,}000 = 354.6 \text{ kips} \end{aligned}$$

where d is permitted to be taken equal to $0.8\ell_w = 0.8 \times 292 = 233.6$ in. (ACI 11.10.4).

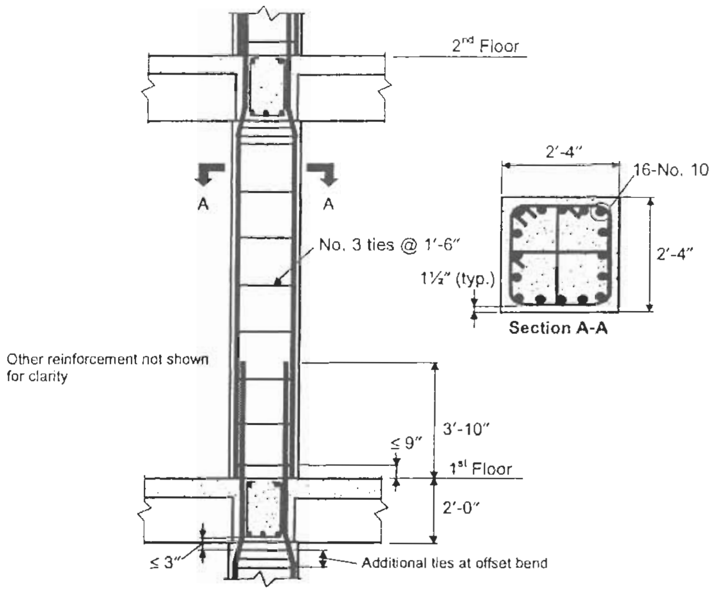


Figure 2-4 Reinforcement Details for Column C4 Supporting the 2nd Floor Level (SDC A)

Table 2-12 Summary of Design Axial Forces, Bending Moments, and Shear Forces at Base of Shear Wall on Line 7 (SDC A)

Load Case	Full and Partial Wind Load Cases*	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)		2,716	0	0
Live (L)		279	0	0
Wind (W)	1	0	± 29,419	± 647
	2	0	± 28,322	± 624
	3	0	± 22,079	± 486
	4	0	± 21,471	± 473
Load Combinations				
$1.4D + 1.7L$		4,277	0	0
$0.75(1.4D + 1.7L + 1.7W)$	1	3,208	37,509	825
	2	3,208	36,111	796
	3	3,208	28,151	620
	4	3,208	27,376	603
$0.9D + 1.3W$	1	2,444	-38,245	-841
	2	2,444	-36,819	-811
	3	2,444	-28,703	-632
	4	2,444	-27,912	-615

*Cases defined in ASCE 7 Figure 6-9

The maximum factored shear force is 841 kips from wind load case 1 in the third load combination (see Table 2-12). Since $\phi V_c = 0.85 \times 354.6 = 301.4$ kips $< V_u = 841$ kips, horizontal shear reinforcement shall be provided in accordance with ACI 11.10.9. With 2 layers of No. 5 bars in the web, the required bar spacing is determined by Eq. 11-33:

$$s_2 = \frac{A_v f_y d}{V_s} = \frac{(2 \times 0.31) \times 60 \times 233.6}{(841/0.85) - 354.6} = 13.7 \text{ in.}$$

Try 2-No. 5 horizontal bars @ 12 in. Note that ACI 14.3.4 requires two layers of reinforcement for walls more than 10 in. thick.

According to ACI 11.10.9.2, ratio of horizontal shear reinforcement shall not be less than 0.0025, and according to ACI 11.10.9.3, spacing of horizontal reinforcement shall not exceed $\ell_w/5 = 292/5 = 58.4$ in., $3h = 3 \times 12 = 36$ in., or 18 in. (governs). For 2-No. 5 horizontal bars spaced at 12 in., which is less than 18 in., the ratio ρ_h of horizontal shear reinforcement area to gross concrete area of vertical section is

$$\rho_h = \frac{2 \times 0.31}{12 \times 12} = 0.0043 > 0.0025 \quad \text{O.K.}$$

Use 2-No. 5 horizontal bars @ 12 in.

The shear strength V_n at any horizontal section must be less than or equal to $10\sqrt{f_c'}hd$ = 1,773 kips (ACI 11.10.3). In this case,

$$V_n = V_c + V_s = 354.6 + \frac{(2 \times 0.31) \times 60 \times 233.6}{12} = 1{,}078.8 \text{ kips} < 1{,}773 \text{ kips} \quad \text{O.K.}$$

The ratio of vertical shear reinforcement area to gross concrete area of horizontal section shall not be less than 0.0025 nor the value obtained from Eq. 11-34 (ACI 11.10.9.4):

$$\rho_n = 0.0025 + 0.5\left(2.5 - \frac{h_w}{\ell_w}\right)(\rho_h - 0.0025)$$

$$= 0.0025 + 0.5\left(2.5 - \frac{148}{24.33}\right)(0.0043 - 0.0025) < 0$$

Thus, $\rho_n = 0.0025$.

According to ACI 11.10.9.5, spacing of vertical shear reinforcement shall not exceed $\ell_w/3 = 292/3 = 97.3$ in., $3h = 3 \times 12 = 36$ in., or 18 in. (governs). For 2-No. 5 vertical bars spaced at 18 in.:

$$\rho_n = \frac{2 \times 0.31}{12 \times 18} = 0.0029 > 0.0025 \quad \text{O.K.}$$

Use 2-No. 5 vertical bars @ 18 in.

The provided vertical and horizontal reinforcement satisfy the requirements of ACI 14.3.2 and 14.3.3 for minimum ratio of vertical and horizontal reinforcement to gross concrete area, respectively, and ACI 14.3.5 for maximum bar spacing.

Design for axial force and bending.

ACI 14.4 requires that walls subjected to axial load or combined flexure and axial load shall be designed as compression members in accordance with ACI 10.2, 10.3, 10.10 through 10.14, 10.17, 14.2, and 14.3 unless the empirical design method of ACI 14.5 or the alternative design method of ACI 14.8 can be used. Clearly, both of these methods cannot be applied in this case, and the wall is designed in accordance with ACI 14.4.

Figure 2-5 contains the interaction diagram of the wall. The wall is reinforced with 16-No. 10 bars in the 28 × 28 in. columns at both ends of the wall and 2-No. 5 vertical bars @ 18 in. in the web. As seen from the figure, the wall is adequate for the load combinations in Table 2-12.

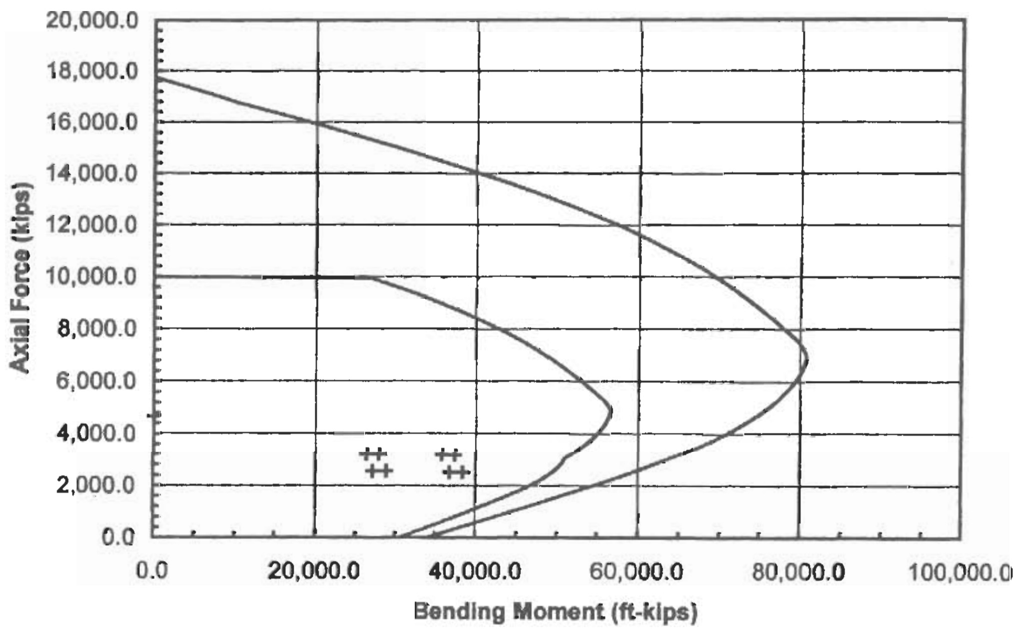


Figure 2-5 Design and Nominal Strength Interaction Diagrams for the Shear Wall Along Line 7 (SDC A)

Splice length of reinforcement.

Class B lap splices are utilized for the longitudinal reinforcement in the columns and the vertical bars in the web. No splices are required for the No. 5 horizontal bars in the web, since full length bars weigh approximately 1.043 × 24.33 = 25.5 lbs. and are easily installed.

For the No. 10 vertical bars in the columns, ℓ_d = 45.2 in., which was computed for column C4 in Section 2.2.4.3 of this publication. Therefore, according to ACI 12.15.1:

$$\text{Class B splice length} = 1.3\ell_d = 1.3 \times 45.2 = 58.8 \text{ in.}$$

Use a 5 ft-0 in. splice length for the No. 10 bars.

For the No. 5 vertical bars in the web, ℓ_d is determined from ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 0.8 for No. 6 and smaller bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 0.75 + 0.625 + \dfrac{0.625}{2} = 1.7 \text{ in. (governs)} \\ \dfrac{1}{2} \times 18 = 9.0 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index = 0

$$\frac{c+K_{tr}}{d_b} = \frac{1.7+0}{0.625} = 2.7 > 2.5, \text{ use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{0.8 \times 1.0 \times 1.0 \times 1.0}{2.5} = 22.8$$

$$\ell_d = 22.8 \times 0.625 = 14.2 \text{ in.} = 1.2 \text{ ft}$$

$$\text{Class B splice length} = 1.3\ell_d = 1.6 \text{ ft}$$

Use a 1 ft-8 in. splice length for the No. 5 bars.

The No. 5 horizontal bars in the web must be developed in the columns. As shown above, the development length ℓ_d is equal to 1.2 ft. This length can be accommodated within the 28 in. = 2.3 ft columns, so that hooks are not needed at the ends of the bars.

Reinforcement details for the shear wall along line 7 are shown in Figure 2-6.

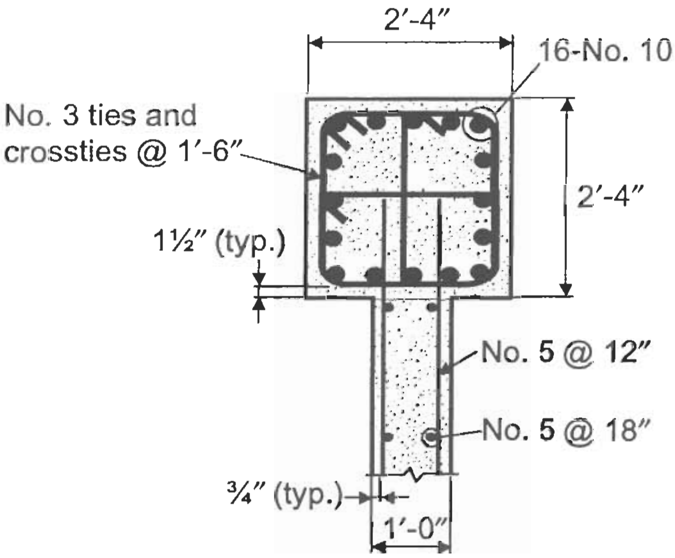


Figure 2-6 Reinforcement Details for Shear Wall Along Line 7 (SDC A)

2.3 DESIGN FOR SDC C

To illustrate the design requirements for Seismic Design Category (SDC) C, the office building in Figure 2-1 is assumed to be located in New York City. Typical beams, columns, and walls are designed and detailed for combined effects of gravity, wind, and seismic forces.

2.3.1 Design Data

- Building Location: New York, NY (zip code 10013)

- Material Properties

  Concrete: $f'_c = 4{,}000$ psi, $w_c = 150$ pcf
  Reinforcement: $f_y = 60{,}000$ psi

- Service Loads

  Live loads: roof = 20 psf
  floor = 50 psf

  Superimposed dead loads: roof = 10 psf + 200 kips for penthouse
  floor = 30 psf (20 psf permanent partitions + 10 psf ceiling, etc.)

- Seismic Design Data

  For zip code 10013: $S_S = 0.424g, S_1 = 0.094g$ [2.1]
  Site Class D (stiff soil profile; IBC Table 1615.1.1)
  For Category (Seismic Use Group) I occupancy, $I_E = 1.0$ (IBC Table 1604.5)

- Wind Design Data

  Basic wind speed = 110 mph (IBC Figure 1609)
  Exposure B (IBC 1609.4)
  For Category I occupancy, $I_W = 1.0$ (IBC Table 1604.5)

- Member Dimensions

  Slab: 8 in.
  Beams: 22 × 22 in.
  Interior columns: 26 × 26 in.
  Edge columns: 24 × 24 in.
  Wall thickness: 12 in.

2.3.2 Seismic Load Analysis

2.3.2.1 Seismic Design Category (SDC)

Analysis procedures for seismic design are given in IBC 1616.6. The appropriate procedure to use depends on the Seismic Design Category (SDC), which is determined in accordance with IBC 1616.3. Structures are assigned to a SDC based on their Seismic Use Group and the design spectral response acceleration parameters S_{DS} and S_{D1}. These parameters can be computed from Eqs. 16-18 and 16-19 in IBC 1615.1.3 or can be obtained from the provisions of IBC 1615.2.5 where site-specific procedures are used as required or permitted by IBC 1615.

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1-second period S_{M1} are determined from IBC Eqs. 16-16 and 16-17, respectively:

$$S_{MS} = F_a S_S = 1.46 \times 0.424 = 0.62g$$

$$S_{M1} = F_v S_1 = 2.40 \times 0.094 = 0.23g$$

where F_a and F_v are the values of site coefficients as a function of site class and of mapped spectral response acceleration at short periods S_S and at 1-second period S_1, respectively. The values of these coefficients for Site Class D in the equations above are contained in IBC Table 1615.1.2(1) for F_a and Table 1615.1.2(2) for F_v. Straight-line interpolation was used to determine F_a.

Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from Eqs. 16-18 and 16-19:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 0.62 = 0.41g$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.23 = 0.15g$$

Tables 1616.3(1) and 1616.3(2) are utilized to determine the SDC. From Table 1616.3(1) for Seismic Use Group I and S_{DS} = 0.41g, the SDC is C. Similarly, from Table 1616.3(2), the SDC is C for $S_{D1} = 0.15g$. Thus, the SDC is C for this building.

2.3.2.2 Seismic Forces

According to IBC 1616.6.2, the equivalent lateral force procedure in IBC 1617.4 may be used to compute the seismic base shear V for structures assigned to SDC C. In a given direction, V is determined from Eq. 16-34:

$$V = C_s W$$

where C_s is the seismic response coefficient determined in accordance with IBC 1617.4.1.1 and W is the effective weight of the structure defined in IBC 1617.4. For the member sizes and superimposed dead loads given above, W = 27,226 kips (see Table 2-13 below).

Seismic Forces in N-S Direction.

In the N-S direction, a dual system is utilized. As a minimum, the dual system must have intermediate reinforced concrete moment frames and ordinary reinforced concrete shear walls to satisfy the provisions of IBC 1910.4.1 for structures assigned to SDC C. For this system, the response modification coefficient $R = 5.5$ and the deflection amplification factor $C_d = 4.5$ (see IBC Table 1617.6).

Approximate period (T_a). The fundamental period of the building T is determined in accordance with IBC 1617.4.2. In lieu of a more exact analysis, an approximate fundamental period T_a is computed from Eq. 16-39 for the dual system:

Building height $h_n = 148$ ft

Building period coefficient $C_T = 0.02$

Period $T_a = C_T (h_n)^{3/4} = 0.020 \times (148)^{3/4} = 0.85$ sec

For comparison purposes, the period was also determined using SAP2000 [2.4], which gave $T = 1.50$ sec. In this example, no further refinement of the period is made.

Seismic base shear (V). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.15}{\left(\frac{5.5}{1.0}\right) \times 0.85} = 0.032$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{0.41}{\left(\frac{5.5}{1.0}\right)} = 0.075$$

Also, C_s must not be less than the following value from Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 0.41 \times 1.0 = 0.018$$

Thus, the value of C_s from Eq. 16-36 governs so that the base shear V in the N-S direction is:

$$V = C_s W = 0.032 \times 27{,}226 = 871 \text{ kips}$$

Vertical distribution of seismic forces. The total base shear is distributed over the height of the building in conformance with Eqs. 16-41 and 16-42:

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

where F_x is the lateral force induced at level x, w_x and w_i are the portions of W assigned to levels x or i, and k is the distribution exponent defined in IBC 1617.4.3. For T = 0.85 sec, k = 1.17 from linear interpolation. The lateral forces F_x and the story shears V_x are contained in Table 2-13.

Table 2-13 Seismic Forces and Story Shears in N-S Direction (SDC C)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
12	2,082	148	720,588	131	131
11	2,281	136	715,098	130	261
10	2,281	124	641,843	116	377
9	2,281	112	569,784	103	480
8	2,281	100	499,028	90	570
7	2,281	88	429,705	78	648
6	2,281	76	361,974	65	713
5	2,281	64	296,044	54	767
4	2,281	52	232,193	42	809
3	2,281	40	170,819	31	840
2	2,281	28	112,538	20	860
1	2,334	16	59,830	11	871
Σ	27,226		4,809,444	871	

Seismic Forces in E-W Direction.

In the E-W direction, a moment-resisting frame system is utilized. As a minimum, this must be an intermediate reinforced concrete moment frame to satisfy the provisions of IBC 1910.4.1 for structures assigned to SDC C. For this system, the response modification coefficient R = 5 and the deflection amplification factor C_d = 4.5, which are found in IBC Table 1617.6.

Approximate period (T_a). Similar to the N-S direction, the fundamental period of the building is determined in accordance with Eq. 16-39 in IBC 1617.4.2:

Building period coefficient $C_T = 0.03$

Period $T_a = C_T (h_n)^{3/4} = 0.030 \times (148)^{3/4} = 1.27$ sec

The period obtained from SAP2000 is $T = 2.36$ sec.

Seismic base shear (V). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.15}{\left(\frac{5.0}{1.0}\right)\times 1.27} = 0.024$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{0.41}{\left(\frac{5.0}{1.0}\right)} = 0.082$$

Also, C_s must not be less than the following value from Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 0.41 \times 1.0 = 0.018$$

Thus, the base shear V in the E-W direction is:

$$V = C_s W = 0.024 \times 27{,}226 = 653 \text{ kips}$$

Vertical distribution of seismic forces. The lateral forces F_x and the story shears V_x are contained in Table 2-14 for seismic forces in the E-W direction. For $T = 1.27$ sec, $k = 1.39$ from linear interpolation.

2.3.2.3 Method of Analysis

A three-dimensional analysis of the building was performed in the N-S and E-W directions for the seismic forces contained in Tables 2-13 and 2-14 using SAP2000 [2.4]. In the model, rigid diaphragms were assigned at each floor level, and rigid-end offsets were defined at the ends of the horizontal members so that results were automatically obtained at the faces of the supports. The stiffness properties of the members were input assuming cracked sections. In lieu of a more accurate analysis, the following cracked section properties were used:

- Beams: $I_{eff} = 0.5 I_g$

- Columns: $I_{eff} = 0.7I_g$
- Shear walls: $I_{eff} = 0.5I_g$

where I_g is the gross moment of inertia of the section. P-delta effects were also considered in the analysis.

Table 2-14 Seismic Forces and Story Shears in E-W Direction (SDC C)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
12	2,082	148	2,163,438	107	107
11	2,281	136	2,107,387	104	211
10	2,281	124	1,853,452	92	303
9	2,281	112	1,608,934	79	382
8	2,281	100	1,374,438	68	450
7	2,281	88	1,150,684	57	507
6	2,281	76	938,547	46	553
5	2,281	64	739,121	37	590
4	2,281	52	553,822	27	617
3	2,281	40	384,582	19	636
2	2,281	28	234,248	12	648
1	2,334	16	110,110	5	653
Σ	27,226		13,218,763	653	

In accordance with IBC 1617.4.4.4, the center of mass was displaced each way from its actual location a distance equal to 5 percent of the building dimension perpendicular to the applied forces to account for accidental torsion in seismic design. Torsional effects need not be amplified, since the building does not possess Type 1a or 1b plan torsional irregularity as defined in Table 1616.5.1 (IBC 1617.4.4.5).

2.3.2.4 Story Drift and P-delta Effects

Story drift determination. Table 2-15 contains the displacements δ_{xe} obtained from the 3-D static, elastic analyses using the design seismic forces in the N-S and E-W directions, including accidental torsional effects. The table also contains the design earthquake displacement δ_x computed by IBC Eq. 16-46:

$$\delta_x = \frac{C_d \delta_{xe}}{I_E}$$

where C_d is the deflection amplification factor in Table 1617.6, which depends on the seismic-force-resisting system. As noted above, C_d is equal to 4.5 for both the dual system in the N-S direction and the moment-resisting frame in the E-W direction.

Table 2-15 Lateral Displacements and Interstory Drifts due to Seismic Forces in N-S and E-W Directions (SDC C)

Story	N-S Direction			E-W Direction		
	δ_{xe} (in.)	δ_x (in.)	Δ (in.)	δ_{xe} (in.)	δ_x (in.)	Δ (in.)
12	1.38	6.21	0.49	5.21	23.45	0.54
11	1.27	5.72	0.54	5.09	22.91	0.91
10	1.15	5.18	0.54	4.89	22.00	1.25
9	1.03	4.64	0.59	4.61	20.75	1.58
8	0.90	4.05	0.58	4.26	19.17	1.84
7	0.77	3.47	0.63	3.85	17.33	2.12
6	0.63	2.84	0.59	3.38	15.21	2.29
5	0.50	2.25	0.58	2.87	12.92	2.48
4	0.37	1.67	0.54	2.32	10.44	2.52
3	0.25	1.13	0.50	1.76	7.92	2.65
2	0.14	0.63	0.36	1.17	5.27	2.61
1	0.06	0.27	0.27	0.59	2.66	2.66

The interstory drifts Δ computed from the δ_x are also contained in the table. For this structure that does not have plan irregularity Type 1a or 1b of Table 1616.5.1, the drift at story level x is determined by subtracting the design earthquake displacement at the center of mass at the bottom of the story from the design earthquake displacement at the center of mass at the top of the story (IBC 1617.4.6.1):

$$\Delta = \delta_x - \delta_{x-1}$$

The design story drifts Δ must not exceed the allowable story drift Δ_a from Table 1617.3 (IBC 1617.3). For Seismic Use Group I, $\Delta_a = 0.020h_{sx}$ where h_{sx} is the story height below level x. Thus, for the 12-ft story heights, $\Delta_a = 0.020 \times 12 \times 12 =$ 2.88 in., and for the 16-ft story height at the first level, $\Delta_a = 3.84$ in. It is evident from Table 2-15 that for all stories, the lateral drifts obtained from the prescribed lateral forces in both directions are less than the limiting values.

P-delta effects. As noted above, P-delta effects were automatically considered in the analysis using SAP2000. The provisions of IBC 1617.4.6.2 are illustrated in Section 2.4.2.4 of this publication for SDC D.

2.3.3 Wind Load Analysis

2.3.3.1 Wind Forces

According to IBC 1609.1.1, wind forces shall be determined in accordance with Section 6 of ASCE 7 [2.2]. Since the building has a mean roof height greater than 30 ft, the simplified procedure (Method 1) given in ASCE 6.4 cannot be used to determine the wind forces. Similarly, the simplified procedure of IBC 1609.6 must not be used, since the building is taller than 60 ft. As was discussed in Section 2.2.3 of this publication, the analytical procedure (Method 2) of ASCE 6.5 may be used to determine the wind forces.

Details on how to compute the wind forces in both the N-S and E-W directions are given in Section 2.2.3 of this publication. In this example, the wind velocity is 110 mph. A summary of the design wind forces in both directions at all floor levels is contained in Table 2-16. Once again it is important to note that the wind directionality factor K_d has been taken equal to 1.0 (see Exception 1 in IBC 1605.2.1).

2.3.3.2 Method of Analysis

Similar to the seismic analysis, a three-dimensional analysis of the building was performed in the N-S and E-W directions for the wind forces contained in Tables 2-16 using SAP2000. The modeling assumptions utilized for the seismic analysis were also used for the wind analysis.

Table 2-16 Design Wind Forces in N-S and E-W Directions (V = 110 mph)

Level	Height above ground level, z (ft)	Total Design Wind Force N-S (kips)	Total Design Wind Force E-W (kips)
12	148	41.7	14.3
11	136	82.2	28.2
10	124	80.9	27.6
9	112	79.5	27.0
8	100	78.0	26.4
7	88	76.3	25.7
6	76	74.5	24.9
5	64	72.5	24.1
4	52	70.1	23.1
3	40	67.4	21.9
2	28	64.0	20.5
1	16	69.1	21.6
	Σ	856.2	285.3

According to ASCE 6.5.12.3, main wind-force-resisting systems of buildings with mean roof height h greater than 60 ft must be designed for the full and partial wind load cases of Figure 6-9 (Cases 1 through 4). These four cases were considered in the three-dimensional analysis.

2.3.4 Design for Combined Load Effects

2.3.4.1 Load Combinations

Basic load combinations for strength design are given in IBC 1605.2.1. As noted above, the first exception in this section requires that the non-seismic load combinations of ACI 9.2 be used for concrete structures. Thus, the following load combinations are utilized in the design of the structural members:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)
4. $1.2D + f_1 L + 1.0E$ (Formula 16-5)
5. $0.9D + 1.0E$ (Formula 16-6)

where D, L, W, and E are the effects due to dead, live, wind, and seismic loads, respectively, and f_1 is equal to either 1.0 for places of public assembly, for live loads in excess of 100 psf, and for parking garage live load, or is equal to 0.5 for other live loads.

The seismic load effect E for use in Formula 16-5, which is the combined effect of horizontal and vertical earthquake-induced forces, is computed from Eq. 16-28 where the effects of gravity and seismic ground motion are additive:

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of horizontal seismic forces
ρ = redundancy coefficient
= 1.0 for structures assigned to SDC A, B, or C (IBC 1617.2.1)

Similarly, E for use in Formula 16-6 is computed from Eq. 16-29 where the effects of gravity and seismic ground motion counteract:

$$E = \rho Q_E - 0.2 S_{DS} D$$

Substituting $S_{DS} = 0.41g$ and $\rho = 1.0$ into the equations for E, and then substituting E along with $f_1 = 0.5$ into load combinations 4 and 5 above results in the following:

4a. $1.2D + 0.5L + 1.0Q_E + (0.2 \times 0.41)D = 1.28D + 0.5L + Q_E$

4b. $1.2D + 0.5L + 1.0Q_E - (0.2 \times 0.41)D = 1.12D + 0.5L + Q_E$

5a. $0.9D + 1.0Q_E + (0.2 \times 0.41)D = 0.98D + Q_E$

5b. $0.9D + 1.0Q_E - (0.2 \times 0.41)D = 0.82D + Q_E$

Values of Q_E are obtained from the structural analysis for both sidesway to the left and to the right based on the code-prescribed horizontal seismic forces acting on the building. Also, the special seismic load combinations in IBC 1605.4 are not applicable in this example.

2.3.4.2 Design of Beam C4-C5

Flexural design.

Comparing the seismic forces in Table 2-14 to the wind forces in Table 2-16, it is clear that seismic effects will govern the design in the E-W direction in this example. Thus, Table 2-17 contains a summary of the governing design bending moments and shear forces for beam C4-C5 at the second floor level due to gravity and seismic forces only.

Table 2-17 Summary of Design Bending Moments and Shear Forces for Beam C4-C5 at the 2nd Floor Level (SDC C)

Load Case	Location	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	Support	-104	25
	Midspan	72	
Live (L)	Support	-16	8
	Midspan	11	
Seismic (Q_E)	Support	± 107	± 9
Load Combinations			
$1.4D + 1.7L$	Support	-173	49
	Midspan	120	
$1.28D + 0.5L + Q_E$	Support	-248	45
	Midspan	98	
$0.82D + Q_E$	Support	22	12
	Midspan	59	

Requirements for intermediate moment frames are given in ACI 21.10. The factored axial load on the member, which is negligible, is less than $A_g f_c' / 10$; thus, the provisions of ACI 21.10.4 for beams must be satisfied. All other applicable provisions in Chapters 1 through 18 of ACI 318-99 are to be satisfied as well.

The required flexural reinforcement is contained in Table 2-18. The provided areas of steel are within the limits prescribed in ACI 10.3.3 for maximum reinforcement and ACI 10.5 for minimum reinforcement. The selected reinforcement satisfies ACI 7.6.1 and 3.3.2 (minimum spacing for concrete placement), ACI 7.7.1 (minimum cover for protection of reinforcement), and ACI 10.6 (maximum spacing for control of flexural cracking).

Table 2-18 Required Flexural Reinforcement for Beam C4-C5 at the 2nd Floor Level (SDC C)

Location	M_u (ft-kips)	A_s^* (in.^{2})	Reinforcement*	ϕM_n (ft-kips)
Support	-248	3.01	4-No. 8	259
	22	1.43	3-No. 7	152
Midspan	120	1.43	3-No. 7	152

$$^*A_{s,min} = \frac{3\sqrt{f'_c}\,b_w d}{f_y} = \frac{3\sqrt{4{,}000}\times 22\times 19.5}{60{,}000} = 1.36 \text{ in.}^2$$

ACI 10.5

$$= \frac{200 b_w d}{f_y} = \frac{200\times 22\times 19.5}{60{,}000} = 1.43 \text{ in.}^2 \text{ (governs)}$$

$$A_{s,max} = \rho_{max} b_w d = 0.0214\times 22\times 19.5 = 9.18 \text{ in.}^2$$

ACI 10.3.3

ACI 21.10.4.1 requires that the positive moment strength at the face of the joint be greater than or equal to 33% of the negative moment strength at that location. This is satisfied, since 152 ft-kips > 259/3 = 86 ft-kips. Also, the negative or positive moment strength at any section along the length of the member must be greater than or equal to 20% of the maximum moment strength provided at the face of either joint. In this case, 20% of the maximum moment strength is equal to 259/5 = 52 ft-kips. Providing 2-No. 8 bars (ϕM_n = 134 ft-kips) or 2-No. 7 bars (ϕM_n = 103 ft-kips) satisfies this provision. However, to satisfy the minimum reinforcement requirement of ACI 10.5 (i.e., minimum A_s = 1.43 in.^{2}), a minimum of 2-No.8 bars (A_s = 1.58 in.^{2}) or 3-No. 7 bars (A_s = 1.80 in.^{2}) must be provided as required.

Shear design.

Shear requirements for beams, columns, and two-way slabs in intermediate moment frames are contained in ACI 21.10.3. Design shear strength shall not be less than either (a) the sum of the shear associated with development of nominal moment strength at each end of the clear span and shear due to factored gravity loads or (b) the maximum shear obtained from load combinations that include E, where E is taken to be twice that prescribed by the governing code. In this example, the first of the two options is utilized.

The largest shear force associated with seismic effects is obtained from the second of the three load combinations in Table 2-17. Figure 2-7 shows the beam and shear forces due to gravity loads plus nominal moment strengths for sidesway to the right. Due to the symmetric distribution of longitudinal reinforcement in the beam, sidesway to the left gives the same maximum shear force. The equivalent factored uniform loads on the beam are determined as follows (see ACI Figure R13.6.8 for definition of tributary area on beam):

$$\text{Total trapezoidal area tributary to beam} = 2\left\{2\left[\frac{1}{2}\times 11\times 11\right]+(4\times 11)\right\} = 330 \text{ ft}^2$$

$$\text{Dead load} = \left(\frac{8}{12}\times 0.15\times 330\right)+\left(\frac{14\times 22}{144}\times 0.15\times 23.83\right)+\left(\frac{30\times 330}{1{,}000}\right) = 50.6 \text{ kips}$$

$$w_D = \frac{50.6}{23.83} = 2.1 \text{ kips/ft}$$

$$\text{Live load} = \frac{50\times 330}{1{,}000} = 16.5 \text{ kips}\,;\; w_L = \frac{16.5}{23.83} = 0.7 \text{ kips/ft}$$

$$w_u = 1.28 w_D + 0.5 w_L = (1.28\times 2.1)+(0.5\times 0.7) = 3.0 \text{ kips/ft}$$

The maximum combined shear force as shown in Figure 2-7 (54.9 kips) is larger than the maximum shear force obtained from the structural analysis (49 kips; see Table 2-17).

The nominal shear strength provided by concrete V_c is computed from ACI Eq. 11-3:

$$V_c = 2\sqrt{f'_c}\,b_w d = 2\sqrt{4{,}000}\times 22\times 19.5/1{,}000 = 54.3 \text{ kips}$$

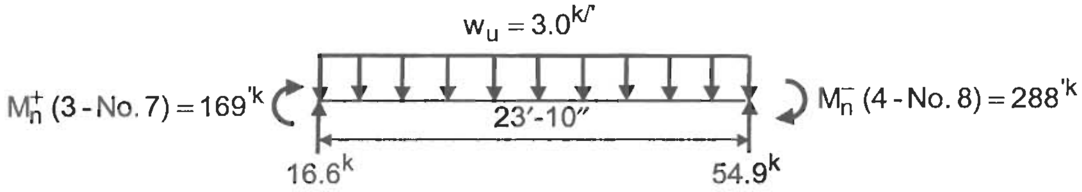


Figure 2-7 Design Shear Forces for Beam C4-C5 (SDC C)

Since V_u = 54.9 kips is greater than $\phi V_c = 0.85\times 54.3 = 46.1$ kips, provide shear reinforcement in accordance with ACI 11.5.6. Assuming No. 3 stirrups, the required spacing s is determined from Eq. (11-15):

$$s = \frac{A_v f_y d}{V_s} = \frac{(2\times 0.11)\times 60\times 19.5}{(54.9/0.85) - 54.3} = 25 \text{ in.}$$

According to ACI 21.10.4.2, the maximum spacing of stirrups over the length $2h = 2 \times 22$ = 44 in. from the face of the support at each end of the member is the smaller of the following:

- $d/4$ = 19.5/4 = 4.9 in. (governs)
- 8(diameter of smallest longitudinal bar) = 8 × 0.875 = 7.0 in.
- 24(diameter of stirrup bar) = 24 × 0.375 = 9.0 in.
- 12 in.

Use 12-No. 3 stirrups at each end of the beam spaced at 4 in. on center with the first stirrup located 2 in. from the face of the support. For the remainder of the beam, the maximum stirrup spacing is $d/2$ = 9.8 in. (ACI 21.10.4.3). Use No. 3 stirrups @ 9 in. for the remainder of the beam.

Reinforcing bar cutoff points.

The negative reinforcement at the supports is 4-No. 8 bars. The location where 2 of the 4 bars can be terminated is determined.

The third load combination is used to determine the cutoff point of the 2-No. 8 bars (0.82 times the dead load in combination with the nominal flexural strengths M_n at the ends of the member), since this combination produces the longest bar lengths. The design flexural strength ϕM_n provided by 2-No. 8 bars is 134 ft-kips. Therefore, the 2-No. 8 bars can be terminated after the required moment strength M_u has been reduced to 134 ft-kips.

The distance x from the support to the location where the moment is equal to 134 ft-kips can readily be determined by summing moments about section a-a in Figure 2-8:

$$\frac{1.7x^2}{2} - 39.4x + 288 = 134$$

Solving for x gives a distance of 4.3 ft from the face of the support.

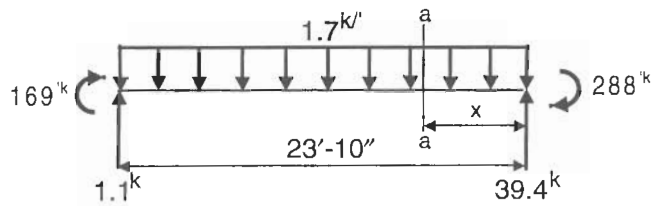


Figure 2-8 Cutoff Location of Negative Bars (SDC C)

The 2-No. 8 bars must extend a distance d = 19.5 in. (governs) or $12d_b = 12 \times 1.0 = 12$ in. beyond the distance x (ACI 12.10.3). Thus, from the face of the support, the total bar length must be at least equal to 4.3 + (19.5/12) = 5.9 ft. Also, the bars must extend a full development length ℓ_d beyond the face of the support (ACI 12.10.4), which is determined from ACI Eq. (12-1):

$$\frac{\ell_d}{d_b} = \frac{3}{40}\frac{f_y}{\sqrt{f_c'}}\frac{\alpha\beta\gamma\lambda}{\left(\frac{c+K_{tr}}{d_b}\right)}$$

where α = reinforcement location factor = 1.3 for top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 8 bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5 + 0.375 + \dfrac{1.0}{2} = 2.4 \text{ in. (governs)} \\ \dfrac{22 - 2(1.5 + 0.375) - 1.0}{2 \times 3} = 2.9 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index = 0 (conservative)

$$\frac{c + K_{tr}}{d_b} = \frac{2.4 + 0}{1.0} = 2.4 < 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.3 \times 1.0 \times 1.0 \times 1.0}{2.4} = 38.5$$

$$\ell_d = 38.5 \times 1.0 = 38.5 \text{ in.} = 3.2 \text{ ft} < 5.9 \text{ ft}$$

Thus, the total required length of the 3-No. 8 bars must be at least 5.9 ft beyond the face of the support.

Flexural reinforcement shall not be terminated in a tension zone unless one or more of the conditions of ACI 12.10.5 are satisfied. In this case, the point of inflection is approximately 9.1 ft from the face of the support, which is greater than 5.9 ft. Thus, the No. 8 bars cannot be terminated here unless one of the conditions of ACI 12.10.5 is satisfied. In this case, check if the factored shear force V_u at the cutoff point does not exceed $2\phi V_n / 3$ (ACI 12.10.5.1). With No. 3 stirrups at 9 in. on center that are provided in this region of the beam, ϕV_n is determined by ACI Eqs. 11-1 and 11-2:

$$\phi V_n = \phi(V_c + V_s) = 0.85 \times \left(54.3 + \frac{0.22 \times 60 \times 19.5}{9} \right) = 70.5 \text{ kips}$$

$$\frac{2}{3}\phi V_n = 47.0 \text{ kips}$$

At 5.9 ft from the face of the support, $V_u = 39.4 - (1.7 \times 5.9) = 29.4$ kips, which is less than 47.0 kips. Therefore, the 2-No. 8 bars can be terminated at 5.9 ft from the face of the support.

It is assumed in this example that all positive reinforcement is continuous with splices over the columns. Note that the structural integrity provisions of ACI 7.13.2.3 require that at least one-quarter of the positive reinforcement be continuous or spliced over the support with a Class A tension splice for other than perimeter beams that do not have closed stirrups.

Figure 2-9 shows the reinforcement details for beam C4-C5.

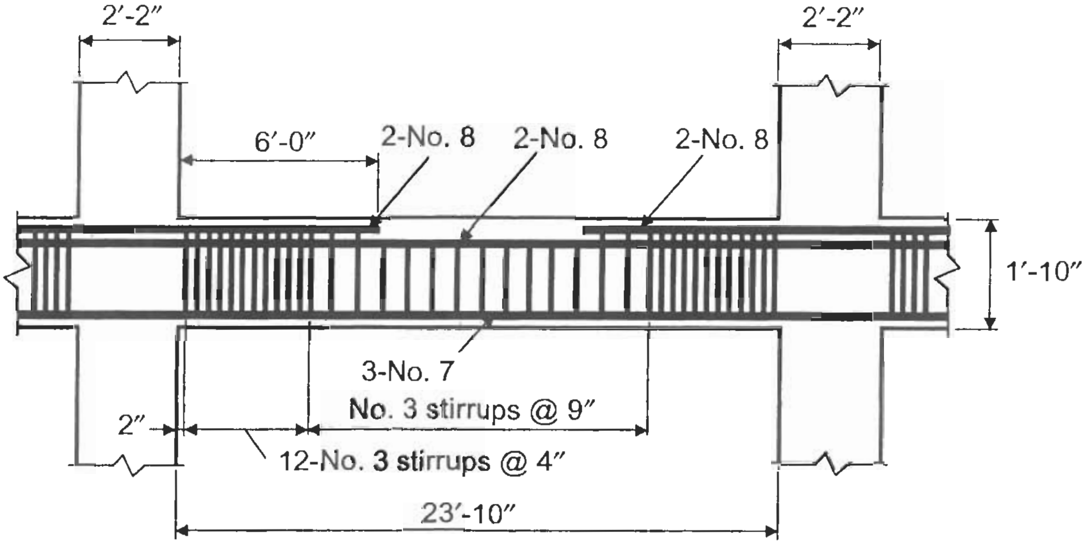


Figure 2-9 Reinforcement Details for Beam C4-C5 at the 2nd Floor Level (SDC C)

2.3.4.3 Design of Column C4

Table 2-19 contains a summary of the design axial forces, bending moments, and shear forces on this column for gravity and seismic loads. Since the factored compressive axial loads exceed $A_g f_c' / 10 = 26^2 \times 4 / 10 =$ 270 kips, the provisions of ACI 21.10.5 must be satisfied, unless spiral reinforcement conforming to Eq. 10-6 is used. In this example, transverse reinforcement consists of ties and crossties.

Design for axial force and bending.

Based on the governing load combinations in Table 2-19, a 26 × 26 in. column with 12-No. 10 bars ($\rho_g = 2.25\%$) is adequate for column C4 supporting the second floor level. Figure 2-10 contains the interaction diagram for this column. As noted above, slenderness effects need not be considered since P-delta effects were included in the analysis. Also, the provided reinforcement ratio is within the allowable range of 1% and 8% (ACI 10.9.1).

Table 2-19 Summary of Design Axial Forces, Bending Moments, and Shear Forces on Column C4 between the 1st and 2nd Floor Levels (SDC C)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	1,027	0	0
Live (L)*	134	0	0
Seismic (Q_E)	0	± 108	± 20
Load Combinations			
$1.4D + 1.7L$	1,666	0	0
$1.28D + 0.5L + Q_E$	1,382	108	20
$0.82D + Q_E$	842	-108	-20

* Live load reduced per IBC 1607.9

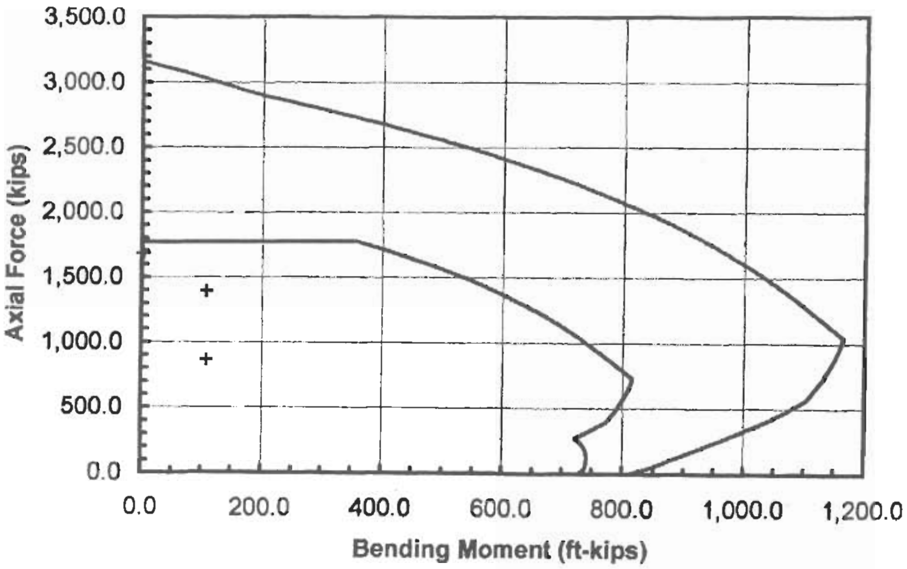


Figure 2-10 Design and Nominal Strength Interaction Diagrams for Column C4 Supporting the 2nd Floor Level (SDC C)

ACI 7.6.3 requires that the clear distance between longitudinal bars shall not be less than $1.5d_b = 1.5 \times 1.27 = 1.9$ in. nor 1.5 in. In this case, assuming No. 3 ties, the clear distance is equal to the following:

$$\frac{26-2\left(1.5+0.375+\dfrac{1.27}{2}\right)}{3}-1.27=5.7 \text{ in.} > 1.9 \text{ in. O.K.}$$

Design for shear.

Similar to beams, columns in intermediate moment frames must satisfy the shear requirements in ACI 21.10.3. The second of the two options in that section is utilized here to determine the design shear strength.

From Table 2-19, the maximum shear force V_u obtained from design load combinations that include E is 20 kips. According to ACI 21.10.3(b), the design shear force must be two times this value, which is 40 kips.

The shear capacity of the column will be checked using ACI Eq. 11-4 for members subjected to axial compression:

$$V_c = 2\left(1+\frac{N_u}{2{,}000A_g}\right)\sqrt{f_c'}b_w d$$

$$= 2\left(1+\frac{842{,}000}{2{,}000\times 26^2}\right)\sqrt{4{,}000}\times 26\times 21.2/1{,}000 = 113.1 \text{ kips}$$

where N_u = 842 kips is the smallest axial force corresponding to the largest shear force on the section (see Table 2-19) and d = 21.2 in. was obtained from a strain compatibility analysis.

Since $V_u < \phi V_c/2 = 0.85\times 113.1/2 = 48.1$ kips, transverse reinforcement requirements would generally have to satisfy ACI 7.10.5. However, for intermediate moment frames, the requirements in ACI 21.10.5.1 take precedence. These requirements are intended primarily to confine the concrete within the core and provide lateral support for the longitudinal reinforcement. For No. 3 ties, the vertical spacing s_o must not exceed the least of the following over a length ℓ_o measured from the joint face:

- 8(smallest longitudinal bar diameter) = 8 × 1.27 = 10.2 in.
- 24(tie bar diameter) = 24 × 0.375 = 9.0 in. (governs)
- Least column dimension/2 = 26/2 = 13 in.
- 12 in.

where ℓ_o is the largest of the following:

- Clear span/6 = [(12 × 12) – 22]/6 = 20.3 in.
- Maximum cross-sectional dimension of member = 26 in. (governs)
- 18 in.

Use 6-No. 3 ties and crossties @ 9 in. with the first tie located at 1 in. (< $s_o/2 = 4.5$ in; ACI 21.10.5.2) from the joint face above the 1st floor level. The 9 in. spacing is used over the lap splice length of 46 in. (> ℓ_o = 26 in.), which is determined below. Below the second floor level, 5-No. 3 ties and crossties are used. For the remainder of the column, tie spacing shall not exceed $2s_o$ = 18 in. (ACI 21.10.5.4).

For comparison purposes, the magnitude of the design shear force is computed according to ACI 21.10.3(a). The factored design axial load P_u = 842 kips develops the largest nominal moment strength of the column, which is 1,147 ft-kips (see the nominal strength interaction diagram in Figure 2-10). For a clear column height of 12 – (22/12) = 10.17 ft, the factored shear force V_u is:

$$V_u = \frac{2 \times 1{,}147}{10.17} = 225.6 \text{ kips}$$

This is approximately 5.6 times the design shear force obtained from ACI 21.10.3(b). In this case, No. 4 ties and crossties spaced at 5 in. would be required over the entire length of the column.

ACI 21.10.5.3 requires that joint reinforcement in intermediate moment frames conform to ACI 11.11.2. Since this beam-column joint is part of the primary seismic load-resisting system, lateral reinforcement in the joint must not be less than that computed by Eq. (11-13). For No. 3 ties with one crosstie, the required spacing is:

$$s = \frac{A_v f_y}{50 b_w} = \frac{(3 \times 0.11) \times 60{,}000}{50 \times 26} = 15.2 \text{ in.}$$

For detailing simplicity, continue the 9 in. spacing at the column ends through the joint.

Splice length of longitudinal reinforcement.

ACI 12.17 contains special provisions for splices in columns. It can be shown for this example that the bar stress due to all factored load combinations is compressive. Thus, a compression lap splice computed in accordance with ACI 12.17.2.1 could be used. However, a Class A tension lap splice is used in this example.
From ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 7 and larger bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5 + 0.375 + \dfrac{1.27}{2} = 2.5 \text{ in.} \quad \text{(governs)} \\ \dfrac{26 - 2(1.5 + 0.375) - 1.27}{2 \times 3} = 3.5 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index = 0 (conservative)

$$\frac{c + K_{tr}}{d_b} = \frac{2.5 + 0}{1.27} = 2.0 < 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.0 \times 1.0 \times 1.0 \times 1.0}{2.0} = 35.6$$

$$\ell_d = 35.6 \times 1.27 = 45.2 \text{ in.} = 3.8 \text{ ft}$$

Class A splice length = $1.0\ell_d$ = 3.8 ft

Use a 3 ft-10 in. splice length with the splice located just above the floor level.

Reinforcement details for column C4 are shown in Figure 2-11.

2.3.4.4 Design of Shear Wall on Line 7

This section outlines the design of the shear wall on line 7. Unlike the E-W direction, design load combinations for wind need to be considered in the N-S direction. Table 2-20 contains a summary of the governing design axial forces, bending moments, and shear forces at the base of the wall. Note that for wind load cases 1 and 2, wind in the N-S direction will cause appreciable reactions in this member.

As noted above, this is an ordinary reinforced concrete shear wall; thus, the provisions of ACI Chapters 1 through 18 must be satisfied.

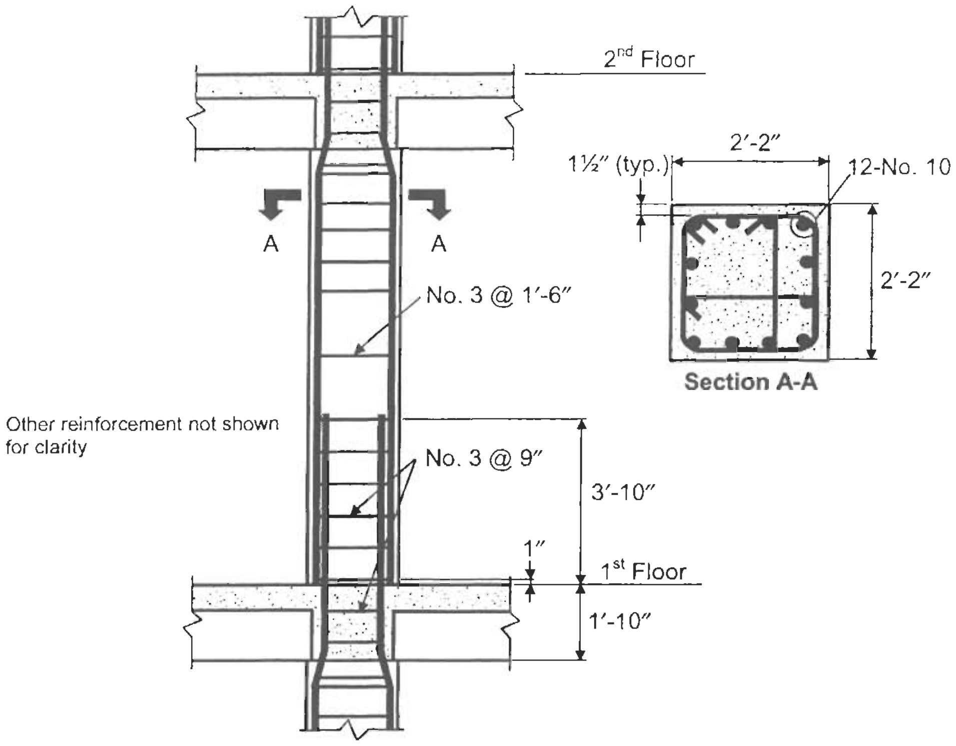


Figure 2-11 Reinforcement Details for Column C4 Supporting the 2nd Floor Level (SDC C)

Design for shear.

The shear strength of the concrete is determined in accordance with ACI 11.10.5 for walls subjected to axial compression:

$$V_c = 2\sqrt{f_c'}hd$$
$$= 2\sqrt{4{,}000} \times 12 \times 232.0/1{,}000 = 352.2 \text{ kips}$$

where d is permitted to be taken equal to $0.8\ell_w = 0.8 \times 290 = 232.0$ in. (ACI 11.10.4). The maximum factored shear force is 490 kips from wind load case 1 in the third load combination (see Table 2-20). Since $\phi V_c = 0.85 \times 352.2 = 299.4 \text{ kips} < V_u = 490 \text{ kips}$, horizontal shear reinforcement shall be provided in accordance with ACI 11.10.9. With 2 layers of No. 4 bars in the web, the required bar spacing is determined by Eq. 11-33:

$$s_2 = \frac{A_v f_y d}{V_s} = \frac{(2 \times 0.20) \times 60 \times 232.0}{(490/0.85) - 352.2} = 24.8 \text{ in.}$$

Note that ACI 14.3.4 requires two layers of reinforcement for walls more than 10 in. thick.

Table 2-20 Summary of Design Axial Forces, Bending Moments, and Shear Forces at Base of Shear Wall on Line 7 (SDC C)

Load Case	Full and Partial Wind Load Cases*	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)		2,625	0	0
Live (L)		279	0	0
Wind (W)	1	0	± 18,084	± 377
	2	0	± 17,402	± 363
	3	0	± 13,564	± 283
	4	0	± 13,180	± 275
Seismic (Q_E)		0	± 25,041	± 441
Load Combinations				
$1.4D + 1.7L$		4,149	0	0
$0.75(1.4D + 1.7L + 1.7W)$	1	3,112	23,057	481
	2	3,112	22,188	463
	3	3,112	17,294	361
	4	3,112	16,805	351
$0.9D + 1.3W$	1	2,363	-23,509	-490
	2	2,363	-22,623	-472
	3	2,363	-17,633	-368
	4	2,363	-17,134	-358
$1.28D + 0.5L + Q_E$		3,500	25,041	441
$0.82D + Q_E$		2,153	-25,041	-441

*Cases defined in ASCE 7 Figure 6-9

According to ACI 11.10.9.2, ratio of horizontal shear reinforcement shall not be less than 0.0025, and according to ACI 11.10.9.3, spacing of horizontal reinforcement shall not exceed $\ell_w/5 = 290/5 = 58$ in., $3h = 3 \times 12 = 36$ in., or 18 in. (governs). For 2-No. 4 horizontal bars spaced at 18 in., the ratio ρ_h of horizontal shear reinforcement area to gross concrete area of vertical section is

$$\rho_h = \frac{2 \times 0.20}{12 \times 18} = 0.0019 < 0.0025 \quad \text{N.G.}$$

Therefore, use 2-No. 4 horizontal bars @ 13 in. ($\rho_h = 0.0026$).

Shear strength V_n at any horizontal section must be less than or equal to $10\sqrt{f_c'}hd =$ 1,761 kips (ACI 11.10.3). In this case,

$$V_n = V_c + V_s = 352.2 + \frac{(2\times 0.20)\times 60\times 232.0}{13} = 780.5 \text{ kips} < 1{,}761 \text{ kips} \quad \text{O.K.}$$

The ratio of vertical shear reinforcement area to gross concrete area of horizontal section shall not be less than 0.0025 nor the value obtained from Eq. 11-34 (ACI 11.10.9.4):

$$\rho_n = 0.0025 + 0.5\left(2.5 - \frac{h_w}{\ell_w}\right)(\rho_h - 0.0025)$$

$$= 0.0025 + 0.5\left(2.5 - \frac{148}{24.167}\right)(0.0026 - 0.0025) = 0.0023$$

Thus, $\rho_n = 0.0025$.

According to ACI 11.10.9.5, spacing of vertical shear reinforcement shall not exceed $\ell_w/3$ = 290/3 = 96.7 in., $3h$ = 3 × 12 = 36 in., or 18 in. (governs). For 2-No. 4 vertical bars spaced at 13 in.,

$$\rho_n = \frac{2\times 0.20}{12\times 13} = 0.0026 > 0.0025 \quad \text{O.K.}$$

Use 2-No. 4 vertical bars @ 13 in.

The provided vertical and horizontal reinforcement satisfy the requirements of ACI 14.3.2 and 14.3.3 for minimum ratio of vertical and horizontal reinforcement to gross concrete area, respectively, and ACI 14.3.5 for maximum bar spacing.

Design for axial force and bending.

ACI 14.4 requires that walls subjected to axial load or combined flexure and axial load shall be designed as compression members in accordance with ACI 10.2, 10.3, 10.10 through 10.14, 10.17, 14.2, and 14.3 unless the empirical design method of ACI 14.5 or the alternative design method of ACI 14.8 can be used. Clearly, both of these methods cannot be applied in this case, and the wall is designed in accordance with ACI 14.4.

Figure 2-12 contains the interaction diagram of the wall. The wall is reinforced with 12-No. 10 bars in the 26 × 26 in. columns at both ends of the wall and 2-No. 4 vertical bars @ 13 in. in the web. As seen from the figure, the wall is adequate for the load combinations in Table 2-20.

Splice length of reinforcement.

Class B lap splices are utilized for the longitudinal reinforcement in the columns and the vertical bars in the web. No splices are required for the No. 4 horizontal bars in the web, since full length bars weigh approximately 0.668 × 24.17 = 16.2 lbs. and are easily installed.

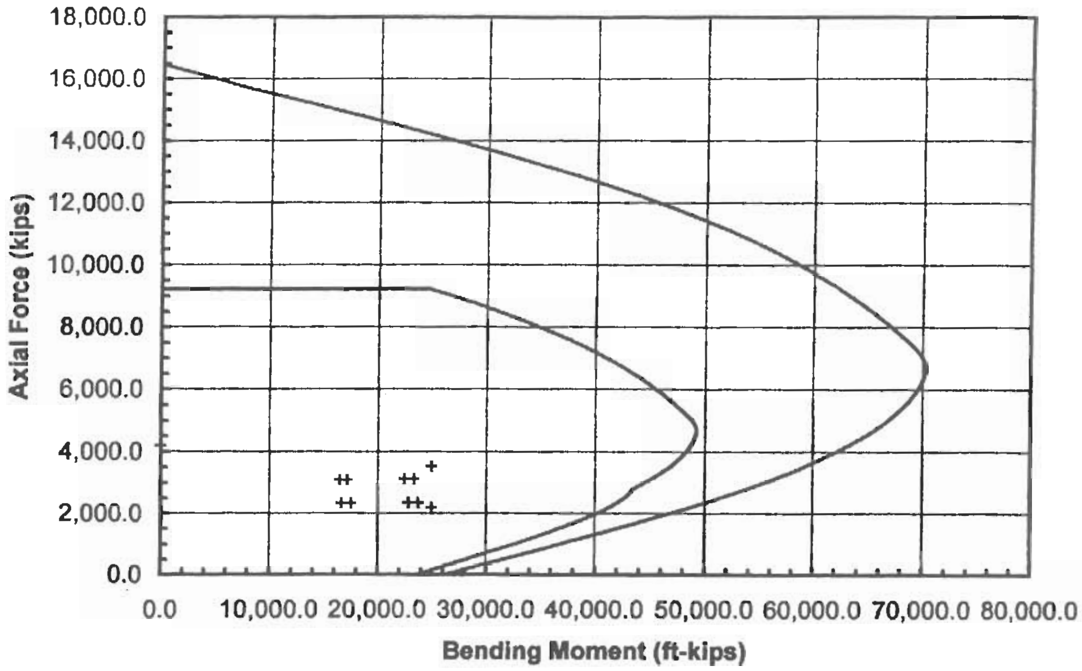


Figure 2-12 Design and Nominal Strength Interaction Diagrams for the Shear Wall Along Line 7 (SDC C)

For the No. 10 vertical bars in the columns, ℓ_d = 45.2 in., which was computed for column C4 in Section 2.3.4.3 of this publication. Therefore, according to ACI 12.15.1:

Class B splice length = $1.3\ell_d = 1.3 \times 45.2 = 58.8$ in.

Use a 5 ft-0 in. splice length for the No. 10 bars.

For the No. 4 vertical bars in the web, ℓ_d is determined from ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 0.8 for No. 6 and smaller bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 0.75 + 0.50 + \dfrac{0.50}{2} = 1.5 \text{ in.} \quad \text{(governs)} \\ \dfrac{1}{2} \times 13 = 6.5 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index = 0

$$\frac{c + K_{tr}}{d_b} = \frac{1.5 + 0}{0.50} = 3.0 > 2.5, \text{ use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{0.8 \times 1.0 \times 1.0 \times 1.0}{2.5} = 22.8$$

$$\ell_d = 22.8 \times 0.50 = 11.4 \text{ in.} < 12 \text{ in., use } 12 \text{ in.}$$

Class B splice length = $1.3\ell_d$ = 1.3 ft

Use a 1 ft-4 in. splice length for the No. 4 bars.

The No. 4 horizontal bars in the web must be developed in the columns. As shown above, the development length ℓ_d is equal to 1 ft. This length can be accommodated within the 26 in. = 2.17 ft columns, so that hooks are not needed at the ends of the bars.

Reinforcement details for the shear wall along line 7 are shown in Figure 2-13.

2.4 DESIGN FOR SDC D

To illustrate the design requirements for Seismic Design Category (SDC) D, the office building in Figure 2-1 is assumed to be located in San Francisco. Typical beams, columns, and walls are designed and detailed for combined effects of gravity, wind, and seismic forces.

2.4.1 Design Data

- Building Location: San Francisco, CA (zip code 94105)

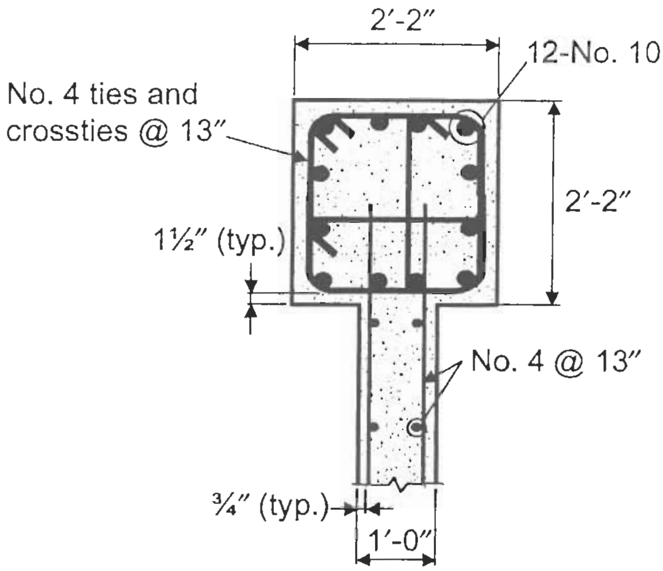


Figure 2-13 Reinforcement Details for Shear Wall Along Line 7 (SDC C)

- Material Properties

Concrete: $f_c' = 4{,}000$ psi, $w_c = 150$ pcf
Reinforcement: $f_y = 60{,}000$ psi

- Service Loads

Live loads: roof = 20 psf
floor = 50 psf

Superimposed dead loads: roof = 10 psf + 200 kips for penthouse
floor = 30 psf (20 psf permanent partitions + 10 psf ceiling, etc.)

- Seismic Design Data

For zip code 94105: $S_S = 1.50g, S_1 = 0.61g$ [2.1]
Site Class D (stiff soil profile; IBC Table 1615.1.1)
For Category (Seismic Use Group) I occupancy, $I_E = 1.0$ (IBC Table 1604.5)

- Wind Design Data

Basic wind speed = 85 mph (IBC Figure 1609)
Exposure B (IBC 1609.4)

For Category I occupancy, $I_W = 1.0$ (IBC Table 1604.5)

- Member Dimensions

Slab: 8 in.
Beams: 28 × 26 in.
Interior columns: 30 × 30 in.
Edge columns: 26 × 26 in.
Wall thickness: 16 in.
Boundary elements: 36 × 36 in.

2.4.2 Seismic Load Analysis

2.4.2.1 Seismic Design Category (SDC)

As was discussed in Section 2.3.2.1 of this publication, the appropriate analysis procedure to use depends on the Seismic Design Category (SDC). For SDC D, E, or F, Table 1616.6.3 contains the minimum allowable analysis procedure for seismic design (IBC 1616.6.3). The SDC for this particular example is determined below.

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1-second period S_{M1} are determined from IBC Eqs. 16-16 and 16-17, respectively:

$$S_{MS} = F_a S_s = 1.0 \times 1.50 = 1.50g$$

$$S_{M1} = F_v S_1 = 1.5 \times 0.61 = 0.92g$$

where F_a and F_v are the values of site coefficients as a function of site class and of mapped spectral response acceleration at short periods S_S and at 1-second period S_1, respectively. The values of these coefficients for Site Class D in the equations above are contained in IBC Table 1615.1.2(1) for F_a and Table 1615.1.2(2) for F_v.

Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from Eqs. 16-18 and 16-19:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 1.50 = 1.00g$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.92 = 0.61g$$

Tables 1616.3(1) and 1616.3(2) are utilized to determine the SDC. From Table 1616.3(1) for Seismic Use Group I and S_{DS} = 1.00g, the SDC is D. Similarly, from Table 1616.3(2), the SDC is D for S_{D1} = 0.61g. Thus, the SDC is D for this building.

2.4.2.2 Seismic Forces

Since the building does not have plan irregularity Type 1a, 1b, or 4 of Table 1616.5.1 or vertical irregularity Type 1a, 1b, 4, or 5 of Table 1616.5.2, it can be considered regular (IBC 1616.6.3; note: it is shown in Section 2.4.2.5 below that the first story is not a soft story). For this regular building that is less than 240 ft in height, Table 1616.6.3 allows the equivalent lateral force procedure in IBC 1617.4 to be used to compute the seismic base shear V (see Eq. 16-34):

$$V = C_s W$$

where C_s is the seismic response coefficient determined in accordance with IBC 1617.4.1.1 and W is the effective weight of the structure defined in IBC 1617.4. For the member sizes and superimposed dead loads given above, W = 30,986 kips (see Table 2-21 below).

Seismic Forces in N-S Direction.

In the N-S direction, a dual system is utilized. In order to satisfy the provisions of IBC 1910.5.1 for structures assigned to SDC D, the dual system must have special reinforced concrete moment frames and special reinforced concrete shear walls. For this system, the response modification coefficient R = 8 and the deflection amplification factor C_d = 6.5 (IBC Table 1617.6).

Approximate period (T_a). The fundamental period T is determined in accordance with IBC 1617.4.2. In lieu of a more exact analysis, an approximate fundamental period T_a is computed from Eq. 16-39 for the dual system:

Building height h_n = 148 ft

Building period coefficient $C_T = 0.02$

Period $T_a = C_T (h_n)^{3/4} = 0.020 \times (148)^{3/4} = 0.85$ sec

For comparison purposes, the period was also determined using SAP2000 [2.4], which gave T = 1.21 sec. In this example, no further refinement of the period is made.

Seismic base shear (V). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.61}{\left(\frac{8}{1.0}\right)\times 0.85} = 0.090$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{1.00}{\left(\frac{8}{1.0}\right)} = 0.125$$

Also, C_s must not be less than the following value from Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 1.0 \times 1.0 = 0.044$$

For buildings assigned to SDC E or F and for those buildings for which $S_1 \geq 0.6g$, C_s shall not be taken less than that computed from Eq. 16-38. Since $S_1 = 0.61g$, Eq. 16-38 is applicable, even though the SDC is D:

$$C_s = \frac{0.5 S_1}{R / I_E} = \frac{0.5 \times 0.61}{8 / 1.0} = 0.038$$

In this case, the lower limit is 0.044 from Eq. 16-37.

The value of C_s from Eq. 16-36 governs; therefore, the base shear V in the N-S direction is:

$$V = C_s W = 0.090 \times 30{,}986 = 2{,}789 \text{ kips}$$

Vertical distribution of seismic forces. The total base shear is distributed over the height of the building in conformance with Eqs. 16-41 and 16-42:

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

where F_x is the lateral force induced at level x, w_x and w_i are the portions of W assigned to levels x or i, and k is the distribution exponent defined in IBC 1617.4.3. For

$T = 0.85$ sec, $k = 1.17$ from linear interpolation. The lateral forces F_x and the story shears V_x are contained in Table 2-21.

Table 2-21 Seismic Forces and Story Shears in N-S Direction (SDC D)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
12	2,350	148	813,344	415	415
11	2,597	136	814,165	415	830
10	2,597	124	730,761	373	1,203
9	2,597	112	648,720	331	1,534
8	2,597	100	568,162	290	1,824
7	2,597	88	489,234	250	2,074
6	2,597	76	412,120	210	2,284
5	2,597	64	337,056	172	2,456
4	2,597	52	264,360	135	2,591
3	2,597	40	194,483	98	2,689
2	2,597	28	128,129	65	2,754
1	2,666	16	68,341	35	2,789
Σ	30,986		5,468,875	2,789	

Seismic Forces in E-W Direction.

In the E-W direction, a moment-resisting frame system is utilized. In order to satisfy the provisions of IBC 1910.5.1 for structures assigned to SDC D, this must be a special reinforced concrete moment frame. For this system, the response modification coefficient $R = 8$ and the deflection amplification factor $C_d = 5.5$ (IBC Table 1617.6).

Approximate period (T_a). Similar to the N-S direction, the fundamental period of the building is determined in accordance with Eq. 16-39 in IBC 1617.4.2:

Building period coefficient $C_T = 0.03$

Period $T_a = C_T (h_n)^{3/4} = 0.030 \times (148)^{3/4} = 1.27$ sec

The period obtained from SAP2000 is $T = 1.93$ sec.

Seismic base shear (V). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.61}{\left(\frac{8}{1.0}\right)\times 1.27} = 0.060$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{1.00}{\left(\frac{8}{1.0}\right)} = 0.125$$

Also, C_s must not be less than the larger value from Eq. 16-37 and 16-38:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 1.0 \times 1.0 = 0.044 \quad \text{(governs)}$$

$$C_s = \frac{0.5 S_1}{R / I_E} = \frac{0.5 \times 0.61}{8/1.0} = 0.038$$

Thus, Eq. 16-36 governs, and the base shear V in the E-W direction is:

$$V = C_s W = 0.060 \times 30{,}986 = 1{,}859 \text{ kips}$$

Vertical distribution of seismic forces. The lateral forces F_x and the story shears V_x are contained in Table 2-22 for seismic forces in the E-W direction. For T = 1.27 sec, k = 1.39 from linear interpolation.

2.4.2.3 Method of Analysis

A three-dimensional analysis of the building was performed in the N-S and E-W directions for the seismic forces contained in Tables 2-21 and 2-22 using SAP2000 [2.4]. In the model, rigid diaphragms were assigned at each floor level, and rigid-end offsets were defined at the ends of the horizontal members so that results were automatically obtained at the faces of the supports. The stiffness properties of the members were input assuming cracked sections. In lieu of a more accurate analysis, the following cracked section properties were used:

- Beams: $I_{eff} = 0.5 I_g$
- Columns: $I_{eff} = 0.7 I_g$
- Shear walls: $I_{eff} = 0.35 I_g$

where I_g is the gross moment of inertia of the section. The shear walls have been assigned an effective moment of inertia equal to one-half of the value assigned for SDC A and C, since it is anticipated that significantly more cracking will occur in the walls for SDC D. The effective moments of inertia for the beams and columns have been taken the same as for SDC A and C. P-delta effects were also considered in the analysis.

Table 2-22 Seismic Forces and Story Shears in E-W Direction (SDC D)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
12	2,350	148	2,441,921	302	302
11	2,597	136	2,399,336	297	599
10	2,597	124	2,110,222	261	860
9	2,597	112	1,831,829	226	1,086
8	2,597	100	1,564,847	194	1,280
7	2,597	88	1,310,095	162	1,442
6	2,597	76	1,068,570	132	1,574
5	2,597	64	841,516	104	1,678
4	2,597	52	630,546	78	1,756
3	2,597	40	437,860	54	1,810
2	2,597	28	266,699	33	1,843
1	2,666	16	125,773	16	1,859
Σ	30,986		15,029,214	1,859	

In accordance with IBC 1617.4.4.4, the center of mass was displaced each way from its actual location a distance equal to 5 percent of the building dimension perpendicular to the applied forces to account for accidental torsion in seismic design. Torsional effects need not be amplified, since the building does not possess Type 1a or 1b plan torsional irregularity as defined in Table 1616.5.1 (IBC 1617.4.4.5).

In a dual system, an additional safeguard is provided by requiring that moment-resisting frames be capable of resisting at least 25% of the design forces without the benefit of shear walls (IBC 1617.6.1). Thus, the building was also analyzed in the N-S direction using 25% of the design forces in Table 2-21 without the shear walls present, including torsional effects.

For all structures assigned to SDC D and higher, IBC 1620.3.5 requires that orthogonal effects of the seismic forces be considered for design and detailing of the components of the seismic-force-resisting system. The orthogonal combination procedure in IBC 1620.2.2 is permitted to be used to satisfy the requirements of IBC 1620.3.5. In the 2002 supplement to the 2000 IBC [2.7], the orthogonal combination procedure is required only for columns or walls that form part of two or more intersecting seismic-force-resisting systems and are subjected to axial load due to seismic forces greater than or equal to 20% of the axial load design strength. Both situations are examined below.

2.4.2.4 Story Drift and P-delta Effects

Story drift determination. Table 2-23 contains the displacements δ_{xe} obtained from the 3-D static, elastic analyses using the design seismic forces in the N-S and E-W directions, including accidental torsional effects. The table also contains the design earthquake displacement δ_x computed by IBC Eq. 16-46:

$$\delta_x = \frac{C_d \delta_{xe}}{I_E}$$

where C_d is the deflection amplification factor in Table 1617.6, which depends on the seismic-force-resisting system. As noted above, C_d is equal to 6.5 for the dual system with special reinforced concrete moment frames and special reinforced concrete shear walls in the N-S direction and is 5.5 for the special reinforced concrete moment frames in the E-W direction.

Table 2-23 Lateral Displacements and Interstory Drifts due to Seismic Forces in N-S and E-W Directions (SDC D)

Story	N-S Direction			E-W Direction		
	δ_{xe} (in.)	δ_x (in.)	Δ (in.)	δ_{xe} (in.)	δ_x (in.)	Δ (in.)
12	2.85	18.53	1.30	3.87	21.29	0.50
11	2.65	17.23	1.56	3.78	20.79	0.82
10	2.41	15.67	1.56	3.63	19.97	1.16
9	2.17	14.11	1.69	3.42	18.81	1.43
8	1.91	12.42	1.76	3.16	17.38	1.70
7	1.64	10.66	1.82	2.85	15.68	1.93
6	1.36	8.84	1.82	2.50	13.75	2.03
5	1.08	7.02	1.75	2.13	11.72	2.20
4	0.81	5.27	1.63	1.73	9.52	2.31
3	0.56	3.64	1.43	1.31	7.21	2.37
2	0.34	2.21	1.17	0.88	4.84	2.36
1	0.16	1.04	1.04	0.45	2.48	2.48

The interstory drifts Δ computed from the δ_x are also contained in the table. For this structure that does not have plan irregularity Type 1a or 1b of Table 1616.5.1, the drift at story level x is determined by subtracting the design earthquake displacement at the center of mass at the bottom of the story from the design earthquake displacement at the center of mass at the top of the story (IBC 1617.4.6.1):

$$\Delta = \delta_x - \delta_{x-1}$$

The design story drifts Δ must not exceed the allowable story drift Δ_a from Table 1617.3 (IBC 1617.3). For Seismic Use Group I, $\Delta_a = 0.020 h_{sx}$ where h_{sx} is the story height below level x. Thus, for the 12-ft story heights, $\Delta_a = 0.020 \times 12 \times 12 =$ 2.88 in., and for the 16-ft story height at the first level, $\Delta_a = 3.84$ in. It is evident from

Table 2-23 that for all stories, the lateral drifts obtained from the prescribed lateral forces in both directions are less than the limiting values.

P-delta effects. As noted above, P-delta effects were automatically considered in the analysis using SAP2000. However, for illustration purposes, the following procedure can be used to determine whether P-delta effects need to be considered or not in accordance with IBC 1617.4.6.2.

P-delta effects need not be considered when the stability coefficient θ determined by Eq. 16-47 is less than or equal to 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d}$$

where P_x = total unfactored vertical design load at and above level x
Δ = design story drift occurring simultaneously with V_x
V_x = seismic shear force acting between level x and $x - 1$
h_{sx} = story height below level x
C_d = deflection amplification factor

The stability coefficient θ must not exceed θ_{max} determined from Eq. 16-48:

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25$$

where β is the ratio of shear demand to shear capacity between level x and $x - 1$, which may be taken equal to 1.0 when it is not calculated.

Table 2-24 contains the calculations for the N-S and E-W directions. It is clear that in both directions, P-delta effects need not be considered at any of the floor levels. Note that θ_{max} is equal to 0.077 and 0.091 in the N-S and E-W directions, respectively, using β = 1.0.

2.4.2.5 Soft Story

The following procedure can be used to determine whether the first story is a soft story or not. It will become evident in this discussion why this procedure is presented after the section on story drifts.

According to IBC Table 1616.5.2 for vertical structural irregularities, a soft story is one in which the lateral stiffness is less 70% of that in the story above or less than 80% of the average stiffness of the 3 stories above (irregularity type 1a). As noted in the table, this irregularity type is applicable to buildings assigned to SDC D, E, and F.

Table 2-24 P-delta Effects (SDC D)

Level	h_{sx} (ft)	P_x (kips)	N-S Direction			E-W Direction		
			V_x (kips)	Δ (in.)	θ	V_x (kips)	Δ (in.)	θ
12	12	2,601	415	1.30	0.009	302	0.50	0.005
11	12	5,449	830	1.56	0.011	599	0.82	0.010
10	12	8,297	1,203	1.56	0.012	860	1.16	0.014
9	12	11,145	1,534	1.69	0.013	1,086	1.43	0.019
8	12	13,993	1,824	1.76	0.014	1,280	1.70	0.024
7	12	16,841	2,074	1.82	0.016	1,442	1.93	0.028
6	12	19,690	2,284	1.82	0.017	1,574	2.03	0.032
5	12	22,538	2,456	1.75	0.017	1,678	2.20	0.037
4	12	25,386	2,591	1.63	0.017	1,756	2.31	0.042
3	12	28,234	2,689	1.43	0.016	1,810	2.37	0.047
2	12	31,082	2,754	1.17	0.014	1,843	2.36	0.050
1	16	33,999	2,789	1.04	0.010	1,859	2.48	0.043

In general, determining the lateral stiffness of a story in a building can be complex. In lieu of computing story stiffness, floor level displacements can be used to ascertain whether a soft story exists or not. A displacement-based method is presented in Example 4 in Reference 2.5. According to this method, a soft story exists in the first floor level when one of the following conditions is satisfied:

$$0.70\left(\frac{\delta_{e1}}{h_1}\right) > \frac{\delta_{e2}-\delta_{e1}}{h_2}$$

$$0.80\left(\frac{\delta_{e1}}{h_1}\right) > \frac{1}{3}\left(\frac{\delta_{e2}-\delta_{e1}}{h_2}+\frac{\delta_{e3}-\delta_{e2}}{h_3}+\frac{\delta_{e4}-\delta_{e3}}{h_4}\right)$$

where δ_{ei} are the elastic displacements at level i due to code-prescribed lateral forces and h_i are the story heights below level i.

Using the data in Table 2-23, check if a soft story exists in the first story based on displacements in the E-W direction:

$$0.70\left(\frac{\delta_{e1}}{h_1}\right) = 0.70\left(\frac{0.45}{16\times 12}\right) = 0.00164 < \frac{\delta_{e2}-\delta_{e1}}{h_2} = \frac{0.88-0.45}{12\times 12} = 0.00299 \quad \text{OK}$$

$$0.80\left(\frac{\delta_{e1}}{h_1}\right) = 0.00188 < \frac{1}{3}\left(\frac{0.88-0.45}{12\times 12}+\frac{1.31-0.88}{12\times 12}+\frac{1.73-1.31}{12\times 12}\right) = 0.00296 \quad \text{OK}$$

Similar calculations with displacements in the N-S direction also show that a soft story does not exist in the first story of this example building.

2.4.3 Wind Load Analysis

2.4.3.1 Wind Forces

According to IBC 1609.1.1, wind forces shall be determined in accordance with Section 6 of ASCE 7 [2.2]. Since the building has a mean roof height greater than 30 ft, the simplified procedure (Method 1) given in ASCE 6.4 cannot be used to determine the wind forces. Similarly, the simplified procedure of IBC 1609.6 must not be used, since the building is taller than 60 ft. As was discussed in Section 2.2.3 of this publication, the analytical procedure (Method 2) of ASCE 6.5 may be used to determine the wind forces.

Details on how to compute the wind forces in both the N-S and E-W directions are given in Section 2.2.3 of this publication. In this example, the wind velocity is 85 mph. A summary of the design wind forces in both directions at all floor levels is contained in Table 2-25. Once again it is important to note that the wind directionality factor K_d has been taken equal to 1.0 (see Exception 1 in IBC 1605.2.1).

2.4.3.2 Method of Analysis

Similar to the seismic analysis, a three-dimensional analysis of the building was performed in both directions for the wind forces contained in Tables 2-25 using SAP2000. The modeling assumptions utilized for the seismic analysis were also used for the wind analysis.

Also considered in the three-dimensional analysis were the full and partial wind load cases of Figure 6-9 (Cases 1 through 4) given in ASCE 6.5.12.3.

2.4.4 Design for Combined Load Effects

2.4.4.1 Load Combinations

Basic load combinations for strength design are given in IBC 1605.2.1. As noted above, the first exception in this section requires that the non-seismic load combinations of ACI 9.2 be used for concrete structures. Thus, the following load combinations are utilized in the design of the structural members:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)
4. $1.2D + f_1 L + 1.0E$ (Formula 16-5)
5. $0.9D + 1.0E$ (Formula 16-6)

Table 2-25 Design Wind Forces in N-S and E-W Directions (V = 85 mph)

Level	Height above ground level, *z* (ft)	Total Design Wind Force N-S (kips)	Total Design Wind Force E-W (kips)
12	148	25.0	8.6
11	136	49.2	16.9
10	124	48.4	16.6
9	112	47.6	16.2
8	100	46.7	15.9
7	88	45.7	15.4
6	76	44.6	15.0
5	64	43.4	14.5
4	52	42.0	13.9
3	40	40.4	13.2
2	28	38.3	12.3
1	16	41.4	13.0
	Σ	512.7	171.5

where D, L, W, and E are the effects due to dead, live, wind, and seismic loads, respectively, and f_1 is equal to either 1.0 for places of public assembly, for live loads in excess of 100 psf, and for parking garage live load, or is equal to 0.5 for other live loads.

The seismic load effect E for use in Formula 16-5, which is the combined effect of horizontal and vertical earthquake-induced forces, is computed from Eq. 16-28 where the effects of gravity and seismic ground motion are additive:

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of horizontal seismic forces

ρ = redundancy coefficient determined in accordance with IBC 1617.2.2 for SDC D, E, or F

Similarly, E for use in Formula 16-6 is computed from Eq. 16-29 where the effects of gravity and seismic ground motion counteract:

$$E = \rho Q_E - 0.2 S_{DS} D$$

According to IBC 1617.2.2, the redundancy coefficient ρ, which shall not be less than 1.0 and need not exceed 1.5, is the largest of the values of ρ_i calculated at each story i from Equation 16-32:

$$\rho_i = 2 - \frac{20}{r_{\max_i}\sqrt{A_i}}$$

Determination of $r_{\max_i}$ depends on the type of lateral-force-resisting system. Considering the types of systems utilized in this example building:

- For moment frames:

  $r_{\max_i}$ = maximum sum of shears in any 2 adjacent columns/total story shear

  In the column shear summation, use 70% of shear in columns common to 2 adjacent bays, with moment-resisting connections on opposite sides.

- For shear walls:

  $r_{\max_i} = (\text{maximum wall shear} \times 10/\ell_w)/\text{total story shear}$

  ℓ_w = length of the wall in feet

- For dual systems:

$$r_{\max_i} = \text{max. of} \begin{cases} r_{\max_i} \text{ calculated for portion of story shear carried by columns} \\ r_{\max_i} \text{ calculated for portion of story shear carried by walls} \end{cases}$$

In buildings combining shear walls and frames (i.e., dual system), shear walls in the lowest portions of the building resist most of the story shear; frames carry more story shear in the upper portions of the building. Consequently, in the first story where the story shear is the largest, $r_{\max_i}$ is computed in accordance with the equation given above for shear walls. Clearly, the lowest story level produces the highest value of ρ_i for dual systems; by definition, this is ρ for the entire building in the in-plane direction of the shear walls. IBC 1617.2.2 allows this value of ρ to be multiplied by 0.8 for dual systems.

In all cases, A_i is the floor area in square feet of the diaphragm level immediately above story i.

A comprehensive discussion on the determination of redundancy coefficients for all types of seismic-force-resisting systems can be found in Reference 2.6.

For the dual system in the N-S direction, the shear force in the most heavily load shear wall in the first story is 1,360 kips and the story shear is 2,789 kips. Therefore,

$$r_{\max_1} = \frac{1{,}360 \times (10/25)}{2{,}789} = 0.195$$

$$\rho_1 = \rho = 2 - \frac{20}{0.195\sqrt{184.167 \times 68.167}} = 1.09$$

For dual systems, ρ need not exceed $0.8 \times 1.09 = 0.87 < 1.0$. Use $\rho = 1.0$.

Computations for the redundancy coefficient in the E-W direction for the moment-resisting frame along column line B (most heavily loaded frame when center of mass is displaced north of actual location) are summarized in Table 2-26. Since the shear force distribution in the columns is symmetric, results are presented for columns on lines 1 through 4 only. Subscripts on column shear forces refer to column lines. To illustrate the computations, $r_{\max_i}$ is calculated for the first story as follows:

$$r_{\max_1} = \frac{\text{maximum sum of shears in 2 adjacent columns}}{\text{total story shear}} = \frac{124.5}{1{,}859} = 0.067$$

From Table 2-26, maximum $r_{\max_i} = 0.069$; thus, ρ is:

$$\rho = 2 - \frac{20}{0.069\sqrt{184.167 \times 68.167}} < 0$$

Therefore, $\rho = 1.0$. For SDC D, IBC 1617.2.2 requires that ρ be less than or equal to 1.25 for special moment frames, which is satisfied in this case.

Table 2-26 Redundancy Coefficient Calculations in E-W Direction (SDC D)*

Story	Story Shear	Column Shear Forces				$V_1 + 0.7V_2$	$0.7(V_2 + V_3)$	$0.7(V_3 + V_4)$	$r_{\max_i}$
		V_1	V_2	V_3	V_4				
12	302	2.7	4.7	15.2	14.6	6.0	13.9	20.9	0.069
11	599	9.5	22.1	24.4	24.0	25.0	32.6	33.9	0.057
10	860	14.3	30.7	35.0	34.2	35.8	46.0	48.4	0.056
9	1,086	18.7	39.4	43.4	42.5	46.3	58.0	60.1	0.055
8	1,280	22.4	46.5	50.8	49.7	55.0	68.1	70.4	0.055
7	1,442	25.6	52.5	56.9	55.7	62.4	76.6	78.8	0.055
6	1,574	28.3	57.3	61.9	60.5	68.4	83.4	85.7	0.054
5	1,678	30.4	61.5	65.6	64.2	73.5	89.0	90.9	0.054
4	1,756	32.2	63.4	68.6	67.1	76.6	92.4	95.0	0.054
3	1,810	33.3	69.8	69.5	68.2	82.2	97.5	96.4	0.053
2	1,843	34.5	56.6	72.9	70.8	74.1	90.7	100.6	0.055
1	1,859	36.5	110.6	67.3	66.6	113.9	124.5	93.7	0.067

*Shear forces are in kips.

Once ρ has been computed in both orthogonal directions, load combinations 4 and 5 can be rewritten as follows. Substituting $S_{DS} = 1.0g$ and $\rho = 1.0$ into the equations for E, and then substituting E along with $f_1 = 0.5$ into load combinations 4 and 5 above results in the following:

4a. $1.2D + 0.5L + 1.0Q_E + (0.2 \times 1.0)D = 1.4D + 0.5L + Q_E$

4b. $1.2D + 0.5L + 1.0Q_E - (0.2 \times 1.0)D = D + 0.5L + Q_E$

5a. $0.9D + 1.0Q_E + (0.2 \times 1.0)D = 1.1D + Q_E$

5b. $0.9D + 1.0Q_E - (0.2 \times 1.0)D = 0.7D + Q_E$

If it had turned out that ρ for the dual system was different than ρ for the moment-resisting system, then two different sets of Eqs. 4a through 5b would be determined, one for each orthogonal direction.

Values of Q_E are obtained from the structural analysis for both sidesway to the left and to the right based on the code-prescribed horizontal seismic forces acting on the building. Also, the special seismic load combinations in IBC 1605.4 are not applicable in this example.

2.4.4.2 Design of Beam C4-C5

Flexural design.

Comparing the seismic forces in Table 2-22 to the wind forces in Table 2-25, it is clear that seismic effects will govern the design in the E-W direction in this example. For this beam, orthogonal effects due to seismic forces are negligible, i.e., seismic forces in the N-S direction have a negligible effect on the design forces of this member. According to the 2002 supplement to the IBC [2.7], orthogonal effects need not be considered for beams. Thus, Table 2-27 contains a summary of the governing design bending moments and shear forces at the second floor level due to gravity and seismic forces in the E-W direction only.

Requirements for special moment frames are given in ACI 21.2 through 21.5. The factored axial load on the member, which is negligible, is less than $A_g f_c'/10$; thus, the provisions of ACI 21.3 for flexural members of special moment frames must be satisfied. All other applicable provisions in Chapters 1 through 18 are to be satisfied as well.

Check limitations on section dimensions per ACI 21.3.1:

- Factored axial compressive force on member is negligible. O.K.
- $\dfrac{\ell_n}{d} = \dfrac{(26 \times 12) - 30}{23.5} = 12 > 4$ O.K.

- $\frac{\text{width}}{\text{depth}} = \frac{28}{26} = 1.1 > 0.3$ O.K.

- width = 28 in. > 10 in. O.K.

  < width of supporting column + (1.5 × depth of beam)

  < 30 + (1.5 × 26) = 69 in. O.K.

Table 2-27 Summary of Design Bending Moments and Shear Forces for Beam C4-C5 at the 2nd Floor Level (SDC D)

Load Case	Location	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	Support	-111	28
	Midspan	76	
Live (L)	Support	-16	8
	Midspan	11	
Seismic (Q_E)	Support	± 366	± 31
Load Combinations			
$1.4D + 1.7L$	Support	-183	53
	Midspan	125	
$1.4D + 0.5L + Q_E$	Support	-529	74
	Midspan	112	
$0.7D + Q_E$	Support	288	-11
	Midspan	53	

The required flexural reinforcement is contained in Table 2-28. The provided areas of steel are within the limits prescribed in ACI 21.3.2.1 for maximum and minimum reinforcement. The selected reinforcement satisfies ACI 7.6.1 and 3.3.2 (minimum spacing for concrete placement), ACI 7.7.1 (minimum cover for protection of reinforcement), and ACI 10.6 (maximum spacing for control of flexural cracking).

Table 2-28 Required Flexural Reinforcement for Beam C4-C5 at the 2nd Floor Level (SDC D)

Location	M_u (ft-kips)	A_s^* (in.^{2})	Reinforcement*	ϕM_n (ft-kips)
Support	-529	5.40	7-No. 8	541
	288	2.83	5-No. 7	305
Midspan	125	2.19	5-No. 7	305

$$^*A_{s,min} = \frac{3\sqrt{f'_c}\,b_w d}{f_y} = \frac{3\sqrt{4{,}000} \times 28 \times 23.5}{60{,}000} = 2.08 \text{ in.}^2$$

$$= \frac{200 b_w d}{f_y} = \frac{200 \times 28 \times 23.5}{60{,}000} = 2.19 \text{ in.}^2 \text{ (governs)}$$

ACI 21.3.2.1

$$A_{s,max} = \rho_{max} b_w d = 0.025 \times 28 \times 23.5 = 16.5 \text{ in.}^2$$

ACI 21.3.2.1

ACI 21.3.2.2 requires that the positive moment strength at the face of the joint be greater than or equal to 50% of the negative moment strength at that location. This is satisfied, since 305 ft-kips > 541/2 = 271 ft-kips. Also, the negative or positive moment strength at any section along the length of the member must be greater than or equal to 25% of the maximum moment strength provided at the face of either joint. In this case, 25% of the maximum moment strength is equal to 541/4 = 135 ft-kips. Providing 3-No. 8 bars (ϕM_n = 243 ft-kips) or 3-No. 7 bars (ϕM_n = 186 ft-kips) satisfies this provision. However, to satisfy the minimum reinforcement requirement of ACI 21.3.2.2 (i.e., minimum A_s = 2.19 in.^{2}), a minimum of 3-No. 8 bars (A_s = 2.37 in.^{2}) or 4-No. 7 bars (A_s = 2.40 in.^{2}) must be provided at any section. This also automatically satisfies the requirement that at least 2 bars be continuous at both the top and bottom of the section (ACI 21.3.2.1).

When reinforcing bars extend through a joint, the column dimension parallel to the beam reinforcement must be at least 20 times the diameter of the largest longitudinal bar for normal weight concrete (ACI 21.5.1.4). In this case, the minimum required column dimension = 20 × 1.0 = 20 in., which is less than the 30-in. column width that is provided.

Shear design.

Shear requirements for beams in special moment frames are contained in ACI 21.3.4. The method of determining design shear forces in beams in special moment frames takes into consideration the likelihood of yielding (i.e., plastic hinges forming) at regions near the supports. In general, the shear forces are determined assuming simultaneous hinging at the beam supports under lateral loading. To properly confine the concrete and to maintain lateral support of the longitudinal bars in regions where yielding is expected, the transverse reinforcement requirements of ACI 21.3.3 must also be satisfied.

According to ACI 21.3.4.1, shear forces are computed from statics assuming that moments of opposite sign corresponding to the probable moment strength M_{pr} act at the joint faces and that the member is loaded with tributary factored gravity load along its span. Sidesway to the right and to the left must be considered when calculating the maximum design shear forces.

The probable moment strength M_{pr} for a section is determined using the stress in the tensile reinforcement equal to $1.25 f_y$ and a strength reduction factor ϕ equal to 1.0 (ACI 21.0). The following equation can be used to compute M_{pr}:

$$M_{pr} = A_s(1.25 f_y)\left(d - \frac{a}{2}\right)$$

where $a = \dfrac{A_s(1.25 f_y)}{0.85 f'_c b}$

For example, for sidesway to the right, the joint on column line 5 is subjected to the negative moment M_{pr} that is determined as follows:

For 7-No. 8 top bars, $A_s = 7 \times 0.79 = 5.53$ in.^{2}

$$a = \frac{5.53 \times 1.25 \times 60}{0.85 \times 4 \times 28} = 4.36 \text{ in.}$$

$$M_{pr} = 5.53 \times 1.25 \times 60 \times \left(23.5 - \frac{4.36}{2}\right) = 8{,}843 \text{ in.-kips} = 737 \text{ ft-kips}$$

Similarly, for the joint on column line 4, the positive moment M_{pr} based on 5-No. 7 bars is equal to 419 ft-kips.

The largest shear force associated with seismic effects is obtained from the second of the three load combinations in Table 2-27. Figure 2-14 shows the beam and shear forces due to factored gravity loads plus probable moment strengths for sidesway to the right. Due to the symmetric distribution of longitudinal reinforcement in the beam, sidesway to the left gives the same maximum shear force. The equivalent factored uniform loads on the beam are determined as follows (see ACI Figure R13.6.8 for definition of tributary area on beam):

$$\text{Total trapezoidal area tributary to beam} = 2\left\{2\left[\frac{1}{2} \times 11 \times 11\right] + (4 \times 11)\right\} = 330 \text{ ft}^2$$

$$\text{Dead load} = \left(\frac{8}{12} \times 0.15 \times 330\right) + \left(\frac{18 \times 28}{144} \times 0.15 \times 23.5\right) + \left(\frac{30 \times 330}{1{,}000}\right) = 55.2 \text{ kips}$$

$$w_D = \frac{55.2}{23.5} = 2.4 \text{ kips/ft}$$

$$\text{Live load} = \frac{50 \times 330}{1{,}000} = 16.5 \text{ kips}$$

$$w_L = \frac{16.5}{23.5} = 0.7 \text{ kips/ft}$$

$$w_u = 1.4 w_D + 0.5 w_L = (1.4 \times 2.4) + (0.5 \times 0.7) = 3.7 \text{ kips/ft}$$

The maximum combined shear force of 92.7 kips shown in Figure 2-14 is larger than the maximum shear force obtained from the structural analysis, which is 74 kips (see Table 2-27).

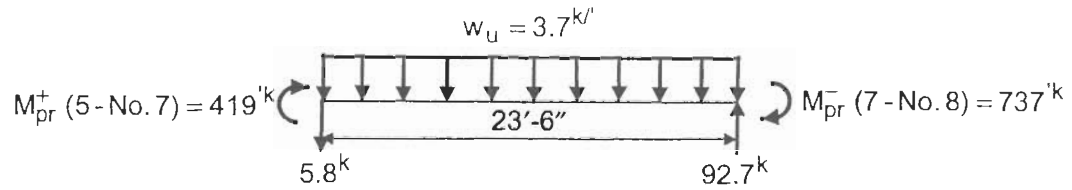


Figure 2-14 Design Shear Forces for Beam C4-C5 (SDC D)

In general, shear strength is provided by both concrete (V_c) and reinforcing steel (V_s). However, according to ACI 21.3.4.2, V_c is to be taken as zero when the earthquake-induced shear force calculated in accordance with ACI 21.3.4.1 is greater than or equal to 50% of the total shear force and the factored axial compressive force including earthquake effects is less than $A_g f'_c/20$. In this example, the beam carries negligible axial forces and the maximum earthquake-induced shear force = (737 + 419)/23.5 = 49.2 kips > 92.7/2 = 46.4 kips. Thus, V_c must be taken equal to zero.

The maximum shear force V_s is (ACI 11.1):

$$V_s = \frac{V_u}{\phi} - V_c = \frac{92.7}{0.85} - 0 = 109.1 \text{ kips}$$

where the strength reduction factor was taken as 0.85 in accordance with ACI 9.3.4.

Shear strength contributed by shear reinforcement shall not exceed $(V_s)_{max}$ (ACI 11.5.6.9):

$$(V_s)_{max} = 8\sqrt{f'_c} b_w d = 8\sqrt{4{,}000} \times 28 \times 23.5/1{,}000 = 332.9 \text{ kips} > 109.1 \text{ kips O.K.}$$

Also, V_s is less than $4\sqrt{f'_c} b_w d$ = 166.5 kips.

Required spacing s of No. 3 closed stirrups (hoops) for a factored shear force of 109.1 kips is determined from Eq. (11-15):

$$s = \frac{A_v f_y d}{V_s} = \frac{(4 \times 0.11) \times 60 \times 23.5}{109.1} = 5.7 \text{ in.}$$

Note that 4 legs are required for support of the longitudinal bars (ACI 21.3.3.3).

Maximum allowable hoop spacing within a distance of $2h = 2 \times 26 = 52$ in. (plastic hinge length) from the face of the support at each end of the member is the smaller of the following (ACI 21.3.3.2):

- $d/4 = 23.5/4 = 5.9$ in. (governs)
- 8(diameter of smallest longitudinal bar) = $8 \times 0.875 = 7.0$ in.
- 24(diameter of hoop bar) = $24 \times 0.375 = 9.0$ in.
- 12 in.

Use 11-No. 3 hoops at each end of the beam spaced at 5 in. on center with the first hoop located 2 in. from the face of the support (ACI 21.3.3.2).

Where hoops are no longer required, stirrups with seismic hooks at both ends may be used (ACI 21.3.3.4). At a distance of 52 in. from the face of the support:

$$V_u = 92.7 - [3.7 \times (52/12)] = 76.7 \text{ kips}$$

Also, the shear strength contributed by the concrete may be utilized outside of the potential plastic hinge zones:

$$V_c = 2\sqrt{4{,}000} \times 28 \times 23.5 / 1{,}000 = 83.2 \text{ kips}$$

Therefore, the required stirrup spacing for No. 3 stirrups is:

$$s = \frac{A_v f_y d}{V_s} = \frac{(2 \times 0.11) \times 60 \times 23.5}{(76.7/0.85) - 83.2} = 44.1 \text{ in.}$$

$$= \frac{A_v f_y}{50 b_w} = \frac{(2 \times 0.11) \times 60{,}000}{50 \times 28} = 9.4 \text{ in.} \quad \text{(governs)}$$

Confinement of the longitudinal bars in accordance with ACI 21.3.3.3 must be provided only where hoops are required; that is why only 2 legs are used in this portion of the beam where stirrups with seismic hooks are required.

The maximum allowable spacing of the stirrups is $d/2 = 11.8$ in. (ACI 21.3.3.4), which is greater than that required for minimum shear reinforcement. A 9 in. spacing, starting at 52 in. from the face of the support is sufficient for the remaining portion of the beam.

Reinforcing bar cutoff points.

The negative reinforcement at the supports is 7-No. 8 bars. The location where 3 of the 7 bars can be terminated will be determined.

The third load combination is used to determine the cutoff point of the 3-No. 8 bars (0.7 times the dead load in combination with the probable flexural strengths M_{pr} at the ends of the member), since this combination produces the longest bar lengths. The design flexural strength ϕM_n provided by 4-No. 8 bars is 320 ft-kips. Therefore, the 3-No. 8 bars can be terminated after the required moment strength M_u has been reduced to 320 ft-kips.

The distance x from the support to the location where the moment is equal to 320 ft-kips can readily be determined by summing moments about section a-a in Figure 2-15:

$$\frac{1.7x^2}{2} - 69.2x + 737 = 320$$

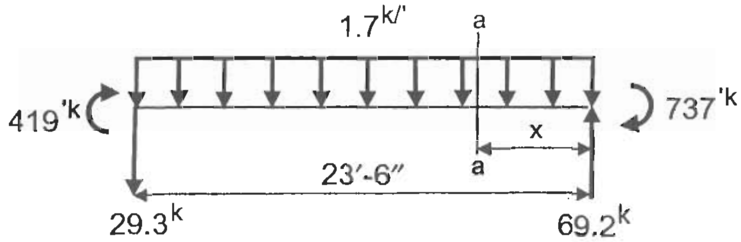


Figure 2-15 Cutoff Location of Negative Bars (SDC D)

Solving for x gives a distance of 6.6 ft from the face of the support.

The 3-No. 8 bars must extend a distance d = 23.5 in. (governs) or $12d_b = 12 \times 1.0 = 12$ in. beyond the distance x (ACI 12.10.3). Thus, from the face of the support, the total bar length must be at least equal to 6.6 + (23.5/12) = 8.6 ft. Also, the bars must extend a full development length ℓ_d beyond the face of the support (ACI 12.10.4), which is determined from ACI Eq. (12-1):

$$\frac{\ell_d}{d_b} = \frac{3}{40} \frac{f_y}{\sqrt{f_c'}} \frac{\alpha\beta\gamma\lambda}{\left(\dfrac{c + K_{tr}}{d_b}\right)}$$

where α = reinforcement location factor = 1.3 for top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 8 bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5 + 0.375 + \dfrac{1.0}{2} = 2.4 \text{ in.} \\ \dfrac{28 - 2(1.5 + 0.375) - 1.0}{2 \times 6} = 1.9 \text{ in.} \quad \text{(governs)} \end{cases}$$

K_{tr} = transverse reinforcement index = 0 (conservative)

$$\frac{c + K_{tr}}{d_b} = \frac{1.9 + 0}{1.0} = 1.9 < 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.3 \times 1.0 \times 1.0 \times 1.0}{1.9} = 48.7$$

$$\ell_d = 48.7 \times 1.0 = 48.7 \text{ in.} = 4.1 \text{ ft} < 8.6 \text{ ft}$$

Thus, the total required length of the 3-No. 8 bars must be at least 8.6 ft beyond the face of the support.

Flexural reinforcement shall not be terminated in a tension zone unless one or more of the conditions of ACI 12.10.5 are satisfied. In this case, the point of inflection is approximately 12.6 ft from the face of the support, which is greater than 8.6 ft. Thus, the No. 8 bars cannot be terminated here unless one of the conditions of ACI 12.10.5 is satisfied. In this case, check if the factored shear force V_u at the cutoff point does not exceed $2\phi V_n / 3$ (ACI 12.10.5.1). With No. 3 stirrups at 9 in. on center that are provided in this region of the beam, ϕV_n is determined by ACI Eqs. 11-1 and 11-2:

$$\phi V_n = \phi(V_c + V_s) = 0.85 \times \left(83.2 + \frac{0.22 \times 60 \times 23.5}{9} \right) = 100.0 \text{ kips}$$

$$\frac{2}{3}\phi V_n = 66.7 \text{ kips}$$

At 8.6 ft from the face of the support, $V_u = 69.2 - (1.7 \times 8.6) = 54.6$ kips, which is less than 66.7 kips. Therefore, the 3-No. 8 bars can be terminated at 8.6 ft from the face of the support.

Flexural reinforcement splices.

According to ACI 21.3.2.3, lap splices of flexural reinforcement must not be placed within a joint, within a distance $2h$ from the face of the joint (plastic hinge region), or at locations where analysis indicates flexural yielding due to inelastic lateral displacements of the frame. Lap splices must be confined by hoops or spiral reinforcement along the entire lap length, and the maximum spacing of the transverse reinforcement is $d/4$ or 4 in. In lieu of lap splices, mechanical and welded splices conforming to ACI 21.2.6 and 21.2.7, respectively, may be used (ACI 21.3.2.4).

Lap splices are determined for the No. 7 bottom bars. Since all of the bars are to be spliced within the required length, a Class B splice must be used (ACI 12.15.1, 12.15.2):

Class B splice length = $1.3\,\ell_d$

The development length ℓ_d is determined from ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 7 bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5 + 0.375 + \dfrac{0.875}{2} = 2.3 \text{ in.} \quad \text{(governs)} \\ \dfrac{28 - 2(1.5+0.375) - 0.875}{2\times 4} = 2.9 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index = 0 (conservative)

$$\frac{c+K_{tr}}{d_b} = \frac{2.3+0}{0.875} = 2.6 > 2.5\text{, use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.0\times1.0\times1.0\times1.0}{2.5} = 28.5$$

$$\ell_d = 28.5 \times 0.875 = 24.9 \text{ in.} = 2.1 \text{ ft}$$

Class B splice length = $1.3\,\ell_d = 1.3 \times 2.1 = 2.7$ ft

Use a 3 ft-0 in. splice length.

Figure 2-16 shows the reinforcement details for beam C4-C5.

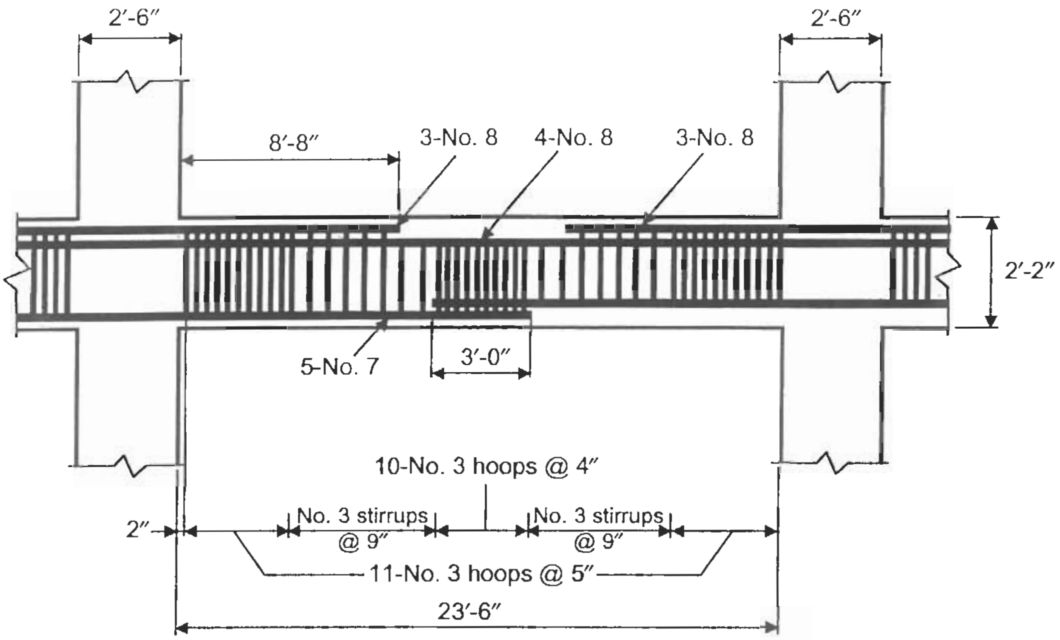


Figure 2-16 Reinforcement Details for Beam C4-C5 at the 2nd Floor Level (SDC D)

2.4.4.3 Design of Column C4

Table 2-29 contains a summary of the design axial forces, bending moments, and shear forces on column C4 for gravity and seismic loads.

Table 2-29 Summary of Design Axial Forces, Bending Moments, and Shear Forces on Column C4 between the 1st and 2nd Floor Levels (SDC D)

Load Case		Axial Force (kips)	Bending Moment (ft-kips)		Shear Force (kips)	
			N-S	E-W	N-S	E-W
Dead (D)		1,158	0	0	0	0
Live (L)*		134	0	0	0	0
Seismic (Q_E)		0**	$\pm$ 209	$\pm$ 354	$\pm$ 36	$\pm$ 71
Load Combinations						
$1.4D + 1.7L$		1,849	0	0	0	0
$1.4D + 0.5L + Q_E$	Case 1	1,688	209	106	36	21
	Case 2	1,688	63	354	11	71
$0.7D + Q_E$	Case 1	811	-209	-106	-36	-21
	Case 2	811	-63	-354	-11	-71

* Live load reduced per IBC 1607.9

** Axial forces due to seismic load effects in both the N-S and E-W directions are zero

Case 1: 100% N-S seismic force effect + 30% E-W seismic force effect

Case 2: 100% E-W seismic force effect + 30% N-S seismic force effect

According to the supplement to the IBC [2.7], orthogonal effects on this member need not be considered, since the axial forces due to seismic forces in both directions (which are equal to zero) are less than 20% of the axial load design strength. For illustration purposes, the column is designed when orthogonal effects are not considered and the results are compared to those where orthogonal effects are considered. Bending moments and shear forces are contained in Table 2-29 for both the N-S and E-W directions. In the N-S direction, the governing bending moment and shear force are due to 25% of the seismic forces F_x from Table 2-21 acting on the building without the presence of shear walls, which is one of the requirements that is stipulated for dual systems (IBC 1617.6.1).

Two methods for combining orthogonal load effects from seismic forces are given in IBC 1620.2.2. In the first method, members are designed for the effects from 100% of the forces acting in one direction plus 30% of the forces acting in the perpendicular direction. Bending moments and shear forces under Cases 1 and 2 in Table 2-29 have been determined using this method. The square root of the sum of the squares (SRSS) method can also be used.

Since the factored compressive axial loads exceed $A_g f_c'/10 = 30^2 \times 4/10 =$ 360 kips, the provisions of ACI 21.4 are applicable. Thus, the following two criteria must be satisfied (ACI 21.4.1):

- Shortest cross-sectional dimension = 30 in. > 12 in. O.K.
- Ratio of shortest cross-sectional dimension to perpendicular dimension = 1.0 > 0.4 O.K.

Design for axial force and bending.

When orthogonal effects are not considered, the governing load combinations are due to seismic forces acting the E-W direction, since bending moments generated in the column due to seismic forces in that direction are larger than those generated from seismic forces in the N-S direction. A 30 × 30 in. column with 12-No. 10 bars ($\rho_g = 1.69\%$) is adequate for column C4 supporting the second floor level. Figure 2-17 contains the interaction diagram for this column for uniaxial bending due to seismic forces in the E-W direction.

When orthogonal effects are considered, the column must be designed for biaxial bending due to the load combinations given in Table 2-29 for seismic load cases 1 and 2 as prescribed in IBC 1620.2.2. For the column reinforced with 12-No. 10 bars, interaction diagrams cut at an angle of 0 degrees (pure axial load), 27 degrees, and 80 degrees through the biaxial interaction surface would show that the load combination points fall well within the interaction surface. Therefore, 12-No. 10 bars are also adequate for the 30 × 30 in. column subjected to biaxial bending. Note that the angles of the interaction diagrams are obtained by taking the inverse tangent of the ratio of E-W and N-S factored bending moments. For example, for seismic load case 1 in the second or third load combinations, the angle of the interaction diagram = $\tan^{-1}(106/209) =$ 27 degrees (see

Table 2-29). The provided reinforcement ratio is within the allowable range of 1% and 6% (ACI 21.4.3.1).

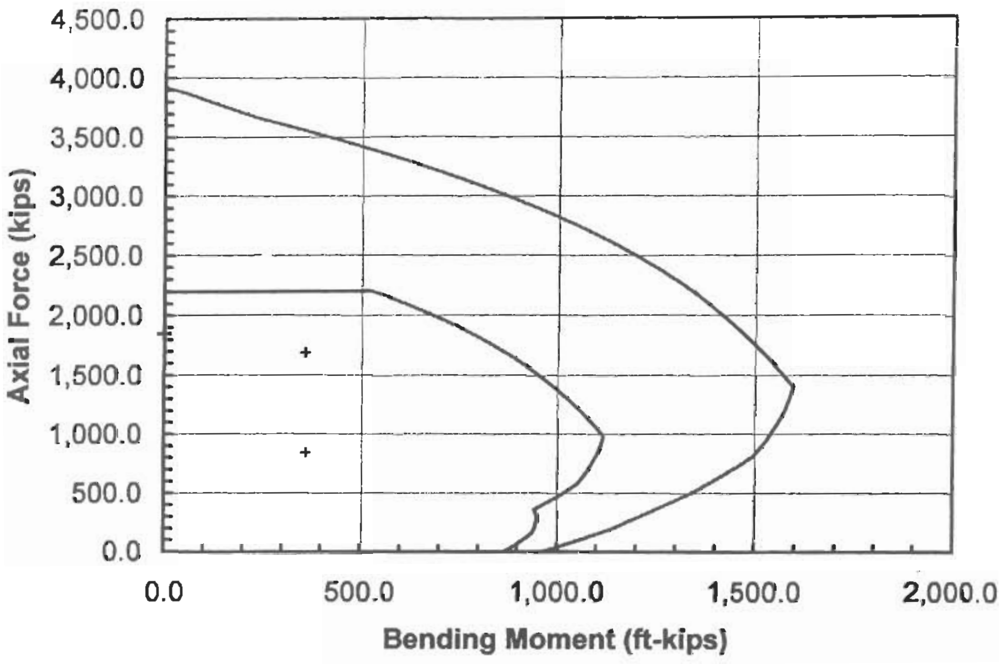


Figure 2-17 Design and Nominal Strength Interaction Diagrams for Column C4 Supporting the 2nd Floor Level when Orthogonal Effects are not Considered (SDC D)

Relative Flexural Strength of Columns and Girders.

ACI 21.4.2 requires that the sum of flexural strengths of columns at a joint must be greater than or equal to 6/5 times the sum of flexural strengths of girders framing into that joint. The intent is to provide columns with sufficient strength so that they will not yield prior to the beams. Yielding at both ends of a column prior to the beams could result in total collapse of the structure. Only seismic load combinations need to be considered when checking the relative strengths of columns and girders.

When computing the nominal flexural strength of girders in T-beam construction, slab reinforcement within an effective slab width defined in ACI 8.10 shall contribute to the flexural strength if the slab reinforcement is developed at the critical section for flexure (ACI 21.4.2.2). For an interior beam spanning in the E-W direction, the effective slab width is determined as follows:

- 16 × slab thickness + beam width = (16 × 8) + 28 = 156 in.
- center-to-center spacing of beams = 22 × 12 = 264 in.
- beam span/4 = (26 × 12)/4 = 78 in. (governs)

The minimum required area of steel in the 78-in. effective width = 0.0018 × 78 × 8 = 1.12 in.^{2} (ACI 13.3.1), which corresponds to 6-No. 4 bars @ 78/6 = 13 in. spacing. This spacing is less than the maximum allowable spacing of $2h$ = 16 in. Provide No. 4 @ 13 in. at both the top and bottom of the slab.

Based on the reinforcement in beam C4-C5 (see Table 2-28) and the reinforcement in the effective width of the slab, a strain compatibility analysis of the section yields $M_n^- =$ 838 ft-kips and $M_n^+ = 405$ ft-kips. Therefore, $\sum M_g = 838 + 405 = 1{,}243$ ft-kips (see Figure 2-18 for sidesway to the left; due to symmetric distribution of flexural reinforcement, sidesway to the right yields the same results for the negative and positive moment strengths).

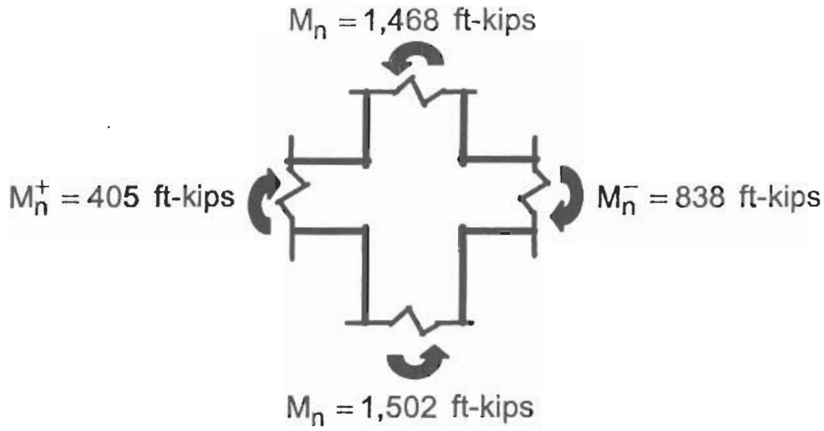


Figure 2-18 Relative Flexural Strength of Columns and Girders when Orthogonal Effects are not Considered (SDC D)

Column flexural strength is determined for the factored axial force resulting in the lowest flexural strength, consistent with the direction of lateral forces considered. For the upper end of the lower column framing into the joint (i.e., the column supporting floor level 2), the minimum $M_n = 1{,}502$ ft-kips, which corresponds to $P_u = 811$ kips when orthogonal effects are not considered (see Figure 2-17). Similarly, for the lower end of the upper column framing into the joint (i.e., the column supporting floor level 3), $M_n = 1{,}468$ ft-kips, which corresponds to $P_u = 736$ kips. Therefore, $\sum M_c = 2{,}970$ ft-kips.

Check Eq. 21-1:

$$\sum M_c = 2{,}970 \text{ ft-kips} > \frac{6}{5}\sum M_g = \frac{6}{5} \times 1{,}243 = 1{,}492 \text{ ft-kips} \quad \text{O.K.}$$

The same calculations are performed when orthogonal effects are considered. For the upper end of the lower column framing into the joint, the minimum $M_n = 1{,}360$ ft-kips, which corresponds to P_u = 811 kips for an angle cut at 27 degrees through the biaxial surface. Similarly, for the lower end of the upper column framing into the joint, M_n = 1,337 ft-kips, which corresponds to P_u = 736 kips. Therefore, $\sum M_c$ = 2,697 ft-kips.

Check Eq. 21-1:

$$\sum M_c = 2{,}697 \text{ ft-kips} > \frac{6}{5}\sum M_g = \frac{6}{5}\times 1{,}243 = 1{,}492 \text{ ft-kips} \quad \text{O.K.}$$

Thus, lateral strength and stiffness of this column can be considered when determining the strength and stiffness of the structure whether orthogonal effects are considered or not.

Design for shear.

Shear requirements for columns in special moment frames are contained in ACI 21.4.5. Similar to beams, the method of determining design shear forces in columns takes into consideration the likelihood of yielding (i.e., plastic hinges forming) in regions near the ends of the column. To properly confine the concrete and to maintain lateral support of the longitudinal bars in regions where yielding is expected, the transverse reinforcement requirements of ACI 21.4.4 must also be satisfied.

Confinement reinforcement. Special transverse reinforcement for confinement is required over a distance ℓ_o from each joint face at both column ends where ℓ_o is equal to the largest of (ACI 21.4.4.4):

- Depth of member = 30 in. (governs)
- Clear span/6 = [(12 × 12) – 26]/6 = 19.7 in.
- 18 in.

Transverse reinforcement within the distance ℓ_o shall not be spaced greater than the smallest of (ACI 21.4.4.2):

- Minimum member dimension/4 = 30/4 = 7.5 in.
- 6(diameter of longitudinal reinforcement) = 6 × 1.27 = 7.6 in.
- $s_x = 4 + \left(\frac{14 - h_x}{3}\right) = 4 + \left(\frac{14 - 10}{3}\right) = 5.3 \text{ in.} > 4 \text{ in.}$ (governs)

  < 6 in.

where h_x = maximum horizontal spacing of hoop or crosstie legs on all faces of the 30 × 30 in. column

$$= \frac{30 - 2(1.5 + 0.5) - 1.27}{3} + 1.27 + 0.5 = 10 \text{ in.} < 14 \text{ in.} \quad \text{O.K. (ACI 21.4.4.3)}$$

assuming No. 4 rectangular hoops with crossties around every longitudinal bar. Therefore, try 5 in. spacing.

Minimum required cross-sectional area of rectangular hoop reinforcement A_{sh} is the larger value obtained from Eqs. 21-3 and 21-4:

$$A_{sh} = \frac{0.3 s h_c f_c'}{f_{yh}} \left[\left(\frac{A_g}{A_{ch}} \right) - 1 \right] = \frac{0.3 \times 5 \times 26.5 \times 4}{60} \left[\left(\frac{30^2}{729} \right) - 1 \right] = 0.6 \text{ in.}^2$$

$$= \frac{0.09 s h_c f_c'}{f_{yh}} = \frac{0.09 \times 5 \times 26.5 \times 4}{60} = 0.80 \text{ in.}^2 \quad \text{(governs)}$$

where h_c = cross-sectional dimension of column core measured center-to-center of confinement reinforcement

= $30 - 2[1.5 + (0.5/2)] = 26.5$ in.

A_{ch} = cross-sectional area of member measured out-to-out of transverse reinforcement

= $[30 - (2 \times 1.5)]^2 = 729$ in.^{2}

Using No. 4 hoops with 2 crossties provides $A_{sh} = 4 \times 0.2 = 0.8$ in.^{2}, which is equal to the minimum required area from Eq. 21-4. Use 5 in. spacing for the transverse reinforcement at the column ends.

Transverse reinforcement for shear. According to ACI **21.4.5.1**, shear forces are computed from statics assuming that moments of opposite sign act at the joint faces corresponding to the probable flexural strengths M_{pr} associated with the range of factored axial loads on the column. For columns in the first story, it is possible for the base of the column to develop its probable flexural strength. For columns above the first story, which is applicable to the column in this example, shear forces based on the probable flexural strengths of the beams framing into the joint will usually control: considering equilibrium at the joint, the largest bending moments and corresponding shear forces that can be transmitted to the column occurs when plastic hinges form at the ends of the beams. Thus, design shear forces in columns need not exceed those determined from joint strengths based on M_{pr} of the beams framing into the joint. Also, the design shear force must not be taken less than that determined from the structural analysis. Sidesway to the right and to the left must be considered when calculating the maximum design shear forces.

It is conservative to assume that the maximum M_{pr} of the column is equal to the moment at the balanced point. When orthogonal effects are not considered and using $f_y = 1.25 \times 60 = 75$ ksi and $\phi = 1.0$, maximum $M_{pr} = 1{,}731$ ft-kips. Design shear force V_e based on the probable flexural strength of the column at the column ends is:

$$V_e = \frac{2 \times 1{,}731}{12 - (26/12)} = 352 \text{ kips}$$

When orthogonal effects are considered, the largest bending moment at the balanced point occurs for an angle cut at 80 degrees through the biaxial surface, which is M_{pr} = 1,590 kips. Design shear force V_e based on this probable flexural strength is:

$$V_e = \frac{2 \times 1{,}590}{12 - (26/12)} = 323 \text{ kips}$$

The positive probable flexural strength of the beam framing into the joint at the face of the column at the 2nd floor level is 419 ft-kips (see Figure 2-14). The negative probable flexural strength of the beam on the other side of the column is 737 ft-kips. Assuming that the flexural reinforcement in the 1st floor beams framing into the column is the same as the 2nd floor beams, V_e based on the probable flexural strengths of the beams is:

$$V_e = \frac{2\left(\dfrac{419 + 737}{2}\right)}{12 - (26/12)} = 118 \text{ kips} \quad \text{(governs)}$$

This design shear force is larger than the maximum shear force obtained from analysis, which is 71 kips (see Table 2-29).

Since the factored axial forces are greater than $A_g f'_c/20 = 180$ kips, the shear strength of the concrete may be used (ACI 21.4.5.2). The shear capacity of the column is checked using ACI Eq. 11-4 for members subjected to axial compression:

$$V_c = 2\left(1 + \frac{N_u}{2{,}000 A_g}\right)\sqrt{f'_c}\, b_w d$$

$$= 2\left(1 + \frac{811{,}000}{2{,}000 \times 30^2}\right)\sqrt{4{,}000} \times 30 \times 25.5/1{,}000 = 140.4 \text{ kips}$$

$$\phi V_c = 0.85 \times 140.4 = 119 \text{ kips} > 118 \text{ kips}$$

where N_u = 811 kips is the smallest axial force on the section (see Table 2-29) and d = 25.5 in. was obtained from a strain compatibility analysis.

Thus, the No. 4 hoops at 5 in. on center required for confinement over the distance ℓ_o at the column ends is also adequate for shear.

According to ACI 21.4.4.6, the remainder of the column must contain hoop reinforcement with center-to center spacing not to exceed 6 times the diameter of the longitudinal column bars = 6 × 1.27 = 7.6 in. or 6 in. (governs). For simpler detailing, use 5 in. spacing of hoops and crossties over entire column height.

Splice length of longitudinal reinforcement.

Lap splices in columns of special moment frames are permitted only within the center half of the member and must be designed as tension lap splices (ACI 21.4.3.2). Also, they must be confined over the entire lap length with transverse reinforcement conforming to ACI 21.4.4.2 and 21.4.4.3. In lieu of lap splices, mechanical splices conforming to ACI 21.2.6 and welded splices conforming to ACI 21.2.7.1 may be utilized.

Since all of the bars are to be spliced at the same location, a Class B splice is required (ACI 12.15.1, 12.15.2). The development length ℓ_d is computed from ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 7 and larger bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5+0.5+\dfrac{1.27}{2} = 2.6 \text{ in.} \quad \text{(governs)} \\[2ex] \dfrac{30-2(1.5+0.5)-1.27}{2\times 3} = 4.1 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index

$$= \frac{A_{tr} f_{yt}}{1{,}500 sn} = \frac{4\times 0.2\times 60{,}000}{1{,}500\times 5\times 4} = 1.6$$

$$\frac{c+K_{tr}}{d_b}=\frac{2.6+1.6}{1.27}=3.3>2.5\,, \quad \text{use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b}=\frac{3}{40}\times\frac{60{,}000}{\sqrt{4{,}000}}\times\frac{1.0\times1.0\times1.0\times1.0}{2.5}=28.5$$

$$\ell_d=28.5\times1.27=36.2 \text{ in.}$$

Class B splice length = $1.3\times36.2=47.1$ in.

Use a 4 ft-0 in. splice length.

Reinforcement details for column C4 are shown in Figure 2-19.

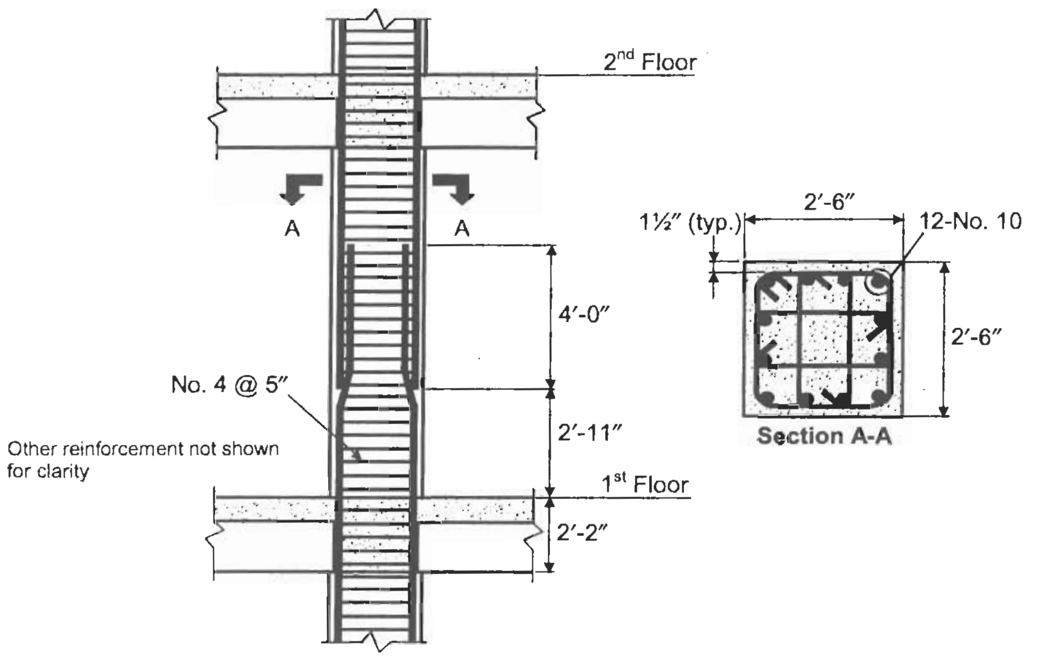


Figure 2-19 Reinforcement Details for Column C4 Supporting the 2nd Floor Level (SDC D)

2.4.4.4 Design of Beam-Column Joint

This section outlines the design of interior and exterior beam-column connections along column line 4. Similar calculations can be performed for joints along other column lines.

The overall integrity of special moment frames is dependent on the behavior of beam-column joints. Degradation of joints can result in large lateral deformations that can cause excessive damage or even failure.

Interior Joint.

Transverse reinforcement for confinement. Transverse reinforcement in a beam-column joint is required to adequately confine the concrete to ensure its ductile behavior and to allow it to maintain its load-carrying capacity even after possible spalling of the outer shell. ACI 21.5.2 requires that transverse hoop reinforcement in accordance with ACI 21.4.4 be provided within the joint, unless the joint is confined on all four sides by beams that have widths equal to at least 75% the column width. When joints are adequately confined on four sides, transverse reinforcement within the joint may be reduced to one-half of that required by ACI 21.4.4 and the hoop spacing is permitted to be a maximum of 6 in.

At an interior joint in the example building, beams frame into all four sides of the column and the width ratio is 28/30 = 0.93 > 0.75. Therefore, transverse reinforcement can be reduced by 50% according to ACI 21.5.2.2 as noted above. However, for detailing simplicity, use the transverse reinforcement required at the column ends (No. 4 @ 5 in.) through the joint.

Shear strength of joint. Figure 2-20 shows the interior beam-column joint at the 2nd floor level. The shear strength is checked in the E-W direction in accordance with ACI 21.5.3. The shear force at section *x-x* is obtained by subtracting the column shear force from the sum of the tensile force in the top beam reinforcement and the compressive force at the top of the beam on the opposite face of the column. Since development of inelastic rotations at the joint face is associated with strains in the beam flexural reinforcement significantly greater than the yield strain, joint shear forces generated by beam reinforcement are calculated based on a stress in the reinforcement equal to 1.25 f_y (ACI 21.5.1.1):

$$T_1 \text{ (7 - No.8)} = A_s(1.25 f_y) = (7 \times 0.79) \times (1.25 \times 60) = 415 \text{ kips}$$

$$T_2 \text{ (5 - No.7)} = (5 \times 0.60) \times (1.25 \times 60) = 225 \text{ kips}$$

Column horizontal shear force V_h can be obtained by assuming that adjoining floors are deformed so that plastic hinges form at the ends of the beams. For the beams in this example, $M_{pr}^{-} = 737$ ft-kips and $M_{pr}^{+} = 419$ ft-kips (see Figure 2-14).

Since the lengths of the columns above and below the joint are equal, moments M_u in the columns above and below the joint are (737 + 419)/2 = 578 ft-kips. Shear force V_h in the column is:

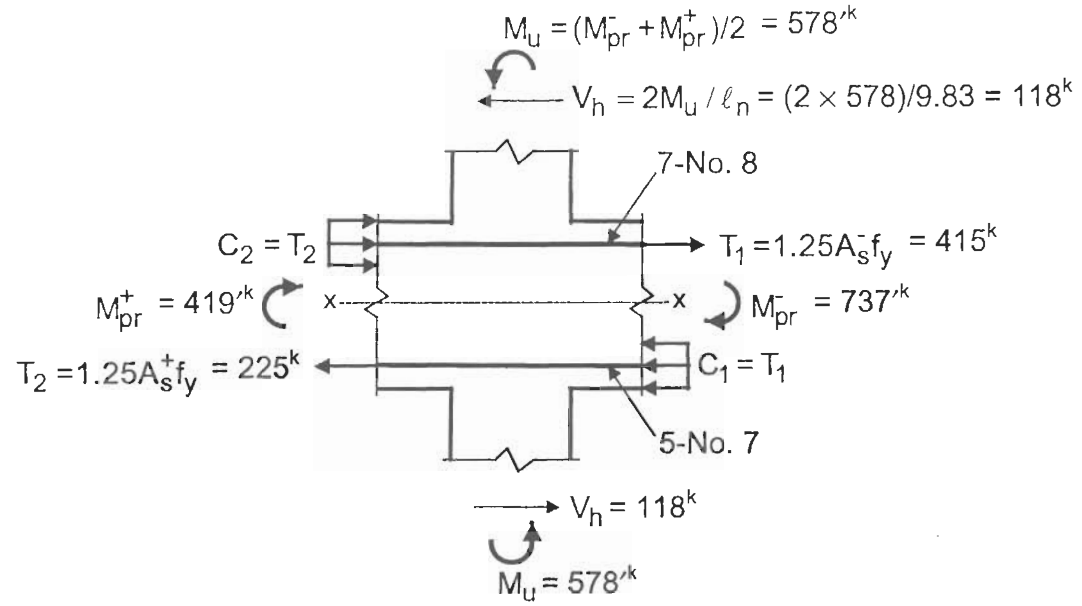


Figure 2-20 Shear Analysis of Interior Beam-Column Joint in E-W Direction (SDC D)

$$V_h = \frac{2M_u}{\ell_n} = \frac{2 \times 578}{12-(26/12)} = 118 \text{ kips}$$

The net shear force at section *x-x* is $T_1 + C_2 - V_h = 415 + 225 - 118 = 522$ kips

For a joint confined on all four faces, nominal shear strength ϕV_c is determined from ACI 21.5.3.1:

$$\phi V_c = \phi 20\sqrt{f_c'}A_j = 0.85 \times 20\sqrt{4{,}000} \times 900/1{,}000 = 968 \text{ kips} > 522 \text{ kips} \quad \text{O.K.}$$

where A_j = effective cross-sectional area within a joint in a plane parallel to the plane of reinforcement generating shear in the joint (see ACI Figure R21.5.3). The joint depth is the overall depth of the column in the direction of analysis, which is 30 in. The effective width of the joint is the smaller of (1) beam width + joint depth = 28 + 30 = 58 in. or (2) beam width plus twice the smaller perpendicular distance from the edge of the beam to the edge of the column = 28 + (2 × 1) = 30 in. (governs). Thus, A_j = 30 × 30 = 900 in.^{2}

Joint shear strength is a function of concrete strength and effective cross-sectional area A_j only. Tests results show that shear strength of a joint is not altered significantly with changes in transverse reinforcement provided a minimum amount of such reinforcement is present. Therefore, it is essential that at least minimum transverse reinforcement as specified in ACI 21.5.2 be provided through the joint regardless of the magnitude of calculated shear force in the joint. In cases when the net shear force exceeds the design strength prescribed in ACI 21.5.3.1, only the concrete strength or the effective cross-sectional area can be increased to increase shear capacity.

Reinforcement details for an interior joint are shown in Figure 2-21.

Shear strength requirements must also be satisfied for the N-S direction.

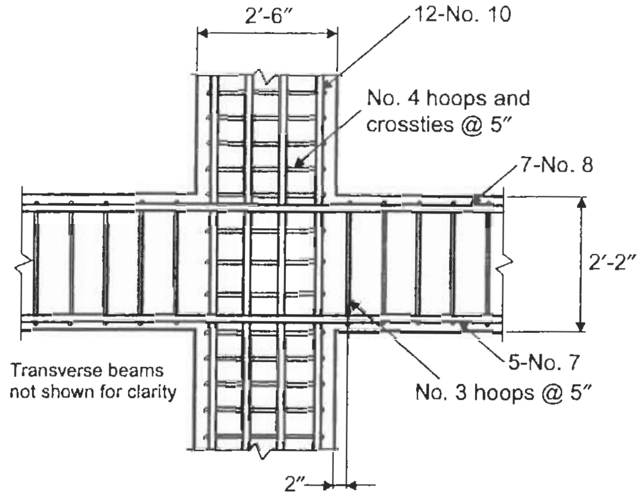


Figure 2-21 Reinforcement Details at Interior Beam-Column Joint (SDC D)

Exterior Joint.

It is assumed for purposes of this example that the beam framing into the edge column along column line 4 has the same flexural reinforcement as the interior beams along this column line.

Transverse reinforcement for confinement. Since an exterior joint is confined on less than four sides, transverse hoop reinforcement in accordance with ACI 21.4.4 must be provided within the joint (ACI 21.5.2.1). For detailing simplicity, use the transverse reinforcement required at the column ends through the joint.

Shear strength of joint. Figure 2-22 shows the exterior joint at the 2nd floor level. In this case, the shear force at section *x-x* is determined by subtracting the column shear force from the tensile force in the top beam reinforcement.

Since the lengths of the columns above and below the joint are equal, moments M_u in the columns above and below the joint are 737/2 = 369 ft-kips. Shear force V_h in the column is:

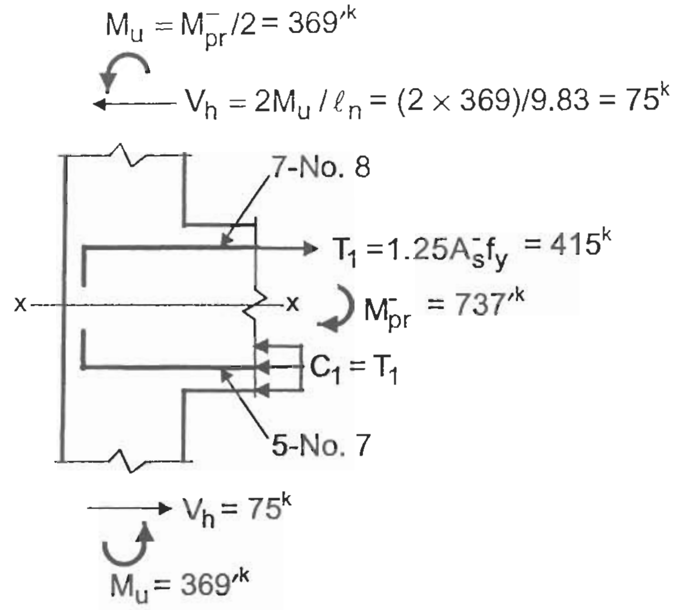


Figure 2-22 Shear Analysis of Exterior Beam-Column Joint in E-W Direction (SDC D)

$$V_h = \frac{2M_u}{\ell_n} = \frac{2\times 369}{12-(26/12)} = 75 \text{ kips}$$

The net shear force at section *x-x* is $T_1 - V_h = 415 - 75 = 340$ kips

For a joint confined on three faces, the nominal shear strength ϕV_c is determined from ACI 21.5.3.1:

$$\phi V_c = \phi 15\sqrt{f_c'}A_j = 0.85\times 15\sqrt{4{,}000}\times 26^2/1{,}000 = 545 \text{ kips} > 340 \text{ kips} \quad \text{O.K.}$$

Since the beam width is slightly larger than the column width, it is possible for some of the beam longitudinal bars to be placed outside of the confined column core. However, these bars are confined by beams framing into the column in the N-S direction; thus, no additional transverse reinforcement in accordance with ACI 21.5.2.3 is required.

Beam flexural reinforcement terminated in a column must extend to the far face of the confined column core and must be anchored in tension and compression according to ACI 21.5.4 and Chapter 12, respectively (ACI 21.5.1.3). The development length ℓ_{dh} for a bar with a standard 90-degree hook in normal weight concrete is the largest of (ACI 21.5.4.1):

- 8(diameter of longitudinal bar) = 8 × 1.0 = 8 in. for No. 8 top bars

  = 8 × 0.875 = 7 in. for No. 7 bottom bars

- 6 in.

- $\dfrac{f_y d_b}{65\sqrt{f_c'}} = \dfrac{60{,}000 \times 1.0}{65\sqrt{4{,}000}} = 14.6$ in. for No. 8 top bars (governs)

  $= \dfrac{60{,}000 \times 0.875}{65\sqrt{4{,}000}} = 12.8$ in. for No. 7 bottom bars (governs)

Required development lengths for both the top and bottom reinforcement can be accommodated within the 26-in. deep column.

Reinforcement details for the exterior joint are shown in Figure 2-23.

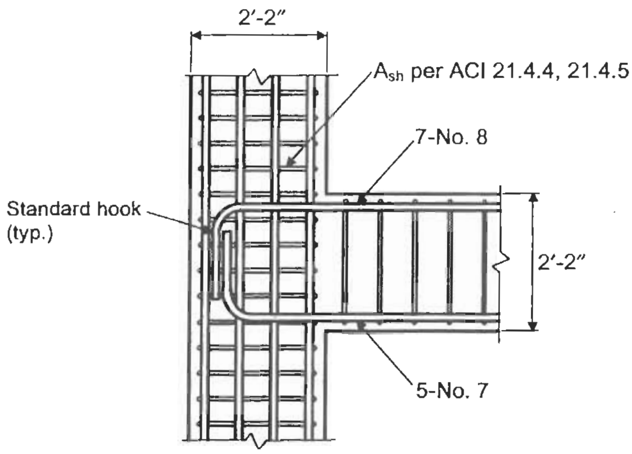


Figure 2-23 Reinforcement Details at Exterior Beam-Column Joint (SDC D)

2.4.4.5 Design of Shear Wall on Line 7

This section outlines the design of the shear wall on line 7. According to the supplement to the IBC [2.7], orthogonal effects need not be considered, since the axial forces due to seismic forces (which are equal to zero) are less than 20% of the axial load design strength. Table 2-30 contains a summary of the governing design axial forces, bending moments, and shear forces at the base of the wall due to gravity and seismic forces.

As noted above, special reinforced concrete structural walls are required in dual systems for buildings assigned to SDC D; thus, the provisions of ACI 21.6, as well as the applicable provisions in Chapters 1 through 18, must be satisfied.

Table 2-30 Summary of Design Axial Forces, Bending Moments, and Shear Forces at Base of Shear Wall on Line 7 (SDC D)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	3,094	0	0
Live (L)	279	0	0
Seismic (Q_E)	0	± 74,783	± 1,360
Load Combinations			
$1.4D + 1.7L$	4,806	0	0
$1.4D + 0.5L + Q_E$	4,471	74,783	1,360
$0.7D + Q_E$	2,166	-74,783	-1,360

Design for shear.

Reinforcement requirements. Special reinforced structural walls are to be provided with reinforcement in two orthogonal directions in the plane of the wall in accordance with ACI 21.6.2. The minimum reinforcement ratio in both directions is 0.0025, unless the design shear force is less than or equal to $A_{cv}\sqrt{f_c'}$, where A_{cv} is the gross area of concrete bounded by the web thickness and the length of wall in the direction of analysis. In such cases, minimum reinforcement in accordance with ACI 14.3 for ordinary walls must be provided. For the wall in this example, $A_{cv} = 16 \times [(22 \times 12) + 36] = 4{,}800$ in.^{2}, so that

$$A_{cv}\sqrt{f_c'} = 4{,}800 \times \sqrt{4{,}000}/1{,}000 = 304 \text{ kips} < V_u = 1{,}360 \text{ kips}$$

Therefore, the minimum reinforcement ratio is 0.0025 and the maximum spacing is 18 in. (ACI 21.6.2.1).

Two curtains of reinforcement are required in a wall when the in-plane factored shear force exceeds $2A_{cv}\sqrt{f_c'} = 2 \times 304 = 608$ kips. In this case, two curtains are required, since 1,360 kips > 608 kips.

The minimum required reinforcement in each direction per foot of wall is $0.0025 \times 16 \times 12 = 0.48$ in.^{2} Assuming No. 5 bars in two curtains, required spacing s is

$$s = \frac{2 \times 0.31}{0.48} \times 12 = 15.5 \text{ in.} < 18 \text{ in.}$$

Try 2 curtains of No. 5 bars spaced at 15 in.

Shear strength requirements. ACI Eq. 21-7 is used to determine nominal shear strength V_n of structural walls:

$$V_n = A_{cv}(\alpha_c \sqrt{f'_c} + \rho_n f_y)$$

where $\alpha_c = 2$ for ratio of wall height to length $h_w / \ell_w =$ 148/25 = 5.9 > 2 (ACI 21.6.4.1).

For 2 curtains of No. 5 horizontal bars spaced at 15 in. ($\rho_n = 0.62/(16 \times 15) = 0.0026$):

$$\phi V_n = 0.85 \times 4{,}800 \times [2\sqrt{4{,}000} + (0.0026 \times 60{,}000)]/1{,}000$$

$$= 1{,}149 \text{ kips} < V_u = 1{,}360 \text{ kips} \qquad \text{N.G.}$$

where $\phi = 0.85$ for walls with $h_w / \ell_w > 2$ (ACI 9.3.4(a)). Therefore, use 2 curtains of No. 5 bars @ 10 in. on center in horizontal direction (ϕV_n = 1,465 kips). Note that V_n = 1,724 kips is less than the upper limit on shear strength, which is $8A_{cv}\sqrt{f'_c} = 8 \times 304 = 2{,}432$ kips (ACI 21.6.4.4).

Reinforcement ratio ρ_v for the vertical reinforcement must not be less than ρ_n when $h_w / \ell_w \leq$ 2.0 (ACI 21.6.4.3). Since h_w / ℓ_w = 5.9 > 2, use minimum reinforcement ratio of 0.0025.

Use 2 curtains of No. 5 bars spaced at 12 in. on center in the vertical direction (ρ_v = 0.0032 > 0.0025).

Design for axial force and bending.

Structural walls subjected to combined flexural and axial loads are designed in accordance with ACI 10.2 and 10.3 except that ACI 10.3.6 and the nonlinear strain requirements of ACI 10.2.2 do not apply (ACI 21.6.5).

Figure 2-24 contains the interaction diagram of the wall. The wall is reinforced with 32-No. 11 bars in the 36 × 36 in. boundary elements at both ends of the wall and 2-No. 5 vertical bars @ 12 in. in the web. As seen from the figure, the wall is adequate for the load combinations in Table 2-30.

Special boundary elements.

The need for special boundary elements at the edges of structural walls is evaluated in accordance with ACI 21.6.6.2 or 21.6.6.3. The displacement-based approach in ACI 21.6.6.2 is utilized in this example. In this method, the wall is displaced at the top an amount equal to the expected design displacement; special boundary elements are required to confine the concrete when the strain in the extreme compression fiber of the wall exceeds a critical value. This method is applicable to walls or wall piers that are essentially continuous in cross-section over the entire height and designed to have one critical section for flexure and axial loads.

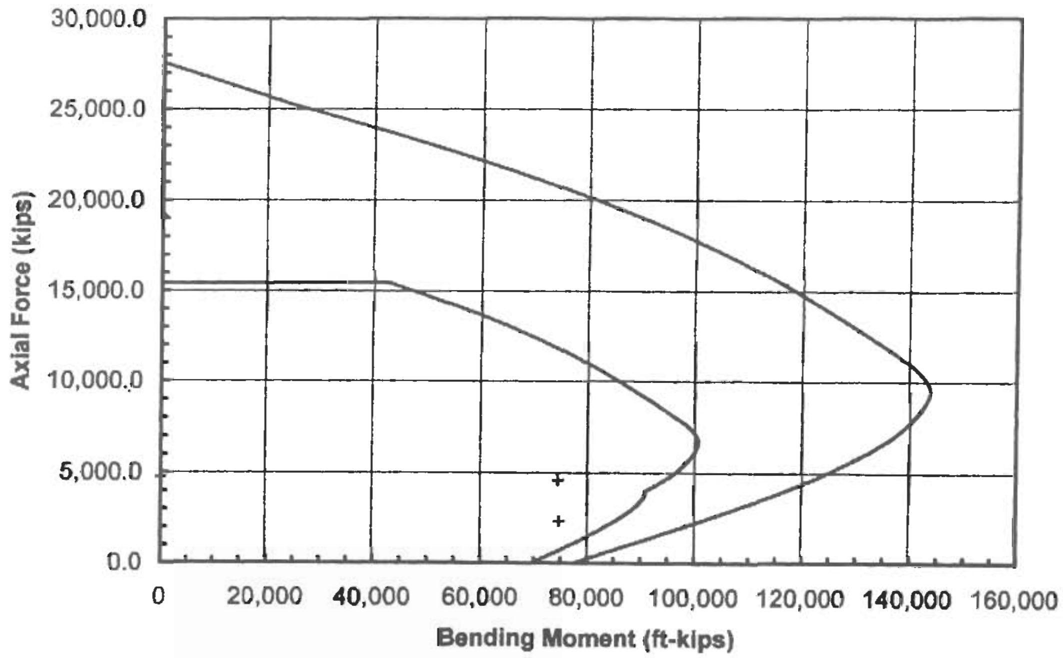


Figure 2-24 Design and Nominal Strength Interaction Diagrams for the Shear Wall Along Line 7 (SDC D)

Compression zones are to be reinforced with special boundary elements where (Eq. 21-8):

$$c \geq \frac{\ell_w}{600(\delta_u / h_w)}, \quad \delta_u / h_w \geq 0.007$$

where c = distance from extreme compression fiber to the neutral axis per ACI 10.2.7 calculated for the factored axial force and nominal moment strength, consistent with the design displacement δ_u, resulting in the largest neutral axis depth

ℓ_w = length of entire wall or segment of wall considered in the direction of the shear force

δ_u = design displacement

= total lateral displacement expected for the design-basis earthquake as specified by the governing code

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

The lower limit on the quantity δ_u / h_w is specified to require moderate wall deformation capacity for stiff buildings.

In this example, ℓ_w = 25 ft = 300 in., h_w = 148 ft = 1,776 in., δ_u is equal to δ_x from Table 2-23, which is 18.53 in. at the top of the wall, and $\delta_u / h_w = 0.0104 > 0.007$. Therefore, special boundary elements are required if c is greater than or equal to $300/(600 \times 0.0104) = 47.9$ in.

The distance c to be used in Eq. 21-8 is the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with the design displacement δ_u. From a strain compatibility analysis, the largest c is equal to 65.5 in. corresponding to a factored axial load of 4,471 kips and nominal moment strength 120,431 ft-kips, which is greater than 47.9 in. Therefore, special boundary elements are required.

Special boundary elements must extend horizontally from the extreme compression fiber a distance not less than the larger of the following (ACI 21.6.6.4):

- $c - 0.1\ell_w = 65.5 - (0.1 \times 300) = 35.5$ in. (governs)
- $c/2 = 65.5/2 = 32.8$ in.

Special boundary transverse reinforcement in accordance with ACI 21.6.6.4 is provided in the 36-in. boundary elements at the ends of the wall.

Vertical extent of special boundary transverse reinforcement from base of wall is the larger of (ACI 21.6.6.2(b)):

- $\ell_w = 300$ in. $= 25$ ft (governs)
- $\dfrac{M_u}{4V_u} = \dfrac{74{,}783}{4 \times 1{,}360} = 13.7$ ft

Special boundary element transverse reinforcement.

Provisions for the amount and spacing of transverse reinforcement required in the special boundary elements are contained in ACI 21.6.6.4. In particular, transverse reinforcement requirements of ACI 21.4.4.1 through 21.4.4.3 for columns in special moment frames must be satisfied, except for Eq. 21-3.

Assuming No. 4 rectangular hoops and crossties around every other longitudinal bar in both directions of the 36 × 36 in. special boundary elements, the maximum allowable spacing is the smallest of the following:

- Minimum member dimension/4 = 36/4 = 9.0 in.
- 6(diameter of longitudinal reinforcement) = 6 × 1.41 = 8.5 in.
- $s_x = 4 + \frac{14 - h_x}{3} = 4 + \frac{14 - 9.6}{3} = 5.5$ in. > 4.0 in. (governs) < 6.0 in.

where h_x = maximum horizontal spacing of hoop or crosstie legs on all faces of the 36 × 36 in. special boundary elements

$$= 2\left[\frac{36 - 2(1.5 + 0.5) - 1.41}{8}\right] + 1.41 + 0.5 = 9.6 \text{ in.} < 14 \text{ in.} \quad \text{O.K.} \quad (21.4.4.3)$$

Assuming a spacing of 5 in., required area of transverse reinforcement is determined from Eq. 21-4:

$$A_{sh} = \frac{0.09 s h_c f'_c}{f_{yh}} = \frac{0.09 \times 5 \times 32.5 \times 4}{60} = 0.98 \text{ in.}^2$$

where h_c = cross-sectional dimension of boundary element core measured center-to-center of confinement reinforcement

= 36 – 2[1.5 + (0.5/2)] = 32.5 in.

No. 4 hoops with crossties around every other longitudinal bar provides A_{sh} = 5 × 0.2 = 1.0 in.^{2} > 0.98 in.^{2} O.K.

Splice length of reinforcement.

Class B lap splices are utilized for the longitudinal reinforcement in the special boundary elements and the vertical bars in the web. Mechanical connectors may be considered as an alternative to lap splices for the large bars in the special boundary element (ACI 21.6.6.4(f)). No splices are required for the No. 5 horizontal bars in the web, since full length bars weigh approximately 1.043 × 25 = 26 lbs. and are easily installed.

Vertical bars in special boundary elements. For the No. 11 vertical bars in the special boundary elements, ℓ_d is determined from Eq. (12-1), since this results in a longer splice length than determined from ACI 21.5.4 (ACI 21.6.2.3). Assuming no more than 50% of the bars are spliced at any one location:

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 7 and larger bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5+0.50+\dfrac{1.41}{2} = 2.7 \text{ in.} \qquad \text{(governs)} \\ \dfrac{36-2(1.5+0.5)-1.41}{2\times 4} = 3.8 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index

$$= \frac{A_{tr} f_{yt}}{1{,}500sn} = \frac{(5\times 0.2)\times 60{,}000}{1{,}500\times 5\times 5} = 1.6$$

$$\frac{c+K_{tr}}{d_b} = \frac{1.5+2.7}{1.41} = 3.0 > 2.5, \quad \text{use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40}\times\frac{60{,}000}{\sqrt{4{,}000}}\times\frac{1.0\times 1.0\times 1.0\times 1.0}{2.5} = 28.5$$

$$\ell_d = 28.5\times 1.41 = 40.2 \text{ in.}$$

Class B splice length = $1.3\ell_d = 1.3\times 40.2 = 52.3$ in.

Use a 4 ft-6 in. splice length for the No. 11 bars, with splices staggered at least 24 in. (ACI 12.15.4.1).

Vertical bars in wall web. The provisions of ACI Chapter 12 are utilized to determine the splice length of the vertical bars in the wall web instead of ACI 21.5.4, since ACI 21.5.4 assumes the bars are confined, which they are not in the wall web.

Again assuming that no more than 50% of the bars are spliced at any one location:

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 0.8 for No. 6 and smaller bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 0.75 + 0.625 + \dfrac{0.625}{2} = 1.7 \text{ in.} & \text{(governs)} \\ \\ \dfrac{12}{2} = 6.0 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index = 0

$$\frac{c + K_{tr}}{d_b} = \frac{1.7 + 0}{0.625} = 2.7 > 2.5, \quad \text{use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{0.8 \times 1.0 \times 1.0 \times 1.0}{2.5} = 22.8$$

$$\ell_d = 22.8 \times 0.625 = 14.3 \text{ in.}$$

Class B splice length = $1.3\ell_d$ = 1.3 × 14.3 = 18.6 in.

Use a 1 ft-7 in. splice length for the No. 5 vertical bars in the web.

Horizontal bars in web. The development length of the No. 5 horizontal bars is determined assuming that no hooks are used in the special boundary elements (ACI 21.6.6.4(e)). The provisions of ACI 21.5.4 are used in this case, since the horizontal bars are terminated in the confined core of the special boundary elements.

Since it is reasonable to assume that the depth of concrete cast in one lift beneath a horizontal bar is greater than 12 in., the required development length must not be less than 3.5 times the length required by ACI 21.5.4.1 (ACI 21.5.4.2):

- 8(diameter of bar) = $8 \times 0.625 = 5.0$ in.
- 6 in.
- $\dfrac{f_y d_b}{65\sqrt{f'_c}} = \dfrac{60{,}000 \times 0.625}{65\sqrt{4{,}000}} = 9.1$ in. (governs)

Therefore, required development length $\ell_d = 3.5 \times 9.1 = 31.9$ in.

This length can be accommodated within the confined core of the special boundary element so that hooks are not required. However, ACI R21.6.6.4 recommends to anchor the bars with standard 90-degree hooks or mechanical anchorage schemes instead of straight bar development, since large transverse cracks can our in the special boundary elements. Thus, 90-degree hooks are provided at the ends of the No. 5 horizontal bars.

Reinforcement details for the shear wall along line 7 are shown in Figure 2-25.

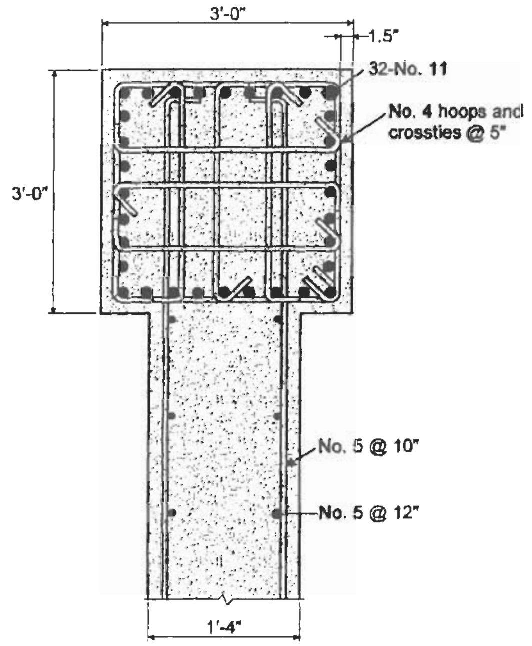


Figure 2-25 Reinforcement Details for Shear Wall Along Line 7 (SDC D)

2.5 DESIGN FOR SDC E

To illustrate the design requirements for Seismic Design Category (SDC) E, the office building in Figure 2-1 is assumed to be located in Berkeley, CA. In this example, additional shear walls are located along column lines 3 and 6 (see Figure 2-26). Typical beams, columns, and walls are designed and detailed for combined effects of gravity, wind, and seismic forces.

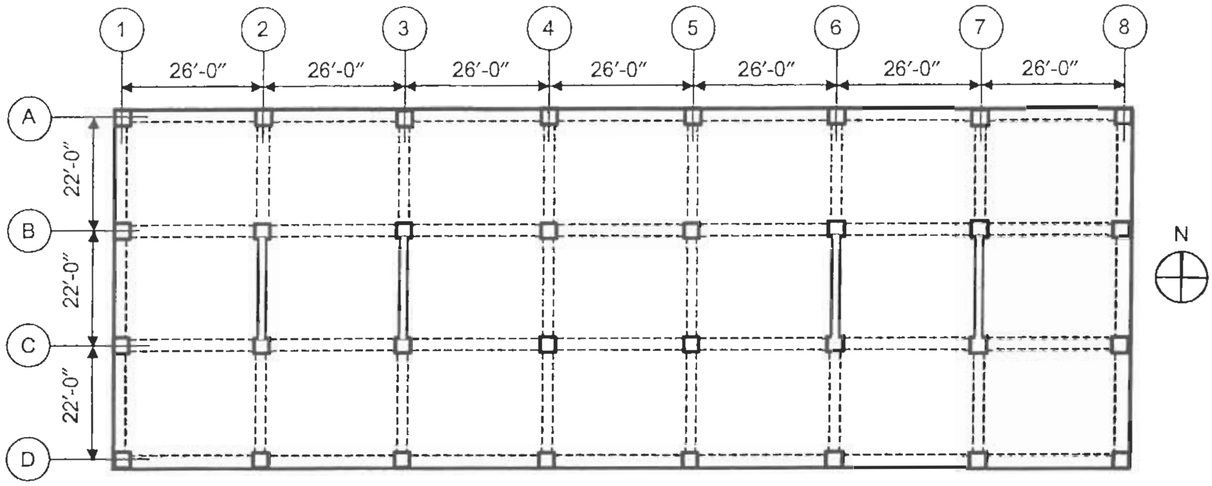


Figure 2-26 Typical Plan of Example Building (SDC E)

2.5.1 Design Data

- Building Location: Berkeley, CA (zip code 94705)
- Material Properties

  Concrete: $f'_c = 4{,}000$ psi, $w_c = 150$ pcf
  Reinforcement: $f_y = 60{,}000$ psi
- Service Loads

  Live loads: roof = 20 psf
  floor = 50 psf

  Superimposed dead loads: roof = 10 psf + 200 kips for penthouse
  floor = 30 psf (20 psf permanent partitions + 10 psf ceiling, etc.)
- Seismic Design Data

  For zip code 94705: $S_S = 2.08g, S_1 = 0.92g$ [2.1]
  Site Class D (stiff soil profile; IBC Table 1615.1.1)
  For Category (Seismic Use Group) I occupancy, $I_E = 1.0$ (IBC Table 1604.5)
- Wind Design Data

  Basic wind speed = 85 mph (IBC Figure 1609)

Exposure B (IBC 1609.4)
For Category I occupancy, $I_W = 1.0$ (IBC Table 1604.5)

- Member Dimensions

  Slab: 8 in.
  Beams: 28 × 32 in.
  Interior columns: 34 × 34 in.
  Edge columns: 30 × 30 in.
  Wall thickness: 18 in.
  Boundary elements: 40 × 40 in.

2.5.2 Seismic Load Analysis

2.5.2.1 Seismic Design Category (SDC)

As discussed previously, the appropriate analysis procedure to use depends on the Seismic Design Category (SDC). For SDC D, E, or F, Table 1616.6.3 contains the minimum allowable analysis procedure for seismic design (IBC 1616.6.3). The SDC for this particular example is determined below.

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1-second period S_{M1} are determined from IBC Eqs. 16-16 and 16-17, respectively:

$$S_{MS} = F_a S_s = 1.0 \times 2.08 = 2.08g$$

$$S_{M1} = F_v S_1 = 1.5 \times 0.92 = 1.38g$$

where F_a and F_v are the values of site coefficients as a function of site class and of mapped spectral response acceleration at short periods S_S and at 1-second period S_1, respectively. The values of these coefficients for Site Class D in the equations above are contained in IBC Table 1615.1.2(1) for F_a and Table 1615.1.2(2) for F_v.

Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from Eqs. 16-18 and 16-19:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 2.08 = 1.39g$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 1.38 = 0.92g$$

Tables 1616.3(1) and 1616.3(2) are utilized to determine the SDC. From the footnote to Table 1616.3(1), Seismic Use Group I structures located on sites with $S_1 \geq 0.75g$ shall be assigned to SDC E when $S_{DS} \geq 0.50g$. Similarly, from the footnote to Table 1616.3(2), the SDC is E when $S_{D1} \geq 0.20g$. Thus, the SDC is E for this building.

2.5.2.2 Seismic Forces

Since the building does not have plan irregularity Type 1a, 1b, or 4 of Table 1616.5.1 or vertical irregularity Type 1a, 1b, 4, or 5 of Table 1616.5.2, it can be considered regular (IBC 1616.6.3; note: it is shown in Section 2.5.2.5 below that the first story is not a soft story). Note that structures assigned to SDC E or F are not permitted to have plan irregularity Type 1b and vertical irregularity Type 1b or 5 (IBC 1620.4.1). For this regular building that is less than 240 ft in height, Table 1616.6.3 allows the equivalent lateral force procedure in IBC 1617.4 to be used to compute the seismic base shear V (see Eq. 16-34):

$$V = C_s W$$

where C_s is the seismic response coefficient determined in accordance with IBC 1617.4.1.1 and W is the effective weight of the structure defined in IBC 1617.4. For the member sizes and superimposed dead loads given above, W = 34,742 kips (see Table 2-31 below).

Seismic Forces in N-S Direction.

In the N-S direction, a dual system is utilized. In order to satisfy the provisions of IBC 1910.5.1 for structures assigned to SDC E, the dual system must have special reinforced concrete moment frames and special reinforced concrete shear walls. For this system, the response modification coefficient R = 8 and the deflection amplification factor $C_d = 6.5$ (IBC Table 1617.6).

Approximate period (T_a). The fundamental period T is determined in accordance with IBC 1617.4.2. In lieu of a more exact analysis, an approximate fundamental period T_a is computed from Eq. 16-39 for the dual system:

Building height $h_n = 148$ ft

Building period coefficient $C_T = 0.02$

Period $T_a = C_T (h_n)^{3/4} = 0.020 \times (148)^{3/4} = 0.85$ sec

For comparison purposes, the period was also determined using SAP2000 [2.4], which gave $T = 0.98$ sec. As in previous examples, no further refinement of the period is made.

Seismic base shear (V). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.92}{\left(\frac{8}{1.0}\right)\times 0.85} = 0.135$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{1.39}{\left(\frac{8}{1.0}\right)} = 0.174$$

Also, C_s must not be less than the value from Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 1.39 \times 1.0 = 0.061$$

For buildings assigned to SDC E or F and for those buildings for which $S_1 \geq 0.6g$, C_s shall not be taken less than that computed from Eq. 16-38:

$$C_s = \frac{0.5 S_1}{R / I_E} = \frac{0.5 \times 0.92}{8/1.0} = 0.058$$

In this case, the lower limit is 0.061 from Eq. 16-37.

The value of C_s from Eq. 16-36 governs; therefore, the base shear V in the N-S direction is:

$$V = C_s W = 0.135 \times 34{,}742 = 4{,}690 \text{ kips}$$

Vertical distribution of seismic forces. The total base shear is distributed over the height of the building in conformance with Eqs. 16-41 and 16-42:

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

where F_x is the lateral force induced at level x, w_x and w_i are the portions of W assigned to levels x or i, and k is the distribution exponent defined in IBC 1617.4.3. For $T = 0.85$ sec, $k = 1.17$ from linear interpolation. The lateral forces F_x and the story shears V_x are contained in Table 2-31.

Table 2-31 Seismic Forces and Story Shears in N-S Direction (SDC E)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
12	2,559	148	885,679	679	679
11	2,916	136	914,172	701	1,380
10	2,916	124	820,523	629	2,009
9	2,916	112	728,405	559	2,568
8	2,916	100	637,951	489	3,057
7	2,916	88	549,327	421	3,478
6	2,916	76	462,743	355	3,833
5	2,916	64	378,458	290	4,123
4	2,916	52	296,832	228	4,351
3	2,916	40	218,372	169	4,520
2	2,916	28	143,867	110	4,630
1	3,023	16	77,492	60	4,690
Σ	34,742		6,113,821		

Seismic Forces in E-W Direction.

In the E-W direction, a moment-resisting frame system is utilized. In order to satisfy the provisions of IBC 1910.5.1 for structures assigned to SDC E, this must be a special reinforced concrete moment frame. For this system, the response modification coefficient $R = 8$ and the deflection amplification factor $C_d = 5.5$ (IBC Table 1617.6).

Approximate period (T_a). Similar to the N-S direction, the fundamental period of the building is determined in accordance with Eq. 16-39 in IBC 1617.4.2:

Building period coefficient $C_T = 0.03$

Period $T_a = C_T (h_n)^{3/4} = 0.030 \times (148)^{3/4} = 1.27$ sec

The period obtained from SAP2000 is $T = 1.93$ sec.

Seismic base shear (V). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.92}{\left(\frac{8}{1.0}\right)\times 1.27} = 0.091$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{1.39}{\left(\frac{8}{1.0}\right)} = 0.174$$

Also, C_s must not be less than the larger value from Eq. 16-37 and 16-38:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 1.39 \times 1.0 = 0.061 \quad \text{(governs)}$$

$$C_s = \frac{0.5 S_1}{R / I_E} = \frac{0.5 \times 0.92}{8/1.0} = 0.058$$

Thus, Eq. 16-36 governs, and the base shear V in the E-W direction is:

$$V = C_s W = 0.091 \times 34{,}742 = 3{,}162 \text{ kips}$$

Vertical distribution of seismic forces. The lateral forces F_x and the story shears V_x are contained in Table 2-32 for seismic forces in the E-W direction. For $T = 1.27$ sec, $k = 1.39$ from linear interpolation.

2.5.2.3 Method of Analysis

A three-dimensional analysis of the building was performed in the N-S and E-W directions for the seismic forces contained in Tables 2-31 and 2-32 using SAP2000 [2.4]. In the model, rigid diaphragms were assigned at each floor level, and rigid-end offsets were defined at the ends of the horizontal members so that results were automatically obtained at the faces of the supports. The stiffness properties of the members were input assuming cracked sections. In lieu of a more accurate analysis, the following cracked section properties were used:

- Beams: $I_{eff} = 0.5 I_g$
- Columns: $I_{eff} = 0.7 I_g$
- Shear walls: $I_{eff} = 0.35 I_g$

where I_g is the gross moment of inertia of the section. P-delta effects were also considered in the analysis.

Table 2-32 Seismic Forces and Story Shears in E-W Direction (SDC E)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
12	2,559	148	2,659,096	501	501
11	2,916	136	2,694,056	507	1,008
10	2,916	124	2,369,429	446	1,454
9	2,916	112	2,056,840	387	1,841
8	2,916	100	1,757,064	331	2,172
7	2,916	88	1,471,020	277	2,449
6	2,916	76	1,199,826	226	2,675
5	2,916	64	944,882	178	2,853
4	2,916	52	707,998	133	2,986
3	2,916	40	491,644	93	3,079
2	2,916	28	299,459	56	3,135
1	3,023	16	142,615	27	3,162
Σ	34,742		16,793,929		

In accordance with IBC 1617.4.4.4, the center of mass was displaced each way from its actual location a distance equal to 5 percent of the building dimension perpendicular to the applied forces to account for accidental torsion in seismic design. Torsional effects need not be amplified, since the building does not possess Type 1a or 1b plan torsional irregularity as defined in Table 1616.5.1 (IBC 1617.4.4.5).

In a dual system, an additional safeguard is provided by requiring that moment-resisting frames be capable of resisting at least 25% of the design forces without the benefit of shear walls (IBC 1617.6.1). Thus, the building was also analyzed in the N-S direction using 25% of the design forces in Table 2-31 without the shear walls present, including torsional effects.

As was discussed in Section 2.4.2.3 of this publication, changes in IBC 1620.2.2 regarding orthogonal effects and when such effects need to be considered appeared in the supplement to the IBC [2.7]. In the following sections, designs based on when orthogonal effects are considered and when they are not considered are compared for illustrative purposes.

2.5.2.4 Story Drift and P-delta Effects

Story drift determination. Table 2-33 contains the displacements δ_{xe} obtained from the 3-D static, elastic analyses using the design seismic forces in the N-S and E-W directions, including accidental torsional effects. The table also contains the design earthquake displacement δ_x computed by IBC Eq. 16-46:

$$\delta_x = \frac{C_d \delta_{xe}}{I_E}$$

where C_d is the deflection amplification factor in Table 1617.6, which depends on the seismic-force-resisting system. As noted above, C_d is equal to 6.5 for the dual system with special reinforced concrete moment frames and special reinforced concrete shear walls in the N-S direction and is 5.5 for the special reinforced concrete moment frames in the E-W direction.

Table 2-33 Lateral Displacements and Interstory Drifts due to Seismic Forces in N-S and E-W Directions (SDC E)

Story	N-S Direction			E-W Direction		
	δ_{xe} (in.)	δ_x (in.)	Δ (in.)	δ_{xe} (in.)	δ_x (in.)	Δ (in.)
12	2.78	18.07	1.36	4.56	25.08	0.60
11	2.57	16.71	1.56	4.45	24.48	0.99
10	2.33	15.15	1.56	4.27	23.49	1.32
9	2.09	13.59	1.69	4.03	22.17	1.71
8	1.83	11.90	1.76	3.72	20.46	1.98
7	1.56	10.14	1.75	3.36	18.48	2.20
6	1.29	8.39	1.69	2.96	16.28	2.42
5	1.03	6.70	1.70	2.52	13.86	2.64
4	0.77	5.00	1.55	2.04	11.22	2.64
3	0.53	3.45	1.37	1.56	8.58	2.75
2	0.32	2.08	1.10	1.06	5.83	2.75
1	0.15	0.98	0.98	0.56	3.08	3.08

The interstory drifts Δ computed from the δ_x are also contained in the table. For this structure that does not have plan irregularity Type 1a or 1b of Table 1616.5.1, the drift at story level x is determined by subtracting the design earthquake displacement at the center of mass at the bottom of the story from the design earthquake displacement at the center of mass at the top of the story (IBC 1617.4.6.1):

$$\Delta = \delta_x - \delta_{x-1}$$

The design story drifts Δ must not exceed the allowable story drift Δ_a from Table 1617.3 (IBC 1617.3). For Seismic Use Group I, $\Delta_a = 0.020h_{sx}$ where h_{sx} is the story height below level x. Thus, for the 12-ft story heights, $\Delta_a = 0.020 \times 12 \times 12 =$ 2.88 in., and for the 16-ft story height at the first level, $\Delta_a = 3.84$ in. It is evident from Table 2-33 that for all stories, the lateral drifts obtained from the prescribed lateral forces in both directions are less than the limiting values.

P-delta effects. As noted above, P-delta effects were automatically considered in the analysis using SAP2000. However, for illustration purposes, the following procedure can be used to determine whether P-delta effects need to be considered or not in accordance with IBC 1617.4.6.2.

P-delta effects need not be considered when the stability coefficient θ determined by Eq. 16-47 is less than or equal to 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d}$$

where P_x = total unfactored vertical design load at and above level x
Δ = design story drift occurring simultaneously with V_x
V_x = seismic shear force acting between level x and $x - 1$
h_{sx} = story height below level x
C_d = deflection amplification factor

The stability coefficient θ must not exceed θ_{max} determined from Eq. 16-48:

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25$$

where β is the ratio of shear demand to shear capacity between level x and $x - 1$, which may be taken equal to 1.0 when it is not calculated.

Table 2-34 contains the calculations for the N-S and E-W directions. It is clear that in both directions, P-delta effects need not be considered at any of the floor levels. Note that θ_{max} is equal to 0.077 and 0.091 in the N-S and E-W directions, respectively, using β = 1.0.

2.5.2.5 Soft Story

The following procedure, which was introduced in Section 2.4.2.5 of this publication, can be used to determine whether the first story is a soft story or not. Using the displacement-based method presented in Example 4 in Reference 2.5, a soft story exists in the first floor level when one of the following conditions is satisfied:

$$0.70\left(\frac{\delta_{e1}}{h_1}\right) > \frac{\delta_{e2} - \delta_{e1}}{h_2}$$

$$0.80\left(\frac{\delta_{e1}}{h_1}\right) > \frac{1}{3}\left(\frac{\delta_{e2} - \delta_{e1}}{h_2} + \frac{\delta_{e3} - \delta_{e2}}{h_3} + \frac{\delta_{e4} - \delta_{e3}}{h_4}\right)$$

where δ_{ei} are the elastic displacements at level i due to code-prescribed lateral forces and h_i are the story heights below level i.

Table 2-34 P-delta Effects (SDC E)

Level	h_{sx} (ft)	P_x (kips)	N-S Direction			E-W Direction		
			V_x (kips)	Δ (in.)	θ	V_x (kips)	Δ (in.)	θ
12	12	2,812	679	1.36	0.006	501	0.60	0.004
11	12	5,981	1,380	1.56	0.007	1,008	0.99	0.007
10	12	9,149	2,009	1.56	0.008	1,454	1.32	0.011
9	12	12,318	2,568	1.69	0.009	1,841	1.71	0.015
8	12	15,487	3,057	1.76	0.009	2,172	1.98	0.018
7	12	18,656	3,478	1.75	0.010	2,449	2.20	0.021
6	12	21,824	3,833	1.69	0.010	2,675	2.42	0.025
5	12	24,993	4,123	1.70	0.011	2,853	2.64	0.029
4	12	28,162	4,351	1.55	0.011	2,986	2.64	0.032
3	12	31,331	4,520	1.37	0.010	3,079	2.75	0.036
2	12	34,499	4,630	1.10	0.009	3,135	2.75	0.038
1	16	37,775	4,690	0.98	0.006	3,162	3.08	0.035

Using the data in **Table 2-33, check if a** soft story exists in the first story based on displacements in the E-W direction:

$$0.70\left(\frac{\delta_{e1}}{h_1}\right) = 0.70\left(\frac{0.56}{16\times 12}\right) = 0.00204 < \frac{\delta_{e2} - \delta_{e1}}{h_2} = \frac{1.06 - 0.56}{12\times 12} = 0.00347 \quad \text{OK}$$

$$0.80\left(\frac{\delta_{e1}}{h_1}\right) = 0.00233 < \frac{1}{3}\left(\frac{1.06 - 0.56}{12\times 12} + \frac{1.56 - 1.06}{12\times 12} + \frac{2.04 - 1.56}{12\times 12}\right) = 0.00343 \quad \text{OK}$$

Similar calculations with displacements in the N-S direction also show that a soft story does not exist in the first story of this example building.

2.5.3 Wind Load Analysis

Details on how to compute the wind forces in both the N-S and E-W directions are given in Section 2.2.3 of this publication. In this example, the wind velocity is 85 mph, which is the same wind velocity for the example building in Section 2.4 located in San Francisco. A summary of the design wind forces in both directions at all floor levels is contained in Table 2-35. Once again it is important to note that the wind directionality factor K_d has been taken equal to 1.0 (see Exception 1 in IBC 1605.2.1).

Similar to the seismic analysis, a three-dimensional analysis of the building was performed in both directions for the wind forces contained in Tables 2-35 using SAP2000. The modeling assumptions utilized for the seismic analysis were also used for the wind analysis. Also considered in the three-dimensional analysis were the full and partial wind load cases of Figure 6-9 (Cases 1 through 4) given in ASCE 6.5.12.3.

Table 2-35 Design Wind Forces in N-S and E-W Directions (V = 85 mph)

Level	Height above ground level, z (ft)	Total Design Wind Force N-S (kips)	Total Design Wind Force E-W (kips)
12	148	25.0	8.6
11	136	49.2	16.9
10	124	48.4	16.6
9	112	47.6	16.2
8	100	46.7	15.9
7	88	45.7	15.4
6	76	44.6	15.0
5	64	43.4	14.5
4	52	42.0	13.9
3	40	40.4	13.2
2	28	38.3	12.3
1	16	41.4	13.0
	Σ	512.7	171.5

2.5.4 Design for Combined Load Effects

2.5.4.1 Load Combinations

Basic load combinations for strength design are given in IBC 1605.2.1. As noted above, the first exception in this section requires that the non-seismic load combinations of ACI 9.2 be used for concrete structures. Thus, the following load combinations are utilized in the design of the structural members:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)
4. $1.2D + f_1 L + 1.0E$ (Formula 16-5)
5. $0.9D + 1.0E$ (Formula 16-6)

The seismic load effect E for use in Formula 16-5, which is the combined effect of horizontal and vertical earthquake-induced forces, is computed from Eq. 16-28 where the effects of gravity and seismic ground motion are additive:

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of horizontal seismic forces

ρ = redundancy coefficient determined in accordance with IBC 1617.2.2 for SDC D, E, or F

Similarly, E for use in Formula 16-6 is computed from Eq. 16-29 where the effects of gravity and seismic ground motion counteract:

$$E = \rho Q_E - 0.2 S_{DS} D$$

According to IBC 1617.2.2, the redundancy coefficient ρ, which shall not be less than 1.0 and need not exceed 1.5, is the largest of the values of ρ_i calculated at each story i from Equation 16-32:

$$\rho_i = 2 - \frac{20}{r_{\max_i} \sqrt{A_i}}$$

For the dual system in the N-S direction, the shear force in the most heavily loaded shear wall in the first story is 1,460 kips and the story shear is 4,690 kips. Therefore,

$$r_{\max_1} = \frac{\text{Shear force in wall}}{\text{Story shear}} \times \frac{10}{\ell_w} = \frac{1{,}460}{4{,}690} \times \frac{10}{25.33} = 0.123$$

$$\rho_1 = \rho = 2 - \frac{20}{0.123\sqrt{184.5 \times 68.5}} = 0.55$$

For dual systems, ρ need not exceed 0.8 × 0.55 = 0.44 < 1.0. Use ρ = 1.0 (IBC 1617.2.2).

Computations for the redundancy coefficient in the E-W direction for the moment-resisting frame along column line B (most heavily loaded frame when center of mass is displaced north of actual location) are summarized in Table 2-36. Since the shear force distribution in the columns is symmetric, results are presented for columns on lines 1 through 4 only. Subscripts on column shear forces refer to column lines. To illustrate the computations, $r_{\max_i}$ is calculated for the first story as follows:

$$r_{\max_1} = \frac{\text{maximum sum of shears in 2 adjacent columns}}{\text{total story shear}} = \frac{180.0}{3{,}162} = 0.057$$

From Table 2-36, maximum $r_{\max_i} = 0.067$; thus, ρ is:

$$\rho = 2 - \frac{20}{0.067\sqrt{184.5 \times 68.5}} < 0$$

Therefore, $\rho = 1.0$. For SDC E, IBC 1617.2.2 requires that ρ be less than or equal to 1.1 for special moment frames, which is satisfied in this case.

Table 2-36 Redundancy Coefficient Calculations in E-W Direction (SDC E)*

Story	Story Shear	Column Shear Forces				$V_1 + 0.7V_2$	$0.7(V_2 + V_3)$	$0.7(V_3 + V_4)$	$r_{\max,i}$
		V_1	V_2	V_3	V_4				
12	501	2.9	15.6	23.5	24.7	13.8	27.4	33.7	0.067
11	1,008	16.1	36.3	41.4	42.4	41.5	54.4	58.7	0.058
10	1,454	24.5	52.3	58.1	60.2	61.1	77.3	82.8	0.057
9	1,841	31.7	66.6	72.2	74.9	78.3	97.2	103.0	0.056
8	2,172	38.1	78.8	84.3	87.6	93.3	114.2	120.3	0.055
7	2,449	43.6	89.0	94.2	98.1	105.9	128.2	134.6	0.055
6	2,675	48.1	97.4	102.3	106.6	116.3	139.8	146.2	0.055
5	2,853	51.9	104.0	108.5	113.2	124.7	148.8	155.2	0.054
4	2,986	54.6	109.1	113.0	117.8	131.0	155.5	161.6	0.054
3	3,079	58.1	112.7	115.9	121.3	137.0	160.0	166.0	0.054
2	3,135	55.9	113.7	116.1	121.0	135.5	160.9	166.0	0.053
1	3,162	72.7	126.5	127.0	130.1	161.3	177.5	180.0	0.057

*Shear forces are in kips.

Once ρ has been computed in both orthogonal directions, load combinations 4 and 5 can be rewritten as follows. Substituting $S_{DS} = 1.39g$ and $\rho = 1.0$ into the equations for E, and then substituting E along with $f_1 = 0.5$ into load combinations 4 and 5 above results in the following:

4a. $1.2D + 0.5L + 1.0Q_E + (0.2 \times 1.39)D = 1.48D + 0.5L + Q_E$

4b. $1.2D + 0.5L + 1.0Q_E - (0.2 \times 1.39)D = 0.92D + 0.5L + Q_E$

5a. $0.9D + 1.0Q_E + (0.2 \times 1.39)D = 1.18D + Q_E$

5b. $0.9D + 1.0Q_E - (0.2 \times 1.39)D = 0.62D + Q_E$

If it had turned out that ρ for the dual system was different than ρ for the moment-resisting system, then two different sets of Eqs. 4a through 5b would be determined, one for each orthogonal direction.

Values of Q_E are obtained from the structural analysis for both sidesway to the left and to the right based on the code-prescribed horizontal seismic forces acting on the building. Also, the special seismic load combinations in IBC 1605.4 are not applicable in this example.

2.5.4.2 Design of Beam C4-C5

Flexural design.

Comparing the seismic forces in Table 2-32 to the wind forces in Table 2-35, it is clear that seismic effects will govern the design in the E-W direction in this example. For this beam, orthogonal effects due to seismic forces are negligible, i.e., seismic forces in the N-S direction have a negligible effect on the design forces of this member. According to the 2002 supplement to the IBC [2.7], orthogonal effects need not be considered for beams. Thus, Table 2-37 contains a summary of the governing design bending moments and shear forces for beam C4-C5 at the second floor level due to gravity and seismic forces in the E-W direction only.

Requirements for special moment frames are given in ACI 21.2 through 21.5. The factored axial load on the member, which is negligible, is less than $A_g f_c'/10$; thus, the provisions of ACI 21.3 for flexural members of special moment frames must be satisfied. All other applicable provisions in Chapters 1 through 18 are to be satisfied as well.

Check limitations on section dimensions per ACI 21.3.1:

- Factored axial compressive force on member is negligible. O.K.
- $\dfrac{\ell_n}{d} = \dfrac{(26 \times 12) - 34}{29.5} = 9.4 > 4$ O.K.
- $\dfrac{\text{width}}{\text{depth}} = \dfrac{28}{32} = 0.9 > 0.3$ O.K.
- width = 28 in. > 10 in. O.K.
  < width of supporting column + (1.5 × depth of beam)
  < 34 + (1.5 × 32) = 82 in. O.K.

The required flexural reinforcement is contained in Table 2-38. The provided areas of steel are within the limits prescribed in ACI 21.3.2.1 for maximum and minimum reinforcement. The selected reinforcement satisfies ACI 7.6.1 and 3.3.2 (minimum spacing for concrete placement), ACI 7.7.1 (minimum cover for protection of reinforcement), and ACI 10.6 (maximum spacing for control of flexural cracking).

ACI 21.3.2.2 requires that the positive moment strength at the face of the joint be greater than or equal to 50% of the negative moment strength at that location. This is satisfied, since 597 ft-kips > 860/2 = 430 ft-kips. Also, the negative or positive moment strength at any section along the length of the member must be greater than or equal to 25% of the maximum moment strength provided at the face of either joint. In this case, 25% of the maximum moment strength is equal to 860/4 = 215 ft-kips. Providing 2-No. 9 bars (ϕM_n = 260 ft-kips) or 3-No. 8 bars (ϕM_n = 307 ft-kips) satisfies this provision. However, to satisfy the minimum reinforcement requirement of ACI 21.3.2.2 (i.e., minimum A_S = 2.75 in.^{2}), a minimum of 3-No. 9 bars (A_S = 3.00 in.^{2}) or 4-No. 8 bars (A_S = 3.16 in.^{2}) must be provided at any section. This also automatically satisfies the requirement that at least 2 bars be continuous at both the top and bottom of the section (ACI 21.3.2.1).

Table 2-37 Summary of Design Bending Moments and Shear Forces for Beam C4-C5 at the 2nd Floor Level (SDC E)

Load Case	Location	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	Support	-117	30
	Midspan	81	
Live (L)	Support	-16	8
	Midspan	11	
Seismic (Q_E)	Support	± 645	± 56
Load Combinations			
$1.4D + 1.7L$	Support	-191	56
	Midspan	132	
$1.48D + 0.5L + Q_E$	Support	-826	104
	Midspan	125	
$0.62D + Q_E$	Support	573	-37
	Midspan	50	

Table 2-38 Required Flexural Reinforcement for Beam C4-C5 at the 2nd Floor Level (SDC E)

Location	M_u (ft-kips)	A_s^* (in.^{2})	Reinforcement*	ϕM_n (ft-kips)
Support	-826	6.70	7-No. 9	860
	573	4.53	6-No. 8	597
Midspan	132	2.75	6-No. 8	597

$$^*A_{s,min} = \frac{3\sqrt{f'_c}\,b_w d}{f_y} = \frac{3\sqrt{4{,}000} \times 28 \times 29.5}{60{,}000} = 2.61 \text{ in.}^2$$

ACI 21.3.2.1

$$= \frac{200 b_w d}{f_y} = \frac{200 \times 28 \times 29.5}{60{,}000} = 2.75 \text{ in.}^2 \text{ (governs)}$$

$$A_{s,max} = \rho_{max} b_w d = 0.025 \times 28 \times 29.5 = 20.7 \text{ in.}^2$$

ACI 21.3.2.1

When reinforcing bars extend through a joint, the column dimension parallel to the beam reinforcement must be at least 20 times the diameter of the largest longitudinal bar for normal weight concrete (ACI 21.5.1.4). In this case, the minimum required column dimension = 20 × 1.128 = 22.6 in., which is less than the 34-in. column width that is provided.

Shear design.

Shear requirements for beams in special moment frames are contained in ACI 21.3.4. The method of determining design shear forces in beams in special moment frames takes into consideration the likelihood of yielding (i.e., plastic hinges forming) at regions near the supports. In general, the shear forces are determined assuming simultaneous hinging at the beam supports under lateral loading. To properly confine the concrete and to maintain lateral support of the longitudinal bars in regions where yielding is expected, the transverse reinforcement requirements of ACI 21.3.3 must also be satisfied.

According to ACI 21.3.4.1, shear forces are computed from statics assuming that moments of opposite sign corresponding to the probable moment strength M_{pr} act at the joint faces and that the member is loaded with tributary factored gravity load along its span. Sidesway to the right and to the left must be considered when calculating the maximum design shear forces.

The probable moment strength M_{pr} for a section is determined using the stress in the tensile reinforcement equal to $1.25 f_y$ and a strength reduction factor ϕ equal to 1.0 (ACI 21.0). The following equation can be used to compute M_{pr}:

$$M_{pr} = A_s(1.25 f_y)\left(d - \frac{a}{2}\right)$$

where $a = \dfrac{A_s(1.25 f_y)}{0.85 f'_c b}$

For example, for sidesway to the right, the joint on column line 5 is subjected to the negative moment M_{pr} that is determined as follows:

For 7-No. 9 top bars, $A_s = 7 \times 1.00 = 7.00$ in.^{2}

$$a = \frac{7.00 \times 1.25 \times 60}{0.85 \times 4 \times 28} = 5.51 \text{ in.}$$

$$M_{pr} = 7.00 \times 1.25 \times 60 \times \left(29.5 - \frac{5.51}{2}\right) = 14{,}041 \text{ in.-kips} = 1{,}170 \text{ ft-kips}$$

Similarly, for the joint on column line 4, the positive moment M_{pr} based on 6-No. 8 bars is equal to 819 ft-kips.

The largest shear force associated with seismic effects is obtained from the second of the three load combinations in Table 2-37. Figure 2-27 shows the beam and shear forces due to factored gravity loads plus probable moment strengths for sidesway to the right. Due to the symmetric distribution of longitudinal reinforcement in the beam, sidesway to the left gives the same maximum shear force. The equivalent factored uniform loads on the beam are determined as follows (see ACI Figure R13.6.8 for definition of tributary area on beam):

$$\text{Total trapezoidal area tributary to beam} = 2\left\{2\left[\frac{1}{2}\times 11\times 11\right]+(4\times 11)\right\} = 330\text{ ft}^2$$

$$\text{Dead load} = \left(\frac{8}{12}\times 0.15\times 330\right)+\left(\frac{28\times 24}{144}\times 0.15\times 23.17\right)+\left(\frac{30\times 330}{1{,}000}\right) = 59.1\text{ kips}$$

$$w_D = \frac{59.1}{23.17} = 2.6\text{ kips/ft}$$

$$\text{Live load} = \frac{50\times 330}{1{,}000} = 16.5\text{ kips}$$

$$w_L = \frac{16.5}{23.17} = 0.7\text{ kips/ft}$$

$$w_u = 1.48 w_D + 0.5 w_L = (1.48\times 2.6)+(0.5\times 0.7) = 4.2\text{ kips/ft}$$

The maximum combined shear force of 134.5 kips shown in Figure 2-27 is larger than the maximum shear force obtained from the structural analysis, which is 104 kips (see Table 2-37).

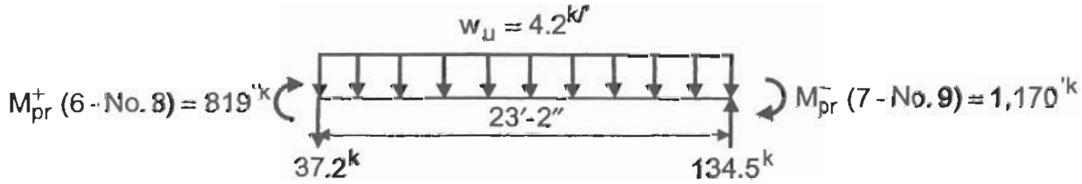


Figure 2-27 Design Shear Forces for Beam C4-C5 (SDC E)

In general, shear strength is provided by both concrete (V_c) and reinforcing steel (V_s). However, according to ACI 21.3.4.2, V_c is to be taken as zero when the earthquake-induced shear force calculated in accordance with ACI 21.3.4.1 is greater than or equal to 50% of the total shear force and the factored axial compressive force including earthquake effects is less than $A_g f_c'/20$. In this example, the beam carries negligible axial forces and the maximum earthquake-induced shear force = (1,170 + 819)/23.17 = 85.8 kips > 134.5/2 = 67.3 kips. Thus, V_c must be taken equal to zero.

The maximum shear force V_s is (ACI 11.1):

$$V_s = \frac{V_u}{\phi} - V_c = \frac{134.5}{0.85} - 0 = 158.2 \text{ kips}$$

where the strength reduction factor was taken as 0.85 in accordance with ACI 9.3.4.

Shear strength contributed by shear reinforcement shall not exceed $(V_s)_{max}$ (ACI 11.5.6.9):

$$(V_s)_{max} = 8\sqrt{f_c'}b_w d = 8\sqrt{4{,}000} \times 28 \times 29.5/1{,}000 = 417.9 \text{ kips} > 158.2 \text{ kips O.K.}$$

Also, V_s is less than $4\sqrt{f_c'}b_w d = 209.0$ kips.

Required spacing s of No. 4 closed stirrups (hoops) for a factored shear force of 158.2 kips is determined from Eq. (11-15):

$$s = \frac{A_v f_y d}{V_s} = \frac{(4 \times 0.2) \times 60 \times 29.5}{158.2} = 9.0 \text{ in.}$$

Note that 4 legs are required for support of the longitudinal bars (ACI 21.3.3.3).

Maximum allowable hoop spacing within a distance of $2h = 2 \times 32 = 64$ in. (plastic hinge length) from the face of the support at each end of the member is the smaller of the following (ACI 21.3.3.2):

- $d/4 = 29.5/4 = 7.4$ in. (governs)
- 8(diameter of smallest longitudinal bar) = $8 \times 1.0 = 8.0$ in.
- 24(diameter of hoop bar) = $24 \times 0.5 = 12.0$ in.
- 12 in.

Use 10-No. 4 hoops at each end of the beam spaced at 7 in. on center with the first hoop located 2 in. from the face of the support (ACI 21.3.3.2).

Where hoops are no longer required, stirrups with seismic hooks at both ends may be used (ACI 21.3.3.4). At a distance of 65 in. from the face of the support:

$$V_u = 134.5 - [4.2 \times (65/12)] = 111.8 \text{ kips}$$

Also, the shear strength contributed by the concrete may be utilized outside of the potential plastic hinge zones:

$$V_c = 2\sqrt{4{,}000} \times 28 \times 29.5/1{,}000 = 104.5 \text{ kips}$$

Therefore, the required stirrup spacing for No. 4 stirrups is:

$$s = \frac{A_v f_y d}{V_s} = \frac{(2 \times 0.2) \times 60 \times 29.5}{(111.8/0.85) - 104.5} = 26.2 \text{ in.}$$

$$= \frac{A_v f_y}{50 b_w} = \frac{(2 \times 0.2) \times 60{,}000}{50 \times 28} = 17.1 \text{ in.} \quad \text{(governs)}$$

Confinement of the longitudinal bars in accordance with ACI 21.3.3.3 must be provided only where hoops are required; that is why only 2 legs are used in this portion of the beam where stirrups with seismic hooks are required.

The maximum allowable spacing of the stirrups is $d/2$ = 14.8 in. (ACI 21.3.3.4), which is less than that required for minimum shear reinforcement. A 14 in. spacing, starting at 65 in. from the face of the support is sufficient for the remaining portion of the beam.

Reinforcing bar cutoff points.

The negative reinforcement at the supports is 7-No. 9 bars. The location where 3 of the 7 bars can be terminated will be determined.

The third load combination is used to determine the cutoff point of the 3-No. 9 bars (0.62 times the dead load in combination with the probable flexural strengths M_{pr} at the ends of the member), since this combination produces the longest bar lengths. The design flexural strength ϕM_n provided by 4-No. 9 bars is 508 ft-kips. Therefore, the 3-No. 9 bars can be terminated after the required moment strength M_u has been reduced to 508 ft-kips.

The distance x from the support to the location where the moment is equal to 508 ft-kips can readily be determined by summing moments about section *a-a* in Figure 2-28:

$$\frac{1.6x^2}{2} - 104.4x + 1{,}170 = 508$$

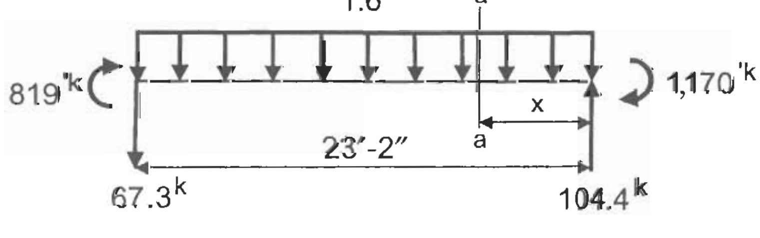


Figure 2-28 Cutoff Location of Negative Bars (SDC E)

Solving for x gives a distance of 6.7 ft from the face of the support.

The 3-No. 9 bars must extend a distance d = 29.5 in. (governs) or $12d_b = 12\times 1.128 = 13.5$ in. beyond the distance x (ACI 12.10.3). Thus, from the face of the support, the total bar length must be at least equal to 6.7 + (29.5/12) = 9.2 ft. Also, the bars must extend a full development length ℓ_d beyond the face of the support (ACI 12.10.4), which is determined from ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.3 for top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 9 bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5 + 0.5 + \dfrac{1.128}{2} = 2.6 \text{ in.} \\ \\ \dfrac{28 - 2(1.5+0.5) - 1.128}{2\times 6} = 1.9 \text{ in.} \quad \text{(governs)} \end{cases}$$

K_{tr} = transverse reinforcement index = 0 (conservative)

$$\frac{c+K_{tr}}{d_b} = \frac{1.9+0}{1.128} = 1.7 < 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40}\times\frac{60{,}000}{\sqrt{4{,}000}}\times\frac{1.3\times 1.0\times 1.0\times 1.0}{1.7} = 54.4$$

$$\ell_d = 54.4\times 1.128 = 61.4 \text{ in.} = 5.1 \text{ ft} < 9.2 \text{ ft}$$

Thus, the total required length of the 3-No. 9 bars must be at least 9.2 ft beyond the face of the support.

Flexural reinforcement shall not be terminated in a tension zone unless one or more of the conditions of ACI 12.10.5 are satisfied. In this case, the point of inflection is approximately 12.4 ft from the face of the support, which is greater than 9.2 ft. Thus, the No. 9 bars cannot be terminated here unless one of the conditions of ACI 12.10.5 is satisfied. In this case, check if the factored shear force V_u at the cutoff point does not exceed $2\phi V_n/3$ (ACI 12.10.5.1). With No. 4 stirrups at 14 in. on center that are provided in this region of the beam, ϕV_n is determined by ACI Eqs. 11-1 and 11-2:

$$\phi V_n = \phi(V_c + V_s) = 0.85 \times \left(104.5 + \frac{0.40 \times 60 \times 29.5}{14}\right) = 131.8 \text{ kips}$$

$$\frac{2}{3}\phi V_n = 87.9 \text{ kips}$$

At 9.2 ft from the face of the support, $V_u = 104.4 - (1.6 \times 9.2) = 89.7$ kips, which is greater than 87.9 kips. Therefore, the 3-No. 9 bars cannot be terminated at this location. However, by decreasing the stirrup spacing to 12 in., ϕV_n = 139.0 kips and $2\phi V_n/3$ = 92.7 kips, which is greater than V_u = 89.7 kips. Therefore, use 12 in. spacing for the stirrups in the regions of the beam outside the plastic hinge zones; this allows the No. 9 top bars to be cut off at 9.2 ft from the face of the support.

Flexural reinforcement splices.

According to ACI 21.3.2.3, lap splices of flexural reinforcement must not be placed within a joint, within a distance $2h$ from the face of the joint (plastic hinge region), or at locations where analysis indicates flexural yielding due to inelastic lateral displacements of the frame. Lap splices must be confined by hoops or spiral reinforcement along the entire lap length, and the maximum spacing of the transverse reinforcement is $d/4$ or 4 in. In lieu of lap splices, mechanical and welded splices conforming to ACI 21.2.6 and 21.2.7, respectively, may be used (ACI 21.3.2.4).

Lap splices are determined for the No. 8 bottom bars. Since all of the bars are to be spliced within the required length, a Class B splice must be used (ACI 12.15.1, 12.15.2):

Class B splice length = $1.3\ell_d$

The development length ℓ_d is determined from ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 8 bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5 + 0.5 + \dfrac{1.0}{2} = 2.5 \text{ in.} \\ \dfrac{28 - 2(1.5 + 0.5) - 1.0}{2 \times 5} = 2.3 \text{ in.} \quad \text{(governs)} \end{cases}$$

K_{tr} = transverse reinforcement index = 0 (conservative)

$$\frac{c + K_{tr}}{d_b} = \frac{2.3 + 0}{1.0} = 2.3 < 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.0 \times 1.0 \times 1.0 \times 1.0}{2.3} = 30.9$$

$$\ell_d = 30.9 \times 1.0 = 30.9 \text{ in.} = 2.6 \text{ ft}$$

Class B splice length = $1.3\,\ell_d = 1.3 \times 2.6 = 3.4$ ft

Use a 3 ft-6 in. splice length.

Figure 2-29 shows the reinforcement details for beam C4-C5.

2.5.4.3 Design of Column C4

Table 2-39 contains a summary of the design axial forces, bending moments, and shear forces on column C4 for gravity and seismic loads.

According to the supplement to the IBC [2.7], orthogonal effects on this member need not be considered, since the axial forces due to seismic forces in both directions (which are equal to zero) are less than 20% of the axial load design strength. For illustration purposes, the column is designed when orthogonal effects are not considered and the results are compared to those where orthogonal effects are considered. Bending moments and shear forces are contained in Table 2-39 for both the N-S and E-W directions. In the N-S direction, the governing bending moment and shear force are due to 25% of the seismic forces F_x from Table 2-31 acting on the building without the presence of shear walls, which is one of the requirements that is stipulated for dual systems (IBC 1617.6.1).

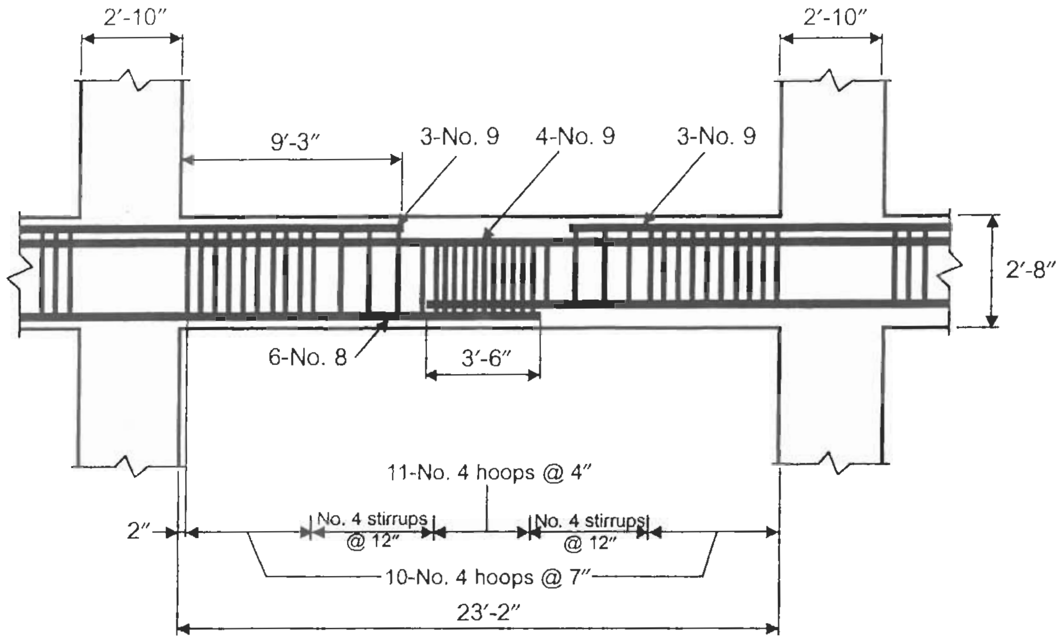


Figure 2-29 Reinforcement Details for Beam C4-C5 at the 2nd Floor Level (SDC E)

Table 2-39 Summary of Design Axial Forces, Bending Moments, and Shear Forces on Column C4 between the 1st and 2nd Floor Levels (SDC E)

Load Case		**Axial Force (kips)**	**Bending Moment (ft-kips)**		**Shear Force (kips)**	
			N-S	**E-W**	**N-S**	**E-W**
Dead (D)		1,270	0	0	0	0
Live (L)*		134	0	0	0	0
Seismic (Q_E)		0**	± 251	± 604	± 54	± 129
Load Combinations						
$1.4D + 1.7L$		2,006	0	0	0	0
$1.48D + 0.5L + Q_E$	Case 1	1,947	251	181	54	39
	Case 2	1,947	75	604	16	129
$0.62D + Q_E$	Case 1	787	-251	-181	-54	-39
	Case 2	787	-75	-604	-16	-129

* Live load reduced per IBC 1607.9
** Axial forces due to seismic load effects in both the N-S and E-W directions are zero
Case 1: 100% N-S seismic force effect + 30% E-W seismic force effect
Case 2: 100% E-W seismic force effect + 30% N-S seismic force effect

As was done in Section 2.4.4.3 of this publication, orthogonal load effects are combined in accordance with the first method given in IBC 1620.2.2: 100% of the forces acting in one direction plus 30% of the forces acting in the perpendicular direction. Bending moments and shear forces under Cases 1 and 2 in Table 2-39 have been determined using this method.

Since the factored compressive axial loads exceed $A_g f_c'/10 = 34^2 \times 4/10 =$ 462 kips, the provisions of ACI 21.4 are applicable. Thus, the following two criteria must be satisfied (ACI 21.4.1):

- Shortest cross-sectional dimension = 34 in. > 12 in. O.K.
- Ratio of shortest cross-sectional dimension to perpendicular dimension = 1.0 > 0.4 O.K.

Design for axial force and bending.

When orthogonal effects are not considered, the governing load combinations are due to seismic forces acting the E-W direction, since bending moments generated in the column due to seismic forces in that direction are larger than those generated from seismic forces in the N-S direction. A 34 × 34 in. column with 12-No. 10 bars ($\rho_g = 1.32\%$) is adequate for column C4 supporting the second floor level. Figure 2-30 contains the interaction diagram for this column for uniaxial bending due to seismic forces in the E-W direction.

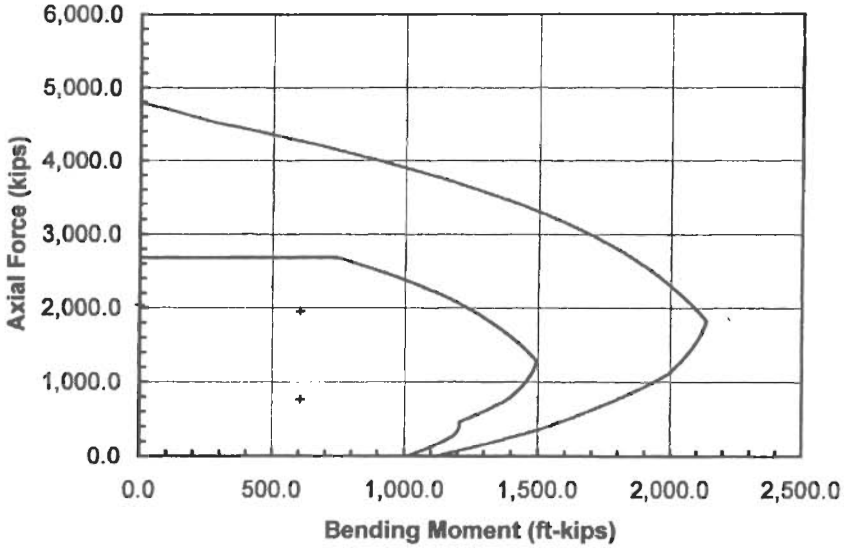


Figure 2-30 Design and Nominal Strength Interaction Diagrams for Column C4 Supporting the 2nd Floor Level when Orthogonal Effects are not Considered (SDC E)

When orthogonal effects are considered, the column must be designed for biaxial bending due to the load combinations given in Table 2-39 for seismic load cases 1 and 2 as prescribed in IBC 1620.2.2. For the column reinforced with 12-No. 10 bars, interaction diagrams cut at an angle of 0 degrees (pure axial load), 36 degrees, and 83 degrees through the biaxial interaction surface would show that the load combination points fall well within the interaction surface. Therefore, 12-No. 10 bars are also adequate for the 34 × 34 in. column subjected to biaxial bending. Note that the angles of the interaction diagrams are obtained by taking the inverse tangent of the ratio of E-W and N-S factored bending moments. For example, for seismic load case 1 in the second or third load combinations, the angle of the interaction diagram = $\tan^{-1}(181/251) = 36$ degrees (see Table 2-39). The provided reinforcement ratio is within the allowable range of 1% and 6% (ACI 21.4.3.1).

Relative Flexural Strength of Columns and Girders.

ACI 21.4.2 requires that the sum of flexural strengths of columns at a joint must be greater than or equal to 6/5 times the sum of flexural strengths of girders framing into that joint. The intent is to provide columns with sufficient strength so that they will not yield prior to the beams. Yielding at both ends of a column prior to the beams could result in total collapse of the structure. Only seismic load combinations need to be considered when checking the relative strengths of columns and girders.

When computing the nominal flexural strength of girders in T-beam construction, slab reinforcement within an effective slab width defined in ACI 8.10 shall contribute to the flexural strength if the slab reinforcement is developed at the critical section for flexure (ACI 21.4.2.2). Calculations for the effective slab width for an interior beam spanning in the E-W direction and required reinforcement in the slab are given in Section 2.4.4.3 of this publication and are not repeated here. In summary, the effective slab width is equal to 78 in., and 6-No. 4 @ 13 in. are required at both the top and bottom of the slab.

Based on the reinforcement in beam C4-C5 (see Table 2-38) and the reinforcement in the effective width of the slab, a strain compatibility analysis of the section yields $M_n^- =$ 1,264 ft-kips and $M_n^+ = 736$ ft-kips. Therefore, $\sum M_g = 1{,}264 + 736 = 2{,}000$ ft-kips (see Figure 2-31 for sidesway to the left; due to symmetric distribution of flexural reinforcement, sidesway to the right yields the same results for the negative and positive moment strengths).

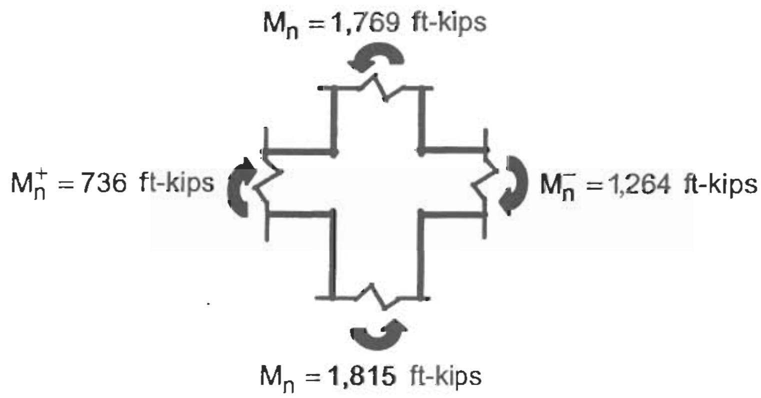


Figure 2-31 Relative Flexural Strength of Columns and Girders when Orthogonal Effects are not Considered (SDC E)

Column flexural strength is determined for the factored axial force resulting in the lowest flexural strength, consistent with the direction of lateral forces considered. For the upper end of the lower column framing into the joint (i.e., the column supporting floor level 2), M_n = 1,815 ft-kips, which corresponds to P_u = 787 kips when orthogonal effects are not considered (see Figure 2-30). Similarly, for the lower end of the upper column framing into the joint (i.e., the column supporting floor level 3), M_n = 1,769 ft-kips, which corresponds to P_u = 716 kips. Therefore, $\sum M_c$ = 3,584 ft-kips.

Check Eq. 21-1:

$$\sum M_c = 3{,}584 \text{ ft-kips} > \frac{6}{5}\sum M_g = \frac{6}{5} \times 2{,}000 = 2{,}400 \text{ ft-kips} \quad \text{O.K.}$$

The same calculations are performed when orthogonal effects are considered. For the upper end of the lower column framing into the joint, the minimum M_n = 1,640 ft-kips, which corresponds to P_u = 787 kips for an angle cut at 36 degrees through the biaxial surface. Similarly, for the lower end of the upper column framing into the joint, M_n = 1,600 ft-kips, which corresponds to P_u = 716 kips. Therefore, $\sum M_c$ = 3,240 ft-kips.

Check Eq. 21-1:

$$\sum M_c = 3{,}240 \text{ ft-kips} > \frac{6}{5}\sum M_g = \frac{6}{5} \times 2{,}000 = 2{,}400 \text{ ft-kips} \quad \text{O.K.}$$

Thus, lateral strength and stiffness of this column can be considered when determining the strength and stiffness of the structure whether orthogonal effects are considered or not.

Design for shear.

Shear requirements for columns in special moment frames are contained in ACI 21.4.5. Similar to beams, the method of determining design shear forces in columns takes into consideration the likelihood of yielding (i.e., plastic hinges forming) in regions near the ends of the member. To properly confine the concrete and to maintain lateral support of the longitudinal bars in regions where yielding is expected, the transverse reinforcement requirements of ACI 21.4.4 must also be satisfied.

Confinement reinforcement. Special transverse reinforcement for confinement is required over a distance ℓ_o from each joint face at both column ends where ℓ_o is equal to the largest of (ACI 21.4.4.4):

- Depth of member = 34 in. (governs)
- Clear span/6 = [(12 × 12) – 32]/6 = 18.7 in.

- 18 in.

Transverse reinforcement within the distance ℓ_o shall not be spaced greater than the smallest of (ACI 21.4.4.2):

- Minimum member dimension/4 = 34/4 = 8.5 in.
- 6(diameter of longitudinal reinforcement) = 6 × 1.27 = 7.6 in.
- $s_x = 4 + \left(\frac{14 - h_x}{3}\right) = 4 + \left(\frac{14 - 11.4}{3}\right) = 4.9 \text{ in.} > 4 \text{ in.}$ (governs)

  < 6 in.

where h_x = maximum horizontal spacing of hoop or crosstie legs on all faces of the 34 × 34 in. column

$$= \frac{34 - 2(1.5 + 0.5) - 1.27}{3} + 1.27 + 0.5 = 11.4 \text{ in.} < 14 \text{ in. O.K. (ACI 21.4.4.3)}$$

assuming No. 4 rectangular hoops with crossties around every longitudinal bar. Therefore, try 4 in. spacing.

Minimum required cross-sectional area of rectangular hoop reinforcement A_{sh} is the larger value obtained from Eqs. 21-3 and 21-4:

$$A_{sh} = \frac{0.3sh_c f_c'}{f_{yh}}\left[\left(\frac{A_g}{A_{ch}}\right) - 1\right] = \frac{0.3 \times 4 \times 30.5 \times 4}{60}\left[\left(\frac{34^2}{961}\right) - 1\right] = 0.50 \text{ in.}^2$$

$$= \frac{0.09sh_c f_c'}{f_{yh}} = \frac{0.09 \times 4 \times 30.5 \times 4}{60} = 0.73 \text{ in.}^2 \quad \text{(governs)}$$

where h_c = cross-sectional dimension of column core measured center-to-center of confinement reinforcement

= 34 – 2[1.5 + (0.5/2)] = 30.5 in.

A_{ch} = cross-sectional area of member measured out-to-out of transverse reinforcement

= $[34 - (2 \times 1.5)]^2 = 961 \text{ in.}^2$

Using No. 4 hoops with 2 crossties provides A_{sh} = 4 × 0.2 = 0.8 in.^{2}, which is greater than the minimum required area from Eq. 21-4. Use 4 in. spacing for the transverse reinforcement at the column ends.

Transverse reinforcement for shear. According to ACI 21.4.5.1, shear forces are computed from statics assuming that moments of opposite sign act at the joint faces corresponding to the probable flexural strengths M_{pr} associated with the range of factored axial loads on the column. For columns in the first story, it is possible for the base of the column to develop its probable flexural strength. For columns above the first story, which is applicable to the column in this example, shear forces based on the probable flexural strengths of the beams framing into the joint will usually control: considering equilibrium at the joint, the largest bending moments and corresponding shear forces that can be transmitted to the column occurs when plastic hinges form at the ends of the beams. Thus, design shear forces in columns need not exceed those determined from joint strengths based on M_{pr} of the beams framing into the joint. Also, the design shear force must not be taken less than that determined from the structural analysis. Sidesway to the right and to the left must be considered when calculating the maximum design shear forces.

It is conservative to assume that the maximum M_{pr} of the column is equal to the moment at the balanced point. When orthogonal effects are not considered and using $f_y = 1.25 \times 60 = 75$ ksi and $\phi = 1.0$, maximum $M_{pr} = 2{,}300$ ft-kips. Design shear force V_e based on the probable flexural strength of the column at the column ends is:

$$V_e = \frac{2 \times 2{,}300}{12 - (26/12)} = 468 \text{ kips}$$

When orthogonal effects are considered, the largest bending moment at the balanced point occurs for an angle cut at 83 degrees through the biaxial surface, which is M_{pr} = 2,162 ft- kips. Design shear force V_e based on this probable flexural strength is:

$$V_e = \frac{2 \times 2{,}162}{12 - (26/12)} = 440 \text{ kips}$$

The positive probable flexural strength of the beam framing into the joint at the face of the column at the 2nd floor level is 819 ft-kips (see Figure 2-27). The negative probable flexural strength of the beam on the other side of the column is 1,170 ft-kips. Assuming that the flexural reinforcement in the 1st floor beams framing into the column is the same as the 2nd floor beams, V_e based on the probable flexural strengths of the beams is:

$$V_e = \frac{2\left(\dfrac{819 + 1{,}170}{2}\right)}{12 - (32/12)} = 213 \text{ kips} \quad \text{(governs)}$$

This design shear force is larger than the maximum shear force obtained from analysis, which is 129 kips (see Table 2-39).

Since the factored axial forces are greater than $A_g f_c' / 20 = 231$ kips, the shear strength of the concrete may be used (ACI 21.4.5.2). The shear capacity of the column is checked using ACI Eq. 11-4 for members subjected to axial compression:

$$V_c = 2\left(1 + \frac{N_u}{2{,}000 A_g}\right)\sqrt{f_c'}\, b_w d$$

$$= 2\left(1 + \frac{787{,}000}{2{,}000 \times 34^2}\right)\sqrt{4{,}000} \times 34 \times 24.2 / 1{,}000 = 139.5 \text{ kips}$$

$$V_s = \frac{A_v f_y d}{s} = \frac{(4 \times 0.2) \times 60 \times 24.2}{4} = 290.4 \text{ kips}$$

$$\phi V_n = \phi(V_c + V_s) = 0.85 \times (139.5 + 290.4) = 365.4 \text{ kips} > 213 \text{ kips}$$

where $N_u = 787$ kips is the smallest axial force on the section (see Table 2-39) and $d =$ 24.2 in. was obtained from a strain compatibility analysis.

Thus, the No. 4 hoops at 4 in. on center required for confinement over the distance ℓ_o at the column ends is also adequate for shear.

According to ACI 21.4.4.6, the remainder of the column must contain hoop reinforcement with center-to center spacing not to exceed 6 times the diameter of the longitudinal column bars = 6 × 1.27 = 7.6 in. or 6 in. (governs). For simpler detailing, use 4 in. spacing of hoops and crossties over entire column height.

Splice length of longitudinal reinforcement.

Lap splices in columns of special moment frames are permitted only within the center half of the member and must be designed as tension lap splices (ACI 21.4.3.2). Also, they must be confined over the entire lap length with transverse reinforcement conforming to ACI 21.4.4.2 and 21.4.4.3. In lieu of lap splices, mechanical splices conforming to ACI 21.2.6 and welded splices conforming to ACI 21.2.7.1 may be utilized.

Since all of the bars are to be spliced at the same location, a Class B splice is required (ACI 12.15.1, 12.15.2):

Class B splice length $= 1.3 \ell_d$

where the development length ℓ_d is computed from ACI Eq. (12-1):

$$\frac{\ell_d}{d_b} = \frac{3}{40}\frac{f_y}{\sqrt{f_c'}}\frac{\alpha\beta\gamma\lambda}{\left(\frac{c+K_{tr}}{d_b}\right)}$$

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 7 and larger bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5 + 0.5 + \dfrac{1.27}{2} = 2.6 \text{ in.} \quad \text{(governs)} \\ \\ \dfrac{34 - 2(1.5 + 0.5) - 1.27}{2 \times 3} = 4.8 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index

$$= \frac{A_{tr} f_{yt}}{1{,}500 sn} = \frac{4 \times 0.2 \times 60{,}000}{1{,}500 \times 4 \times 4} = 2.0$$

$$\frac{c + K_{tr}}{d_b} = \frac{2.6 + 2.0}{1.27} = 3.6 > 2.5, \quad \text{use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.0 \times 1.0 \times 1.0 \times 1.0}{2.5} = 28.5$$

$$\ell_d = 28.5 \times 1.27 = 36.2 \text{ in.}$$

Class B splice length = 1.3 × 36.2 = 47.1 in.

Use a 4 ft-0 in. splice length.

Reinforcement details for column C4 are shown in Figure 2-32.

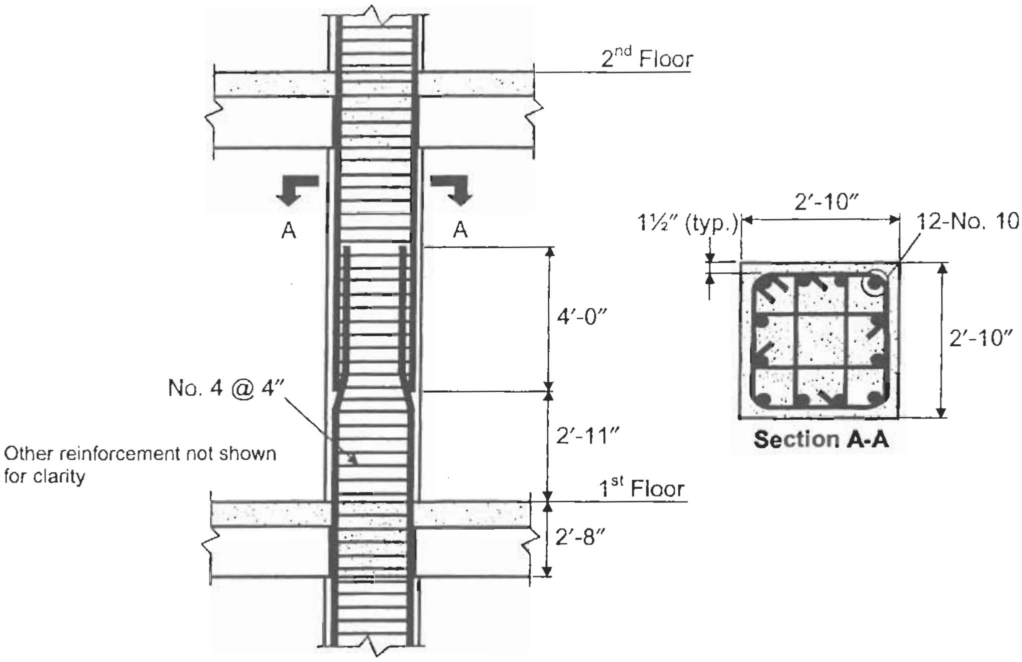


Figure 2-32 Reinforcement Details for Column C4 Supporting the 2nd Floor Level (SDC E)

2.5.4.4 Design of Beam-Column Joint

This section outlines the design of interior and exterior beam-column connections along column line 4. Similar calculations can be performed for joints along other column lines.

The overall integrity of special moment frames is dependent on the behavior of beam-column joints. Degradation of joints can result in large lateral deformations that can cause excessive damage or even failure.

Interior Joint.

Transverse reinforcement for confinement. Transverse reinforcement in a beam-column joint is required to adequately confine the concrete to ensure its ductile behavior and to allow it to maintain its load-carrying capacity even after possible spalling of the outer shell. ACI 21.5.2 requires that transverse hoop reinforcement in accordance with ACI 21.4.4 be provided within the joint, unless the joint is confined on all four sides by beams that have widths equal to at least 75% the column width. When joints are adequately confined on four sides, transverse reinforcement within the joint may be reduced to one-half of that required by ACI 21.4.4 and the hoop spacing is permitted to be a maximum of 6 in.

At an interior joint in the example building, beams frame into all four sides of the column and the width ratio is 28/34 = 0.82 > 0.75. Therefore, transverse reinforcement can be reduced by 50% according to ACI 21.5.2.2 as noted above. However, for detailing simplicity, use the transverse reinforcement required at the column ends (No. 4 @ 4 in.) through the joint.

Shear strength of joint. Figure 2-33 shows the interior beam-column joint at the 2nd floor level. The shear strength is checked in the E-W direction in accordance with ACI 21.5.3. The shear force at section *x-x* is obtained by subtracting the column shear force from the sum of the tensile force in the top beam reinforcement and the compressive force at the top of the beam on the opposite face of the column. Since development of inelastic rotations at the joint face is associated with strains in the beam flexural reinforcement significantly greater than the yield strain, joint shear forces generated by beam reinforcement are calculated based on a stress in the reinforcement equal to $1.25 f_y$ (ACI 21.5.1.1):

$$T_1 \text{ (7 - No.9)} = A_s(1.25 f_y) = (7 \times 1.0) \times (1.25 \times 60) = 525 \text{ kips}$$

$$T_2 \text{ (6 - No.8)} = (6 \times 0.79) \times (1.25 \times 60) = 356 \text{ kips}$$

Column horizontal shear force V_h can be obtained by assuming that adjoining floors are deformed so that plastic hinges form at the ends of the beams. For the beams in this example, $M_{pr}^- = 1{,}170$ ft-kips and $M_{pr}^+ = 819$ ft-kips (see Figure 2-27).

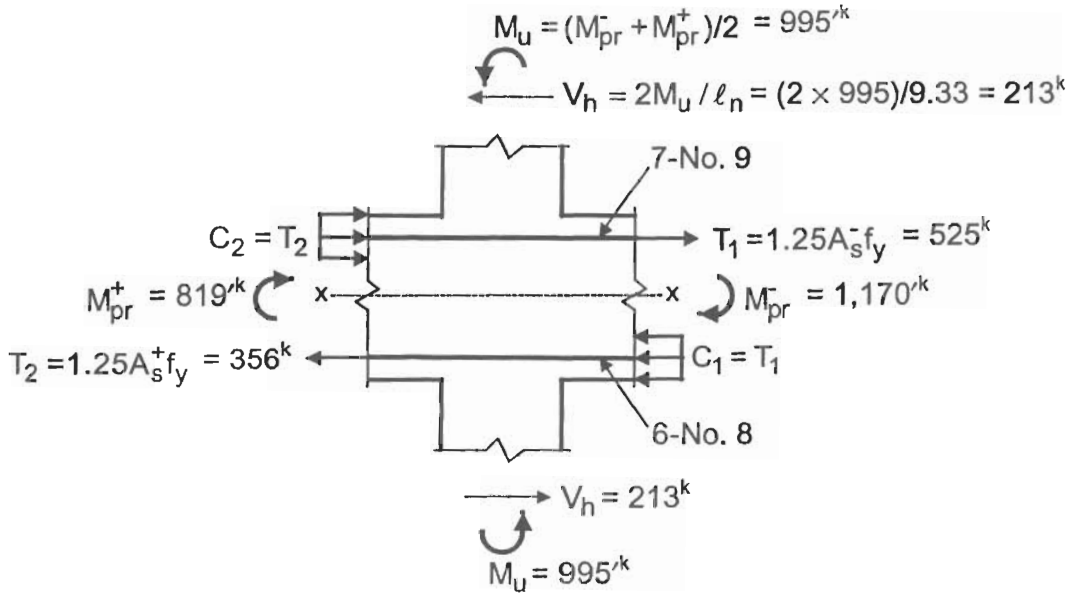


Figure 2-33 Shear Analysis of Interior Beam-Column Joint in E-W Direction (SDC E)

Since the lengths of the columns above and below the joint are equal, moments M_u in the columns above and below the joint are (1,170 + 819)/2 = 995 ft-kips. Shear force V_h in the column is:

$$V_h = \frac{2M_u}{\ell_n} = \frac{2 \times 995}{12-(32/12)} = 213 \text{ kips}$$

The net shear force at section x-x is $T_1 + C_2 - V_h = 525 + 356 - 213 = 668$ kips

For a joint confined on all four faces, nominal shear strength ϕV_c is determined from ACI 21.5.3.1:

$$\phi V_c = \phi 20\sqrt{f_c'} A_j = 0.85 \times 20\sqrt{4{,}000} \times 1{,}156/1{,}000 = 1{,}243 \text{ kips} > 668 \text{ kips} \quad \text{O.K.}$$

where A_j = effective cross-sectional area within a joint in a plane parallel to the plane of reinforcement generating shear in the joint (see ACI Figure R21.5.3). The joint depth is the overall depth of the column in the direction of analysis, which is 34 in. The effective width of the joint is the smaller of (1) beam width + joint depth = 28 + 34 = 62 in. or (2) beam width plus twice the smaller perpendicular distance from the edge of the beam to the edge of the column = 28 + (2 × 3) = 34 in. (governs). Thus, A_j = 34 × 34 = 1,156 in.^{2}

Joint shear strength is a function of concrete strength and effective cross-sectional area A_j only. Tests results show that shear strength of a joint is not altered significantly with changes in transverse reinforcement, provided a minimum amount of such reinforcement is present. Therefore, it is essential that at least minimum transverse reinforcement as specified in ACI 21.5.2 be provided through the joint regardless of the magnitude of calculated shear force in the joint. In cases when the net shear force exceeds the design strength prescribed in ACI 21.5.3.1, only the concrete strength or the effective cross-sectional area can be increased to increase shear capacity.

Reinforcement details for an interior joint are shown in Figure 2-34.

Shear strength requirements must also be satisfied for the N-S direction.

Exterior Joint.

It is assumed for purposes of this example that the beam framing into the edge column along column line 4 has the same flexural reinforcement as the interior beams along this column line.

Transverse reinforcement for confinement. Since an exterior joint is confined on less than four sides, transverse hoop reinforcement in accordance with ACI 21.4.4 must be provided within the joint (ACI 21.5.2.1). For detailing simplicity, use the transverse reinforcement required at the column ends through the joint.

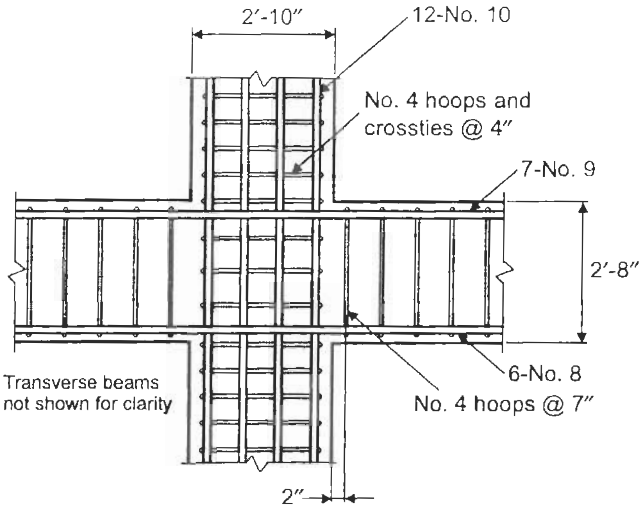


Figure 2-34 Reinforcement Details at Interior Beam-Column Joint (SDC E)

Shear strength of joint. Figure 2-35 shows the exterior joint at the 2nd floor level. In this case, the shear force at section *x-x* is determined by subtracting the column shear force from the tensile force in the top beam reinforcement.

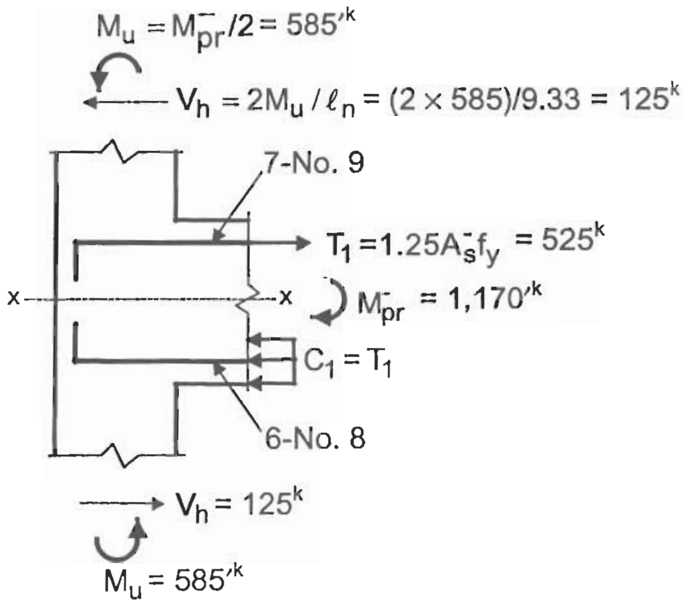


Figure 2-35 Shear Analysis of Exterior Beam-Column Joint in E-W Direction (SDC E)

Since the lengths of the columns above and below the joint are equal, moments M_u in the columns above and below the joint are 1,170/2 = 585 ft-kips. Shear force V_h in the column is:

$$V_h = \frac{2M_u}{\ell_n} = \frac{2 \times 585}{12 - (26/12)} = 125 \text{ kips}$$

The net shear force at section *x-x* is $T_1 - V_h = 525 - 125 = 400$ kips

For a joint confined on three faces, the nominal shear strength ϕV_c is determined from ACI 21.5.3.1:

$$\phi V_c = \phi 15\sqrt{f_c'} A_j = 0.85 \times 15\sqrt{4{,}000} \times 30^2 / 1{,}000 = 726 \text{ kips} > 400 \text{ kips} \quad \text{O.K.}$$

Beam flexural reinforcement terminated in a column must extend to the far face of the confined column core and must be anchored in tension and compression according to ACI 21.5.4 and Chapter 12, respectively (ACI 21.5.1.3). The development length ℓ_{dh} for a bar with a standard 90-degree hook in normal weight concrete is the largest of (ACI 21.5.4.1):

- 8(diameter of longitudinal bar) = 8 × 1.128 = 9 in. for No. 9 top bars
  = 8 × 1.0 = 8 in. for No. 8 bottom bars
- 6 in.
- $\dfrac{f_y d_b}{65\sqrt{f_c'}} = \dfrac{60{,}000 \times 1.128}{65\sqrt{4{,}000}} = 16.5$ in. for No. 9 top bars (governs)

  $= \dfrac{60{,}000 \times 1.0}{65\sqrt{4{,}000}} = 14.6$ in. for No. 8 bottom bars (governs)

Required development lengths for both the top and bottom reinforcement can be accommodated within the 30-in. deep column.

Reinforcement details for the exterior joint are shown in Figure 2-36.

2.5.4.5 Design of Shear Wall on Line 7

This section outlines the design of the shear wall on line 7. According to the supplement to the IBC [2.7], orthogonal effects need not be considered, since the axial forces due to seismic forces (which are equal to zero) are less than 20% of the axial load design strength. Table 2-40 contains a summary of the governing design axial forces, bending moments, and shear forces at the base of the wall due to gravity and seismic forces.

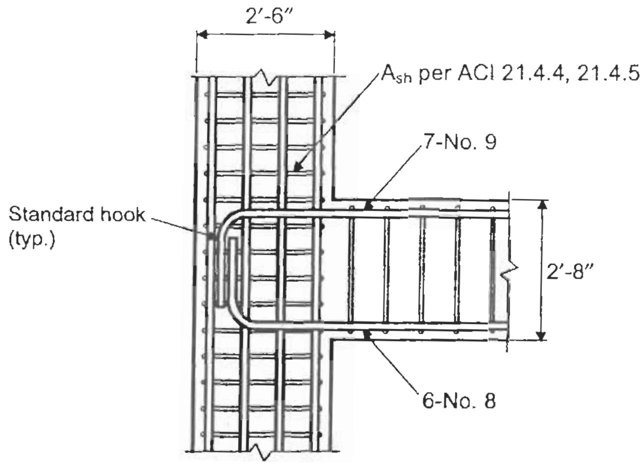


Figure 2-36 Reinforcement Details at Exterior Beam-Column Joint (SDC E)

As noted above, special reinforced concrete structural walls are required in dual systems for buildings assigned to SDC E; thus, the provisions of ACI 21.6, as well as the applicable provisions in Chapters 1 through 18, must be satisfied.

Table 2-40 Summary of Design Axial Forces, Bending Moments, and Shear Forces at Base of Shear Wall on Line 7 (SDC E)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	3,375	0	0
Live (L)	279	0	0
Seismic (Q_E)	0	± 87,479	± 1,460
Load Combinations			
$1.4D + 1.7L$	5,199	0	0
$1.48D + 0.5L + Q_E$	5,135	87,479	1,460
$0.62D + Q_E$	2,093	-87,479	-1,460

Design for shear.

Reinforcement requirements. Special reinforced structural walls are to be provided with reinforcement in two orthogonal directions in the plane of the wall in accordance with ACI 21.6.2. The minimum reinforcement ratio in both directions is 0.0025, unless the design shear force is less than or equal to $A_{cv}\sqrt{f'_c}$, where A_{cv} is the gross area of concrete bounded by the web thickness and the length of wall in the direction of analysis. In such cases, minimum reinforcement in accordance with ACI 14.3 for ordinary walls must be provided. For the wall in this example, $A_{cv} = 18 \times [(22 \times 12) + 40] = 5{,}472$ in.^{2}, so that

$$A_{cv}\sqrt{f'_c} = 5{,}472 \times \sqrt{4{,}000}/1{,}000 = 346 \text{ kips} < V_u = 1{,}460 \text{ kips}$$

Therefore, the minimum reinforcement ratio is 0.0025 and the maximum spacing is 18 in. (ACI 21.6.2.1).

Two curtains of reinforcement are required in a wall when the in-plane factored shear force exceeds $2A_{cv}\sqrt{f'_c}$ = 2 × 346 = 692 kips. In this case, two curtains are required, since V_u = 1,460 kips > 692 kips.

The minimum required reinforcement in each direction per foot of wall is 0.0025 × 18 × 12 = 0.54 in.^{2} Assuming No. 5 bars in two curtains, required spacing s is

$$s = \frac{2 \times 0.31}{0.54} \times 12 = 13.8 \text{ in.} < 18 \text{ in.}$$

Try 2 curtains of No. 5 bars spaced at 13 in.

Shear strength requirements. ACI Eq. 21-7 is used to determine nominal shear strength V_n of structural walls:

$$V_n = A_{cv}(\alpha_c\sqrt{f'_c} + \rho_n f_y)$$

where $\alpha_c = 2$ for ratio of wall height to length h_w/ℓ_w = 148/25.33 = 5.8 > 2 (ACI 21.6.4.1).

For 2 curtains of No. 5 horizontal bars spaced at 13 in. ($\rho_n = 0.62/(18 \times 13) = 0.0026$):

$$\phi V_n = 0.85 \times 5{,}472 \times [2\sqrt{4{,}000} + (0.0026 \times 60{,}000)]/1{,}000$$

$$= 1{,}314 \text{ kips} < V_u = 1{,}460 \text{ kips} \qquad \text{N.G.}$$

where $\phi = 0.85$ for walls with $h_w/\ell_w > 2$ (ACI 9.3.4(a)). Therefore, use 2 curtains of No. 5 bars @ 10 in. on center in horizontal direction (ϕV_n = 1,537 kips). Note that V_n = 1,808 kips is less than the upper limit on shear strength, which is $8A_{cv}\sqrt{f'_c} = 8 \times 346 = 2{,}768$ kips (ACI 21.6.4.4).

Reinforcement ratio ρ_v for the vertical reinforcement must not be less than ρ_n when $h_w/\ell_w \leq 2.0$ (ACI 21.6.4.3). Since h_w/ℓ_w = 5.8 > 2, use minimum reinforcement ratio of 0.0025.

Use 2 curtains of No. 5 bars spaced at 12 in. on center in the vertical direction (ρ_v = 0.0029 > 0.0025).

Design for axial force and bending.

Structural walls subjected to combined flexural and axial loads are designed in accordance with ACI 10.2 and 10.3 except that ACI 10.3.6 and the nonlinear strain requirements of ACI 10.2.2 do not apply (ACI 21.6.5).

Figure 2-37 contains the interaction diagram of the wall. The wall is reinforced with 36-No. 11 bars in the 40 × 40 in. boundary elements at both ends of the wall and 2-No. 5 vertical bars @ 12 in. in the web. As seen from the figure, the wall is adequate for the load combinations in Table 2-40.

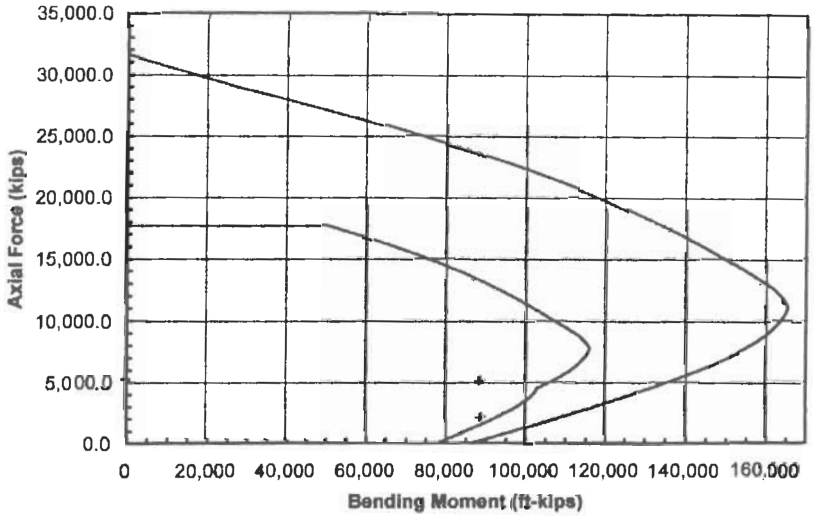


Figure 2-37 Design and Nominal Strength Interaction Diagrams for the Shear Wall Along Line 7 (SDC E)

Special boundary elements.

The need for special boundary elements at the edges of structural walls is evaluated in accordance with ACI 21.6.6.2 or 21.6.6.3. The displacement-based approach in ACI 21.6.6.2 is utilized in this example. In this method, the wall is displaced at the top an amount equal to the expected design displacement; special boundary elements are required to confine the concrete when the strain in the extreme compression fiber of the wall exceeds a critical value. This method is applicable to walls or wall piers that are essentially continuous in cross-section over the entire height and designed to have one critical section for flexure and axial loads.

Compression zones are to be reinforced with special boundary elements where (Eq. 21-8):

$$c \geq \frac{\ell_w}{600(\delta_u / h_w)}, \quad \delta_u / h_w \geq 0.007$$

where c = distance from extreme compression fiber to the neutral axis per ACI 10.2.7 calculated for the factored axial force and nominal moment strength, consistent with the design displacement δ_u, resulting in the largest neutral axis depth

ℓ_w = length of entire wall or segment of wall considered in the direction of the shear force

δ_u = design displacement

= total lateral displacement expected for the design-basis earthquake as specified by the governing code

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

The lower limit on the quantity δ_u / h_w is specified to require moderate wall deformation capacity for stiff buildings.

In this example, $\ell_w = 25.33$ ft = 304 in., $h_w = 148$ ft = 1,776 in., δ_u is equal to δ_x from Table 2-33, which is 18.07 in. at the top of the wall, and $\delta_u / h_w = 0.0102 > 0.007$. Therefore, special boundary elements are required if c is greater than or equal to $304/(600 \times 0.0102) = 49.7$ in.

The distance c to be used in Eq. 21-8 is the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with the design displacement δ_u. From a strain compatibility analysis, the largest c is equal to 64.5 in. corresponding to a factored axial load of 5,135 kips and nominal moment strength 136,330 ft-kips, which is greater than 49.7 in. Therefore, special boundary elements are required.

Special boundary elements must extend horizontally from the extreme compression fiber a distance not less than the larger of the following (ACI 21.6.6.4):

- $c - 0.1\ell_w = 64.5 - (0.1 \times 304) = 34.1$ in. (governs)
- $c/2 = 64.5/2 = 32.3$ in.

Special boundary transverse reinforcement in accordance with ACI 21.6.6.4 is provided in the 40-in. boundary elements at the ends of the wall.

Vertical extent of special boundary transverse reinforcement from base of wall is the larger of (ACI 21.6.6.2(b)):

- $\ell_w = 304$ in. = 25.33 ft (governs)
- $\dfrac{M_u}{4V_u} = \dfrac{87{,}479}{4 \times 1{,}460} = 15.0$ ft

Special boundary element transverse reinforcement.

Provisions for the amount and spacing of transverse reinforcement required in the special boundary elements are contained in ACI 21.6.6.4. In particular, transverse reinforcement requirements of ACI 21.4.4.1 through 21.4.4.3 for columns in special moment frames must be satisfied, except for Eq. 21-3.

Assuming No. 4 rectangular hoops and crossties around every other longitudinal bar in both directions of the 40 × 40 in. special boundary elements, the maximum allowable spacing is the smallest of the following:

- Minimum member dimension/4 = 40/4 = 10.0 in.
- 6(diameter of longitudinal reinforcement) = 6 × 1.41 = 8.5 in.
- $s_x = 4 + \dfrac{14 - h_x}{3} = 4 + \dfrac{14 - 9.6}{3} = 5.5$ in. > 4.0 in. (governs)
  < 6.0 in.

where h_x = maximum horizontal spacing of hoop or crosstie legs on all faces of the 40 × 40 in. special boundary elements

$$= 2\left[\frac{40 - 2(1.5 + 0.5) - 1.41}{9}\right] + 1.41 + 0.5 = 9.6 \text{ in.} < 14 \text{ in.} \quad \text{O.K.} \quad (21.4.4.3)$$

Assuming a spacing of 5 in., required area of transverse reinforcement is determined from Eq. 21-4:

$$A_{sh} = \frac{0.09 s h_c f_c'}{f_{yh}} = \frac{0.09 \times 5 \times 36.5 \times 4}{60} = 1.10 \text{ in.}^2$$

where h_c = cross-sectional dimension of boundary element core measured center-to-center of confinement reinforcement

= 40 – 2[1.5 + (0.5/2)] = 36.5 in.

No. 4 hoops with crossties around every other longitudinal bar provides $A_{sh} = 6 \times 0.2 = 1.20$ in.$^2 > 1.10$ in.^{2} O.K.

Splice length of reinforcement.

Class B lap splices are utilized for the longitudinal reinforcement in the special boundary elements and the vertical bars in the web. Mechanical connectors may be considered as an alternative to lap splices for the large bars in the special boundary element (ACI 21.6.6.4(f)). No splices are required for the No. 5 horizontal bars in the web, since full length bars weigh approximately $1.043 \times 25.33 = 26$ lbs. and are easily installed.

Vertical bars in special boundary elements. For the No. 11 vertical bars in the special boundary elements, ℓ_d is determined from Eq. (12-1), since this results in a longer splice length than determined from ACI 21.5.4 (ACI 21.6.2.3). Assuming no more than 50% of the bars are spliced at any one location:

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 7 and larger bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5+0.50+\dfrac{1.41}{2} = 2.7 \text{ in.} \qquad \text{(governs)} \\[2ex] \dfrac{40-2(1.5+0.5)-1.41}{2\times 4} = 4.3 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index

$$= \frac{A_{tr} f_{yt}}{1{,}500 sn} = \frac{(5\times 0.2)\times 60{,}000}{1{,}500\times 5\times 5} = 1.6$$

$$\frac{c+K_{tr}}{d_b} = \frac{2.7+1.6}{1.41} = 3.1 > 2.5, \quad \text{use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40}\times\frac{60{,}000}{\sqrt{4{,}000}}\times\frac{1.0\times 1.0\times 1.0\times 1.0}{2.5} = 28.5$$

$$\ell_d = 28.5 \times 1.41 = 40.2 \text{ in.}$$

$$\text{Class B splice length} = 1.3\ell_d = 1.3 \times 40.2 = 52.3 \text{ in.}$$

Use a 4 ft-6 in. splice length for the No. 11 bars, with splices staggered at least 24 in. (ACI 12.15.4.1).

Vertical bars in wall web. The provisions of ACI Chapter 12 are utilized to determine the splice length of the vertical bars in the wall web instead of ACI 21.5.4, since ACI 21.5.4 assumes the bars are confined, which they are not in the wall web.

Again assuming that no more than 50% of the bars are spliced at any one location:

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 0.8 for No. 6 and smaller bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 0.75 + 0.625 + \dfrac{0.625}{2} = 1.7 \text{ in.} & \text{(governs)} \\ \dfrac{12}{2} = 6.0 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index = 0

$$\frac{c+K_{tr}}{d_b} = \frac{1.7+0}{0.625} = 2.7 > 2.5, \quad \text{use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{0.8 \times 1.0 \times 1.0 \times 1.0}{2.5} = 22.8$$

$$\ell_d = 22.8 \times 0.625 = 14.3 \text{ in.}$$

Class B splice length = $1.3\ell_d$ = 1.3 × 14.3 = 18.6 in.

Use a 1 ft-7 in. splice length for the No. 5 vertical bars in the web.

Horizontal bars in web. The development length of the No. 5 horizontal bars is determined assuming that no hooks are used in the special boundary elements (ACI 21.6.6.4(e)). The provisions of ACI 21.5.4 are used in this case, since the horizontal bars are terminated in the confined core of the special boundary elements.

Since it is reasonable to assume that the depth of concrete cast in one lift beneath a horizontal bar is greater than 12 in., the required development length must not be less than 3.5 times the length required by ACI 21.5.4.1 (ACI 21.5.4.2):

- 8(diameter of bar) = 8 × 0.625 = 5.0 in.
- 6 in.
- $\dfrac{f_y d_b}{65\sqrt{f'_c}} = \dfrac{60{,}000 \times 0.625}{65\sqrt{4{,}000}} = 9.1$ in. (governs)

Therefore, required development length ℓ_d = 3.5 × 9.1 = 31.9 in. This length can be accommodated within the confined core of the special boundary element so that hooks are not required. However, ACI R21.6.6.4 recommends to anchor the bars with standard 90-degree hooks or mechanical anchorage schemes instead of straight bar development, since large transverse cracks can our in the special boundary elements. Thus, 90-degree hooks are provided at the ends of the No. 5 horizontal bars.

Reinforcement details for the shear wall along line 7 are shown in Figure 2-38.

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2.3 American Concrete Institute, *Building Code Requirements for Structural Concrete (318-99) and Commentary (318R-99)*, Farmington Hills, MI, 1999.

2.4 Computers and Structures, Inc., *SAP2000 – Integrated Finite Element Analysis and Design of Structures*, Berkeley, CA, 1999.

2.5 Structural Engineers Association of California, *2000 IBC Structural/Seismic Design Manual – Volume 1, Code Application Examples*, Sacramento, CA, 2001.

2.6 Ghosh, S.K. and Chittenden, R., *2000 IBC Handbook Structural Provisions*, International Conference of Building Officials, Whittier, CA, 2001.

2.7 International Code Council, *2002 Accumulative Supplement to the International Codes*, Falls Chuch, VA, 2002.

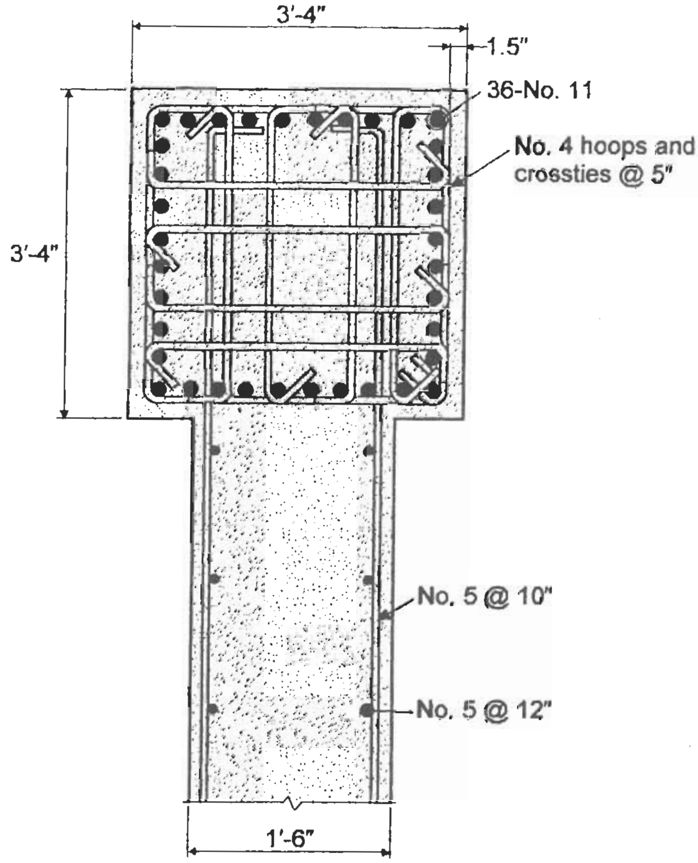

Figure 2-38 Reinforcement Details for Shear Wall Along Line 7 (SDC E)

CHAPTER 3

RESIDENTIAL BUILDING WITH SHEAR WALL-FRAME INTERACTIVE AND BUILDING FRAME SYSTEMS

3.1 INTRODUCTION

According to IBC Table 1617.6, shear wall-frame interactive systems may be used as the seismic-force-resisting system where structures are assigned to low Seismic Design Categories (SDC), i.e., SDC A or B. In such systems, shear walls and frames resist the design lateral forces in proportion to their lateral rigidities.

Building frame systems may be used as the seismic-force-resisting system for structures assigned to SDC C, D, and F, subject to the limitations in IBC Table 1617.6. This system has an essentially complete space frame that resists the gravity loads and shear walls that resist the lateral forces. It is essential that the deformation compatibility requirements of IBC 1617.6.4.3 be satisfied for building frame systems assigned to SDC D and above. These provisions recognize that members that are not designated to be part of the seismic-force-resisting system deform with the members of the seismic-force-resisting system when subjected to the code-specified earthquake design forces, since all of the components are connected at every floor level through the floor systems. Thus, members that are not of the seismic-force-resisting system must be able to carry reactions from gravity forces when subjected to earthquake-induced lateral displacements.

This chapter illustrates the design and detailing of typical structural members in these two types of systems for structures assigned to SDC A, B, C, D, and E.

3.2 DESIGN FOR SDC A

3.2.1 Design Data

A typical plan and elevation of a 9-story residential building is shown in Figure 3-1. The computation of wind and seismic forces according to the 2000 IBC is illustrated below. Typical structural members are designed and detailed for combined effects of gravity, wind, and seismic forces.

A shear wall-frame interactive system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls may be used for structures assigned to SDC A without any limitations according to IBC Table 1617.6. This type of system is utilized in this example.

It is assumed mainly for simplicity that slabs, columns, and walls have constant cross-sections throughout the height of the building, and that the bases of the lowest story segments are fixed. Although the member dimensions in the following sections are within the practical range, the structure itself is a hypothetical one, and has been chosen mainly for illustrative purposes.

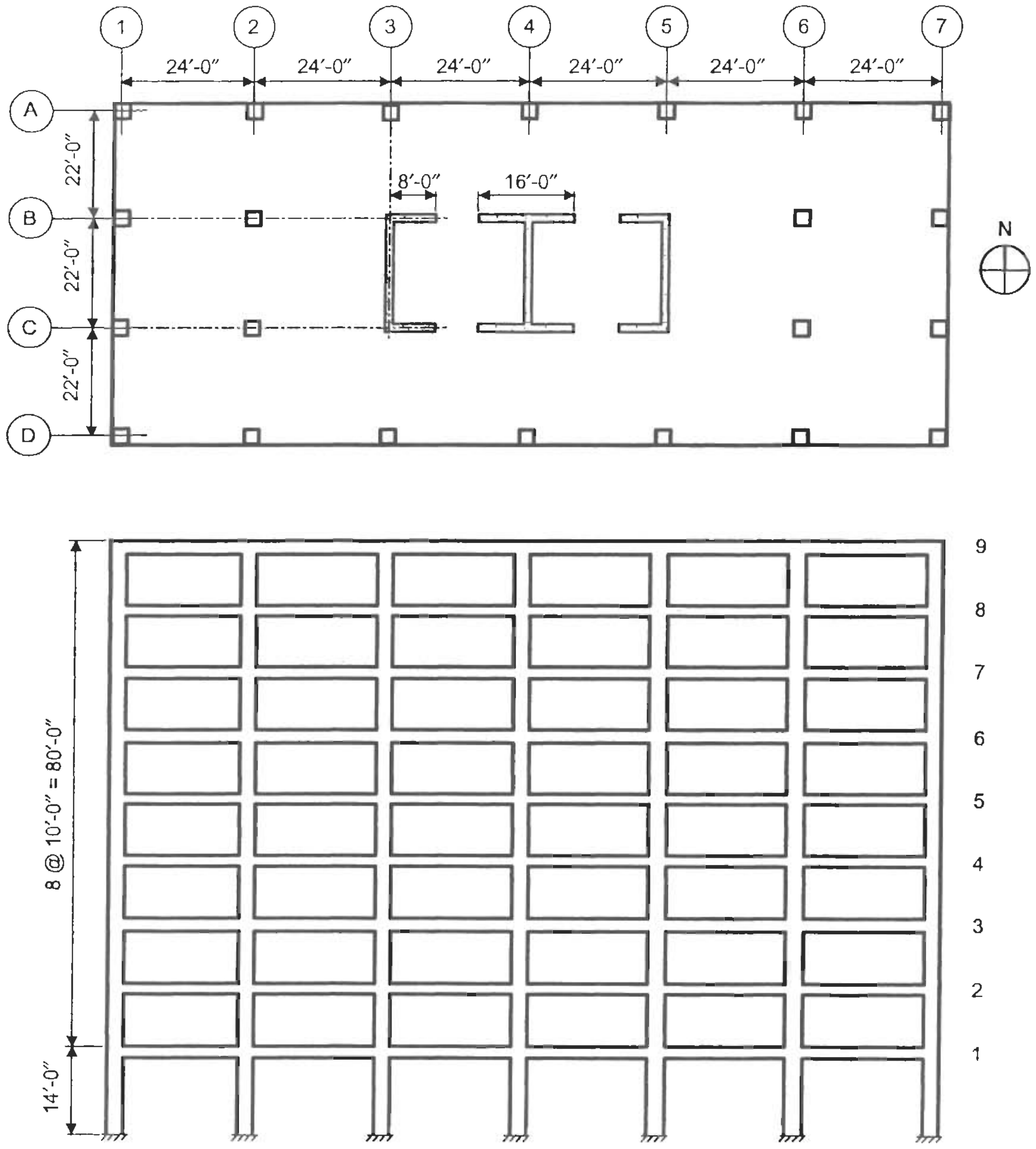

Figure 3-1 Typical Plan and Elevation of Example Building (SDC A)

- Building Location: Miami, FL (zip code 33122)

- Material Properties

  Concrete: $f'_c = 4{,}000$ psi, $w_c = 150$ pcf
  Reinforcement: $f_y = 60{,}000$ psi

- Service Loads

  Live loads: roof = 20 psf
  floor = 50 psf

  Superimposed dead loads: roof = 10 psf + 200 kips for penthouse
  floor = 30 psf (20 psf permanent partitions + 10 psf ceiling, etc.)

- Seismic Design Data

  For zip code 33122: $S_S = 0.065g, S_1 = 0.024g$ [3.1]
  Site Class D (stiff soil profile; IBC Table 1615.1.1)
  For Category (Seismic Use Group) I occupancy, $I_E = 1.0$ (IBC Table 1604.5)

- Wind Design Data

  Basic wind speed = 145 mph (IBC Figure 1609)
  Exposure B (IBC 1609.4)
  For Category I occupancy, $I_W = 1.0$ (IBC Table 1604.5)

- Member Dimensions

  Slab: 9 in.
  Columns: 22 × 22 in.
  Wall thickness: 8 in. in N-S direction, 12 in. in E-W direction

3.2.2 Seismic Load Analysis

Exception 4 of IBC 1614.1 states that structures located where $S_S \le 0.15g$ and $S_1 \le 0.04g$ need only comply with IBC 1616.4, which are design requirements for SDC A. Thus, the minimum lateral force F_x applied simultaneously at each floor level is computed according to IBC Eq. 16-27:

$$F_x = 0.01 w_x$$

where w_x is the portion of the total gravity load W located or assigned to level x and W is defined in IBC 1616.4.1. The seismic forces F_x are summarized in Table 3-1.

Table 3-1 Seismic Forces for SDC A

Level	Story weight, w_x (kips)	Lateral force, $F_x = 0.01 w_x$ (kips)
9	1,533	15.3
8	1,656	16.6
7	1,656	16.6
6	1,656	16.6
5	1,656	16.6
4	1,656	16.6
3	1,656	16.6
2	1,656	16.6
1	1,711	17.1
	Σ	148.6

Note that the seismic forces are the same in both the N-S and E-W directions, and may be applied separately in each of the two orthogonal directions of the building, i.e., orthogonal effects may be neglected (IBC 1616.4.1).

3.2.3 Wind Load Analysis

According to IBC 1609.1.1, wind forces shall be determined in accordance with Section 6 of ASCE 7 [3.2]. Since the building has a mean roof height greater than 30 ft, the simplified procedure (Method 1) given in ASCE 6.4 cannot be used to determine the wind forces. Similarly, the simplified procedure of IBC 1609.6 must not be used, since the building is taller than 60 ft.

The example building is regular shaped by the definition in ASCE 6.2, i.e., it has no unusual geometrical irregularity in spatial form. Also, the building does not have response characteristics making it subject to across wind loading, vortex shedding, or instability due to galloping or flutter. It is assumed that the site location is such that channeling effects or buffeting in the wake of upwind obstructions need not be considered. Thus, the analytical procedure (Method 2) of ASCE 6.5 may be used to determine the wind forces.

Design Procedure.

The design procedure outlined in Section 6.5.3 of ASCE 7 is used to determine the wind forces on the building in both the N-S and E-W directions.

1. Basic wind speed, *V*, and wind directionality factor, K_d. Both quantities are determined in accordance with ASCE 6.5.4. As noted above, *V* is equal to 145 mph for Miami according to IBC Figure 1609 or ASCE Figure 6-1.

The wind directionality factor K_d is equal to 0.85 for main wind-force-resisting systems per ASCE Table 6-6 when load combinations specified in Sections 2.3 and 2.4 are used. Note that these load combinations are essentially the same as those in Sections 1605.2 and 1605.3 of the 2000 IBC. It is important to note exception 1 to IBC 1605.2.1, Basic load combinations: load combinations of ACI 9.2 shall be used for concrete structures where combinations do not include seismic forces. The load factors in the ACI 318 combinations are different than those in ASCE 7 and the IBC. The exception goes on to state that for concrete structures designed for wind in accordance with ASCE 7, wind forces are to be divided by the directionality factor. Thus, in the following computations, instead of multiplying and then subsequently dividing the external wind pressures/forces by 0.85, K_d is taken equal to 1.0.

2. Importance factor, I_W. As noted above, I_W is equal to 1.0 for Category I occupancy according to IBC Table 1604.5 and Category II occupancy according to ASCE Table 1-1 (note that IBC Category I and ASCE 7 Category II are the same).

3. Velocity pressure exposure coefficient, K_z. According to ASCE 6.5.6.4, values of K_z are to be determined from ASCE Table 6-5. In lieu of linear interpolation, K_z may be calculated at any height *z* above ground level from the equations given at the bottom of Table 6-5:

$$K_z = \begin{cases} 2.01\left(\dfrac{15}{z_g}\right)^{2/\alpha} & \text{for } z < 15\text{ ft} \\ 2.01\left(\dfrac{z}{z_g}\right)^{2/\alpha} & \text{for } 15\text{ ft} \le z \le z_g \end{cases}$$

where α = 3-second gust speed power law exponent from ASCE Table 6-4
= 7.0 for Exposure B

z_g = nominal height of the atmospheric boundary layer from ASCE Table 6-4
= 1,200 ft for Exposure B

Values of K_z are summarized in Table 3-2 at the various story heights for the example building.

Table 3-2 Velocity Pressure Exposure Coefficient K_z

Level	Height above ground level, z (ft)	K_z
9	94	0.971
8	84	0.940
7	74	0.907
6	64	0.870
5	54	0.829
4	44	0.782
3	34	0.726
2	24	0.657
1	14	0.575

4. Topographic factor, K_{zt}. The topographic factor is to be determined in accordance with ASCE 6.5.7, Eq. 6-1. Assuming the example building is situated on level ground and not on a hill, ridge, or escarpment, K_{zt} is equal to 1.

5. Gust effect factors, G and G_f. Effects due to wind gust depend on whether a building is rigid or flexible (ASCE 6.5.8). A rigid building has a fundamental natural frequency n_1 greater than or equal to 1 Hz, while a flexible building has a fundamental natural frequency less than 1 Hz (ASCE 6.2).

In lieu of a more exact method, an approximate fundamental period T_a is determined using Eq. 16-39 in IBC 1617.4.2.1. The natural frequency is computed by taking the inverse of the period.

$$T_a = C_T (h_n)^{3/4}$$

where C_T = building period coefficient
= 0.030 for moment-resisting frame systems of concrete
= 0.020 for other types of building systems

Thus,

$$T_a = 0.020 \times (94)^{3/4} = 0.60 \text{ sec} \text{ or } n_1 = 1/0.60 = 1.7 \text{ Hz}$$

Since n_1 > 1.0 Hz, the building is considered rigid, and G may be taken equal to 0.85 or may be calculated by Eq. 6-2 (ASCE 6.5.8.1). For simplicity, G is taken as 0.85.

6. Enclosure classification. It is assumed in this example that the building is enclosed per ASCE 6.5.9 or IBC 1609.2.

7. Internal pressure coefficient, GC_{pi}. According to ASCE 6.5.11.1, internal pressure coefficients are to be determined from Table 6-7 based on building enclosure classification. The building is in a hurricane-prone region. Glazing in the bottom 60 ft of the building is assumed to be debris resistant (ASCE 6.5.9.3). Therefore, for an enclosed building, $GC_{pi} = \pm 0.18$.

8. External pressure coefficients, C_p. External pressure coefficients for main wind-force-resisting systems are given in Figure 6-3 for this example building. For wind in the N-S direction:

Windward wall: $C_p = 0.8$

Leeward wall ($L/B = 67.83/145.83 = 0.47$): $C_p = -0.5$

Side wall: $C_p = -0.7$

Roof ($h/L = 94/67.83 = 1.39$):

$C_p = -1.3$ from 0 to $h/2 = 47$ ft from windward edge. May be reduced to

$0.80 \times -1.3 = -1.04$ for area greater than 1,000 sq ft per Figure 6-3.

$C_p = -0.7$ from 47 ft from windward edge to 67.83 ft

For wind in the E-W direction:

Windward wall: $C_p = 0.8$

Leeward wall ($L/B = 145.83/67.83 = 2.15$): $C_p = -0.29$

Side wall: $C_p = -0.7$

Roof ($h/L = 94/145.83 = 0.65$):

$C_p = -1.02$ from windward edge to $h/2 = 47$ ft

$C_p = -0.84$ from 47 ft to $h = 94$ ft

$C_p = -0.56$ from 94 ft to 145.83 ft

9. Velocity pressure, q_z. The velocity pressure at height z is determined from Eq. 6-13 in ASCE 6.5.10:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I$$

where all terms have been defined previously. Table 3-3 contains a summary of the velocity pressures for the example building.

10. Design wind pressure, p. Design wind pressures on the main wind-force-resisting systems of enclosed and partially enclosed buildings are determined in accordance with

ASCE 6.5.12. For rigid buildings of all heights, design wind pressures are calculated from Eq. 6-15:

$$p = qGC_p - q_i(GC_{pi})$$

Table 3-3 Velocity Pressure q_z (V = 145 mph)

Level	Height above ground level, *z* (ft)	K_z	q_z (psf)
9	94	0.971	52.3
8	84	0.940	50.6
7	74	0.907	48.8
6	64	0.870	46.8
5	54	0.829	44.6
4	44	0.782	42.1
3	34	0.726	39.1
2	24	0.657	35.4
1	14	0.575	30.3

Tables 3-4 and 3-5 contain summaries of design pressures and forces, respectively, for wind in the N-S direction. It has been assumed that the design wind pressure is constant over the tributary height of the floor level. Design wind pressures and forces are contained in Tables 3-6 and 3-7, respectively, for wind acting in the E-W direction.

Table 3-4 Design Wind Pressures in N-S Direction (V = 145 mph)

Location	Level	Height above ground level, *z* (ft)	External Pressure				Internal Pressure		
			q (psf)	G	C_p	qGC_p (psf)	q_i (psf)	GC_{pi}	q_iGC_{pi} (psf)
Windward	9	94	52.3	0.85	0.80	35.6	52.3	± 0.18	± 9.4
	8	84	50.6	0.85	0.80	34.4	52.3	± 0.18	± 9.4
	7	74	48.8	0.85	0.80	33.2	52.3	± 0.18	± 9.4
	6	64	46.8	0.85	0.80	31.8	52.3	± 0.18	± 9.4
	5	54	44.6	0.85	0.80	30.3	52.3	± 0.18	± 9.4
	4	44	42.1	0.85	0.80	28.6	52.3	± 0.18	± 9.4
	3	34	39.1	0.85	0.80	26.6	52.3	± 0.18	± 9.4
	2	24	35.4	0.85	0.80	24.1	52.3	± 0.18	± 9.4
	1	14	30.3	0.85	0.80	20.6	52.3	± 0.18	± 9.4
Leeward	---	All	52.3	0.85	-0.50	-22.2	52.3	± 0.18	± 9.4
Side	---	All	52.3	0.85	-0.70	-31.1	52.3	± 0.18	± 9.4
Roof	---	94*	52.3	0.85	-1.30	-57.8	52.3	± 0.18	± 9.4
	---	94†	52.3	0.85	-0.70	-31.1	52.3	± 0.18	± 9.4

* from windward edge to 47 ft
† from 47 ft to 67.83 ft

Table 3-5 Design Wind Forces in N-S Direction (V = 145 mph)

Level	Height above ground level, z (ft)	Tributary Height (ft)	Windward: External Design Wind Pressure, q_zGC_p (psf)	Windward: Design Wind Force, P* (kips)	Leeward: External Design Wind Pressure, q_hGC_p (psf)	Leeward: Design Wind Force, P* (kips)	Total Design Wind Force (kips)
9	94	5	35.6	25.9	-22.2	16.2	42.1
8	84	10	34.4	50.1	-22.2	32.3	82.4
7	74	10	33.2	48.3	-22.2	32.3	80.6
6	64	10	31.8	46.4	-22.2	32.3	78.7
5	54	10	30.3	44.2	-22.2	32.3	76.5
4	44	10	28.6	41.7	-22.2	32.3	74.0
3	34	10	26.6	38.7	-22.2	32.3	71.0
2	24	10	24.1	35.0	-22.2	32.3	67.3
1	14	12	20.6	36.0	-22.2	38.8	74.8
						Σ	647.4

*$P = qGC_p$ × Tributary height × 145.83

Table 3-6 Design Wind Pressures in E-W Direction (V = 145 mph)

Location	Level	Height above ground level, z (ft)	External Pressure: q (psf)	External Pressure: G_f	External Pressure: C_p	External Pressure: qG_fC_p (psf)	Internal Pressure: q_i (psf)	Internal Pressure: GC_{pi}	Internal Pressure: q_iGC_{pi} (psf)
Windward	9	94	52.3	0.85	0.80	35.6	52.3	± 0.18	± 9.4
	8	84	50.6	0.85	0.80	34.4	52.3	± 0.18	± 9.4
	7	74	48.8	0.85	0.80	33.2	52.3	± 0.18	± 9.4
	6	64	46.8	0.85	0.80	31.8	52.3	± 0.18	± 9.4
	5	54	44.6	0.85	0.80	30.3	52.3	± 0.18	± 9.4
	4	44	42.1	0.85	0.80	28.6	52.3	± 0.18	± 9.4
	3	34	39.1	0.85	0.80	26.6	52.3	± 0.18	± 9.4
	2	24	35.4	0.85	0.80	24.1	52.3	± 0.18	± 9.4
	1	14	30.3	0.85	0.80	20.6	52.3	± 0.18	± 9.4
Leeward	---	All	52.3	0.85	-0.29	-12.9	52.3	± 0.18	± 9.4
Side	---	All	52.3	0.85	-0.70	-31.1	52.3	± 0.18	± 9.4
Roof	---	94*	52.3	0.85	-1.02	-45.3	52.3	± 0.18	± 9.4
	---	94†	52.3	0.85	-0.84	-37.3	52.3	± 0.18	± 9.4
	---	94‡	52.3	0.85	-0.56	-24.9	52.3	± 0.18	± 9.4

*from windward edge to 47 ft
†from 47 ft to 94 ft
‡from 94 ft to 145.83 ft

Table 3-7 Design Wind Forces in E-W Direction (V = 145 mph)

Level	Height above ground level, *z*	Tributary Height	Windward		Leeward		Total Design Wind Force
			External Design Wind Pressure, $q_z G_f C_p$	Design Wind Force, *P* *	External Design Wind Pressure, $q_h G_f C_p$	Design Wind Force, *P* *	
	(ft)	(ft)	(psf)	(kips)	(psf)	(kips)	(kips)
9	94	5	35.6	12.0	-12.9	4.4	16.4
8	84	10	34.4	23.3	-12.9	8.7	32.0
7	74	10	33.2	22.5	-12.9	8.7	31.2
6	64	10	31.8	21.5	-12.9	8.7	30.2
5	54	10	30.3	20.5	-12.9	8.7	29.2
4	44	10	28.6	19.4	-12.9	8.7	28.1
3	34	10	26.6	18.0	-12.9	8.7	26.7
2	24	10	24.1	16.3	-12.9	8.7	25.0
1	14	12	20.6	16.7	-12.9	10.5	27.2
						Σ	246.0

*$P = qGC_p$ × Tributary height × 67.83

3.2.4 Design for Combined Load Effects

It is evident from Tables 3-1, 3-5, and 3-7 that wind forces will govern the design of the structural members for SDC A. It is important to note, however, that IBC 1609.1.5 requires that seismic detailing requirements of the code be satisfied, even when wind load effects are greater than seismic load effects.

Chapter 19 of the 2000 IBC references ACI 318-99 [3.3] for the design of reinforced concrete members; provisions that differ from ACI 318 are given in italicized text. According to ACI 21.2.1.2, design of members in regions of low seismic risk shall satisfy Chapters 1 through 18 and 22, and no special seismic detailing is required.

A three-dimensional analysis of the building was performed in the N-S and E-W directions for the wind loads contained in Tables 3-5 and 3-7 using SAP2000 [3.4]. In the model, rigid diaphragms were assigned at each floor level, and rigid-end offsets were defined at the ends of the horizontal members so that results were automatically obtained at the faces of the supports. The stiffness properties of the members were input assuming cracked sections. In lieu of a more accurate analysis, the following cracked section properties were used:

- Slabs: $I_{eff} = I_g$
- Columns: $I_{eff} = 0.7 I_g$
- Shear walls: $I_{eff} = 0.7 I_g$

where I_g is the gross moment of inertia of the section.

When subjected to lateral forces, flat plate structures behave similarly to rigid frames comprised of columns and beams. For slab-column frames, only a portion of the slab is effective across its full width in resisting the effects of lateral forces. The slab can be replaced by an equivalent beam that has the same flexural stiffness as that of the slab. By using the same thickness, modulus of elasticity, and span length as those of the slab, the only parameter that needs to be determined is the equivalent beam width. An analytical study has shown that for an interior frame, internal forces in the columns and slabs do not vary much as the effective width varies from one-fourth to one-half the bay width [3.5]. Therefore, in this example, the effective beam width is set equal to one-fourth and one-eighth of the bay width in the direction perpendicular to the direction of analysis for interior and exterior frames, respectively. This effective width includes a reduction in stiffness for cracking, so that the effective moment of inertia of the slab can be computed from:

$$I_{eff} = b_e h^3 / 12$$

where b_e = effective width of slab

= (bay width)/4 for interior frames

= (bay width)/8 for exterior frames

h = slab thickness

According to ASCE 6.5.12.3, the main wind-force-resisting systems of buildings with mean roof height h greater than 60 ft must be designed for the full and partial wind load cases of Figure 6-9 (Cases 1 through 4). These four cases were considered in the three-dimensional analysis.

3.2.4.1 Load Combinations

As noted above, IBC 1605.2.1 requires that the non-seismic load combinations of ACI 9.2 be used for concrete structures. Thus, the following load combinations are utilized in the design of the structural members:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)

where D, L, and W are the effects due to dead, live, and wind loads, respectively. It is important to reiterate that the wind loads were computed utilizing a directionality factor equal to one (IBC 1605.2.1, exception 1).

3.2.4.2 Slab Design

Preliminary Slab Thickness.

Minimum overall thickness h for slab systems without edge and interior beams is determined from ACI Table 9.5(c). For systems without drop panels and with Grade 60 reinforcement, the minimum thickness is:

$$\text{Minimum } h = \frac{\ell_n}{30} = \frac{(24 \times 12) - 22}{30} = 8.9 \text{ in.}$$

where ℓ_n is the longest clear span length. Therefore, a 9 in. slab thickness satisfies the serviceability provisions of ACI 9.5.3.2.

When two-way slabs are supported directly on columns, shear near the columns is critically important, especially at exterior slab-column connections without spandrel beams. For flat plates, slab thickness will almost always be governed by two-way shear rather than serviceability requirements. Shear strength requirements are checked for both one-way and two-way shear forces due to gravity loads to determine if the 9-in. thick slab is adequate or not. It is worthwhile to perform these preliminary calculations at this stage so that any if adjustment in slab thickness is required, it can be made before additional calculations are performed.

Live loads on the slab are reduced in accordance with IBC 1607.9.2 where the reduction factor R is determined from Eq. 16-2:

$$R = r(A - 150) \leq 40\% \text{ for horizontal members}$$

where r = rate of reduction = 0.08 for floors

A = area of floor supported by member = $24 \times 22 = 528 \text{ ft}^2 > 150 \text{ ft}^2$

Therefore,

$$R = 0.08(528 - 150) = 30\% < 40\% \quad \text{O.K.}$$

Reduced live load = $(1 - 0.30) \times 50 = 35$ psf

The design loads are:

$$\text{Factored slab weight} = 1.4 \times \frac{9}{12} \times 150 = 158 \text{ psf}$$

Factored superimposed dead load = $1.4 \times 30 = 42$ psf

Factored live load = $1.7 \times 35 = 60$ psf

Total factored load w_u = 158 + 42 + 60 = 260 psf

One-way shear. Investigation of one-way (beam) shear is made at a distance d = 9 – 1.25 = 7.75 in. from the face of the support in either direction, and is shown in Figure 3-2 at the section that gives the largest shear force (ACI 11.12.1.1).

Factored shear force at critical section is:

$$V_u = 0.260 \times \left(12 - \frac{11}{12} - \frac{7.75}{12}\right) \times 22 = 59.7 \text{ kips}$$

From ACI Eq. 11-3, shear design strength is:

$$\phi V_c = \phi 2\sqrt{f'_c}\, b_w d$$

$$= 0.85 \times 2\sqrt{4{,}000} \times (22 \times 12) \times 7.75/1{,}000 = 220.0 \text{ kips} > 59.7 \text{ kips} \quad \text{O.K.}$$

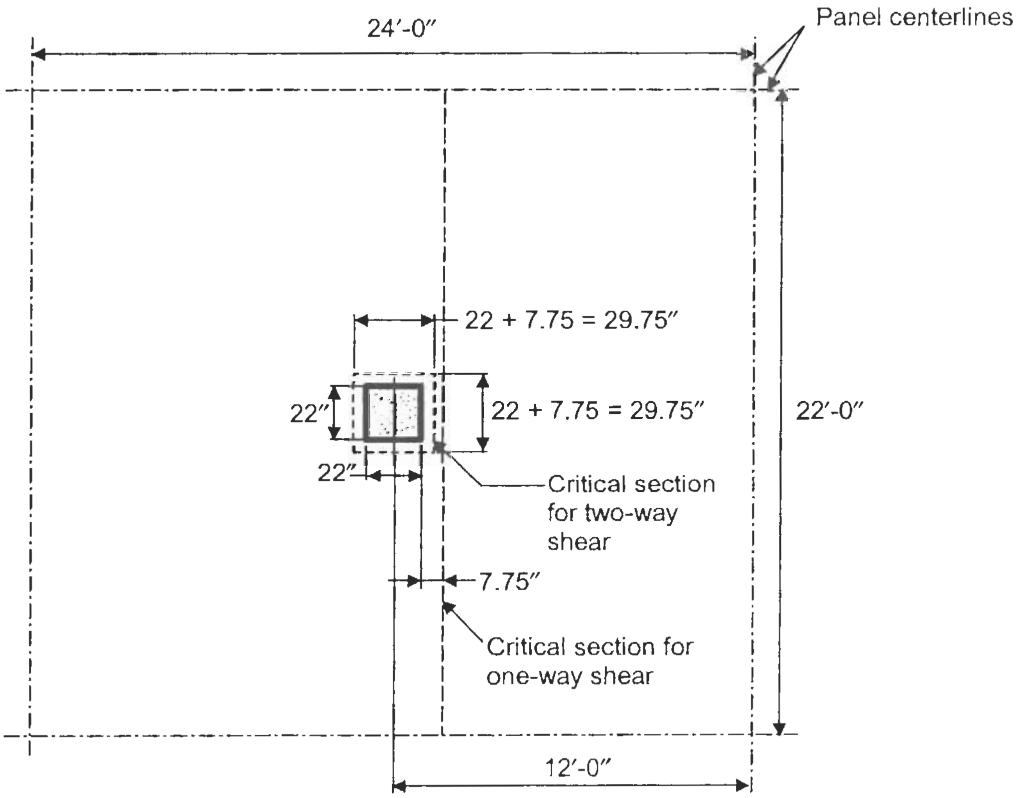

Figure 3-2 Critical Sections for One-way and Two-way Shear at an Interior Column

Two-way shear. Investigation of two-way shear is made at a distance $d/2$ from the face of the support, as shown in Figure 3-2 for an interior column (ACI 11.12.1.2). Maximum factored shear force at this location is:

$$V_u = w_u(\ell_1\ell_2 - b_1 b_2)$$

$$= 0.260\left[(24\times 22) - \frac{29.75\times 29.75}{144}\right] = 135.7 \text{ kips}$$

Design shear strength is the smallest value obtained from Eqs. 11-35 through 11-37. For square columns, Eq. 11-37 governs:

$$\phi V_c = \phi 4\sqrt{f_c'}\,b_o d$$

$$= 0.85\times 4\sqrt{4{,}000}\times(4\times 29.75)\times 7.75/1{,}000 = 198.3 \text{ kips} > 135.7 \text{ kips O.K.}$$

Shear strength requirements at an edge column are also critical, since the total unbalanced moment at this location must be transferred through the slab-column connection. The design aid presented in Figure 3-3 can be used to determine a preliminary slab thickness for a flat plate assuming the following [3.6]:

- square edge columns of size c_1 (three-sided critical section)
- column supports a tributary area A
- square bays ($\ell_1 = \ell_2$)
- gravity load moment transferred between the slab and column in accordance with the Direct Design Method provisions in ACI 13.6.3.6 (note: it is shown below that the Direct Design Method can be utilized to determine the design moments for this system)
- normal weight concrete with $f_c' = 4{,}000$ psi

A preliminary slab thickness can be obtained by adding 1.25 in. to d from the figure. In this example, w_u = 260 psf and A/c_1^2 = (22 × 12.92)/(22/12)^2 = 85. From Figure 3-3, $d/c_1 \approx$ 0.35 so that d = 0.35 × 22 = 7.7 in. and h = 7.7 + 1.25 = 8.95 in.

Therefore, preliminary design indicates that a 9-in. thick slab is adequate for control of deflection and shear strength due to gravity loads. It is important to note that the check for shear strength is only preliminary at this stage. The shear stress needs to be checked for gravity loads and combined gravity and wind loads at both an interior and exterior slab-column connection. A more refined check for shear strength is made at a later stage.

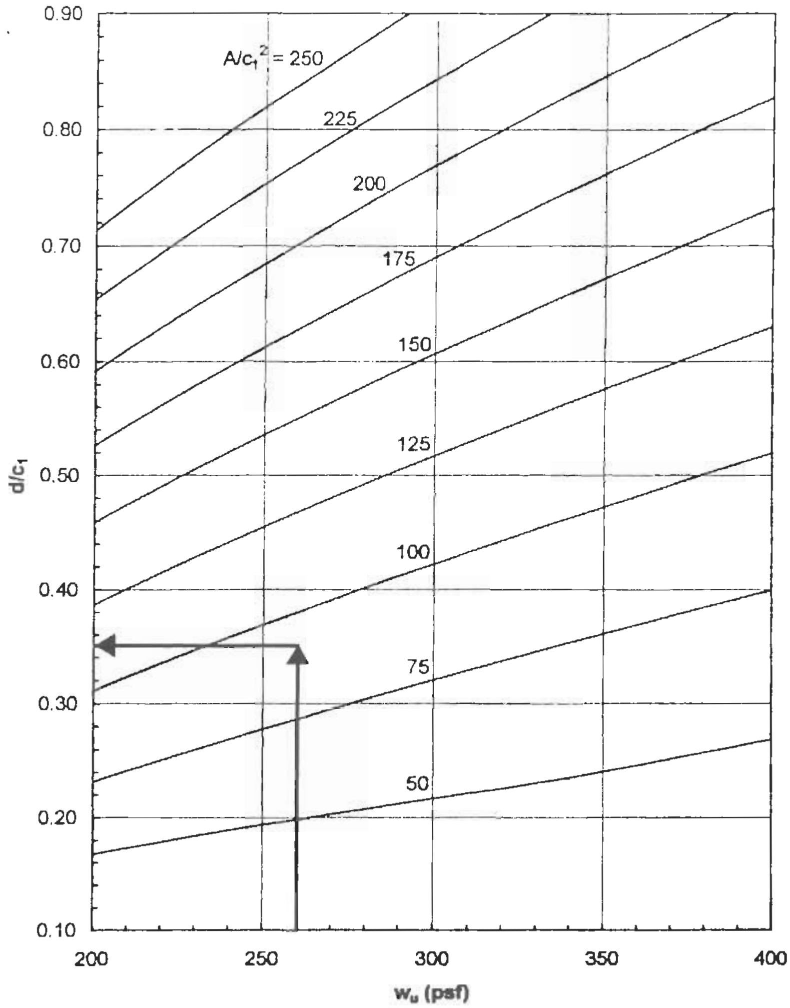

Figure 3-3 Preliminary Slab Thickness for Flat Plate Based on Two-way Shear at an Edge Column

Design for Flexure.

In lieu of other analysis methods, the Direct Design Method in ACI 13.6 is utilized in this example to determine the bending moments in the slab due to gravity loads. This method can be used only if certain geometric and loading criteria are met. All of the criteria are met in this example, as indicated below:

- There shall be a minimum of 3 consecutive spans in each direction.
  ...Structure has 3 spans in one direction, 6 in the other direction. O.K.

- Panels shall be rectangular with a center-to-center longer span to shorter span length ratio not greater than 2.
  ...Longer span/shorter span = 24/22 = 1.1 < 2 O.K.

- Successive center-to-center span lengths in each direction shall not differ by more than one-third the longer span.

  ...In each direction, the span lengths are equal. O.K.

- Columns may be offset a maximum of 10% of the span in the direction of offset from either axis between centerlines of successive columns.

  ...No column offsets are present. O.K.

- Loads must be uniformly distributed gravity loads only and the live load must be less than or equal to 2 times the dead load.

  ...Uniform live to dead load ratio = 35/142.5 = 0.3 < 2 O.K.

Therefore, the Direct Design Method can be used for gravity load analysis.

Using ACI Eq. 13-3, the total static service dead load and live load moments for spans in an interior design strip in the E-W direction are computed as follows:

$$M_o = \frac{w\ell_n^2 \ell_2}{8}$$

$$(M_o)_D = \frac{0.143 \times 22.17^2 \times 22}{8} = 193.3 \text{ ft - kips}$$

$$(M_o)_L = \frac{0.035 \times 22.17^2 \times 22}{8} = 47.3 \text{ ft - kips}$$

These moments must be divided into positive and negative moments, and then into column strip and middle strip moments according to the distribution factors given in ACI 13.6.3, 13.6.4, and 13.6.6. The distribution factors for a flat plate system are contained in Table 3-8, where negative moments are at the face of the supports. Table 3-9 contains a summary of the service gravity load bending moments in the column strip and middle strip of an end span and an interior span.

Table 3-8 Design Moments for Flat Plate Supported Directly on Columns

Slab Moments	**End Span**			**Interior Span**	
	Exterior Negative	Positive	Interior Negative	Positive	Interior Negative
Total Moment	$0.26M_O$	$0.52M_O$	$0.70M_O$	$0.35M_O$	$0.65M_O$
Column Strip	$0.26M_O$	$0.31M_O$	$0.53M_O$	$0.21M_O$	$0.49M_O$
Middle Strip	0	$0.21M_O$	$0.17M_O$	$0.14M_O$	$0.16M_O$

Table 3-9 Service Dead and Live Load Bending Moments (ft-kips) in E-W Interior Design Strip

Location		Moment	M_D	M_L
End Span				
Column Strip	Exterior Negative	$0.26M_O$	-50.3	-12.3
	Positive	$0.31M_O$	59.9	14.7
	Interior Negative	$0.53M_O$	-102.5	-25.1
Middle Strip	Exterior Negative	0	0	0
	Positive	$0.21M_O$	40.6	9.9
	Interior Negative	$0.17M_O$	-32.9	-8.0
Interior Span				
Column Strip	Positive	$0.21M_O$	40.6	9.9
	Interior Negative	$0.49M_O$	-94.7	-23.2
Middle Strip	Positive	$0.14M_O$	27.1	6.6
	Interior Negative	$0.16M_O$	-30.9	-7.6

A summary of the governing design bending moments due to gravity and wind load effects is contained in Table 3-10 at the fourth floor level where the bending moments due to wind are the largest in the slab. These bending moments are the maximum obtained from the full and partial wind load cases of ASCE 7 Figure 6-9 (Cases 1 through 4).

Table 3-10 Summary of Slab Design Bending Moments (ft-kips) at the 4th Floor Level (SDC A)

Load Case	**Location**	**End Span**		**Interior Span**	
		Column Strip	**Middle Strip**	**Column Strip**	**Middle Strip**
Dead (*D*)	Ext. neg.	-50.3	0		
	Positive	59.9	40.6	40.6	27.1
	Int. neg.	-102.5	-32.9	-94.7	-30.9
Live (*L*)	Ext. neg.	-12.3	0		
	Positive	14.7	9.9	9.9	6.6
	Int. neg.	-25.1	-8.0	-23.2	-7.6
Wind (*W*)	Ext. neg.	±5.6			
	Positive				
	Int. neg.	±5.8		±4.8	
Load Combinations					
$1.4D + 1.7L$	Ext. neg.	-91.3	0		
	Positive	108.9	73.7	73.7	49.2
	Int. neg.	-186.2	-59.7	-172.0	-56.2
$0.75(1.4D + 1.7L + 1.7W)$	Ext. neg.	-75.6	0		
	Positive	81.7	55.3	55.3	36.9
	Int. neg.	-147.1	-44.8	-135.1	-42.2
$0.9D + 1.3W$	Ext. neg.	-38.0	0		
	Positive	53.9	36.5	36.5	24.4
	Int. neg.	-84.7	-29.6	-79.0	-27.8

The required flexural reinforcement is contained in Table 3-11. The provided areas of steel are greater than the minimum required (ACI 13.3.1). Also, the provided spacing is less than the maximum allowed according to ACI 13.3.2.

Table 3-11 Required Slab Reinforcement at the 4th Floor Level (SDC A)

Location			M_u (ft-kips)	b (in.)	A_s^* (in.^{2})	Reinforcement*
End Span	Column strip	Ext. neg.	-91.3	132	2.66	9-No. 5
		Positive	108.9	132	3.17	11-No. 5
		Int. neg.	-186.2	132	5.61	19-No. 5
	Middle strip	Ext. neg.	0	132	2.14	8-No. 5
		Positive	73.7	132	2.15	8-No. 5
		Int. neg.	-59.7	132	2.14	8-No. 5
Interior Span	Column strip	Positive	73.7	132	2.15	8-No. 5
		Negative	-172.0	132	5.12	17-No. 5
	Middle strip	Positive	49.2	132	2.14	8-No. 5
		Negative	-56.2	132	2.14	8-No. 5

* Minimum $A_s = 0.0018bh = 0.0018 \times 132 \times 9 = 2.14$ in.^{2} (ACI 13.3.1)
Maximum spacing = $2h$ = 18 in. For b = 132 in., 132/18 = 7.3 spaces, say 8 bars

The amount of slab reinforcement needs to be checked at the end support and the first interior support to ensure that the moment transfer requirements in ACI 13.5.3 are satisfied.

End Support – Additional Flexural Reinforcement Required for Moment Transfer. The total unbalanced moment at this slab-column connection is equal to 91.3 ft-kips (see Table 3-11). A fraction of this moment $\gamma_f M_u$ must be transferred over an effective width equal to $c_2 + 3h = 22 + (3 \times 9) = 49$ in. (ACI 13.5.3.2).

The fraction of unbalanced moment transferred by flexure is calculated from Eq. 13-1:

$$\gamma_f = \frac{1}{1+(2/3)\sqrt{b_1/b_2}} = \frac{1}{1+(2/3)\sqrt{[22+(7.75/2)]/(22+7.75)}} = 0.62$$

For edge columns bending perpendicular to the edge, the value of γ_f computed from Eq. 13-1 is permitted to be increased by up to 25% provided that $V_u \le 0.4\phi V_c$ (ACI 13.5.3.3). No adjustment to γ_f was made in this example.

Unbalanced moment transferred by flexure = $\gamma_f M_u$ = 0.62 × 91.3 = 56.6 ft-kips. The required area of steel to resist this moment in the 49-in. wide strip is A_s = 1.69 in.^{2}, which is equivalent to 6-No. 5 bars.

Provide the 6-No. 5 bars by concentrating 6 of the column strip bars (9-No. 5) within the 49 in. width over the column. For symmetry, add another bar in the column strip and check bar spacing:

- For 6-No. 5 within 49 in. width: 49/6 = 8.2 in. < 18 in. O.K.
- For 4-No. 5 within 132 – 49 = 83 in. width: 83/4 = 20.8 in. > 18 in. N.G.

Therefore, add 2 more No. 5 bars in the 83 in. width; bar spacing = 83/6 = 13.8 in. O.K.

Figure 3-4 shows the reinforcement detail for the top bars at the exterior column.

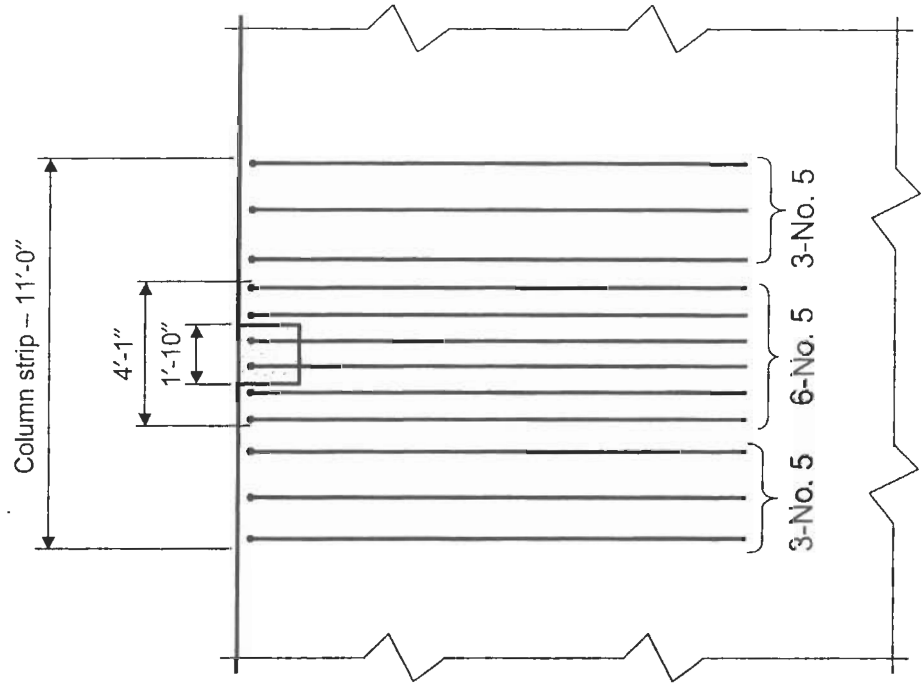

Figure 3-4 Reinforcement Detail at Exterior Column (SDC A)

First Interior Support – Additional Flexural Reinforcement Required for Moment Transfer. The total factored moment in the column strip at the first interior support is 186.2 ft-kips (see Table 3-11). At an interior column, $b_1 = b_2 = 22 + 7.75 = 29.75$ in. Therefore, from Eq. 13-1, $\gamma_f = 0.60$, and $\gamma_f M_u = 0.60 \times 186.2 = 111.7$ ft-kips. Required area of steel to resist this moment in the 49-in. wide strip is $A_s = 3.49$ in.^{2}, which is equivalent to 12-No. 5 bars.

Provide the 12-No. 5 bars by concentrating 12 of the column strip bars (19-No. 5) within the 49 in. width over the column. For symmetry, add another bar in the column strip and check bar spacing:

- For 12-No. 5 within 49 in. width: 49/12 = 4.1 in. < 18 in. O.K.
- For 8-No. 5 within 132 – 49 = 83 in. width: 83/8 = 10.4 in. < 18 in. O.K.

Design for Shear.

In general, the total shear stress is the sum of the direct shear stress plus the shear stress due to the fraction of unbalanced moment transferred by eccentricity of shear (ACI 11.12.6.1). Assuming shear stress from moment transfer by eccentricity of shear varies linearly about the centroid of the section, maximum factored shear stress v_u at the face of the critical section due to direct shear V_u and the unbalanced moment transferred by eccentricity of shear $\gamma_v M_u$ is:

$$v_u = \frac{V_u}{A_c} + \frac{\gamma_v M_u c}{J}$$

where A_c = area of concrete section resisting shear transfer = $b_o d$

b_o = perimeter of critical section

γ_v = fraction of unbalanced moment transferred by eccentricity of shear

$$= 1 - \gamma_f = 1 - \frac{1}{1 + (2/3)\sqrt{b_1 / b_2}} \quad \text{(Eqs. 11-41, 13-1)}$$

M_u = factored unbalanced moment at slab-column connection

J = property of critical section analogous to polar moment of inertia of segments forming A_c

c = distance from centroidal axis of critical section to perimeter of critical section in direction of analysis

Shear strength is checked at the slab-column connections at the end support and the first interior support for gravity forces and for gravity plus wind forces.

End Support – Check for Shear Strength. At this location, the factored shear force V_u due to gravity loads is:

$$V_u = w_u (A_t - b_1 b_2) - \frac{M_1 - M_2}{\ell_n}$$

$$= 0.260\left[284.2 - \frac{25.875 \times 29.75}{144}\right] - \frac{246.0 - 91.4}{24 - (22/12)}$$

$$= 72.5 - 7.0 = 65.5 \text{ kips}$$

where A_t = tributary area of column

$= \left(\frac{24}{2}+\frac{11}{12}\right)\times 22 = 284.2 \text{ ft}^2$

b_1 = length of critical section perimeter in direction of analysis

= 22 + (7.75/2) = 25.875 in. (see Figure 3-5)

b_2 = length of critical section perimeter perpendicular to direction of analysis

= 22 + 7.75 = 29.75 in.

M_1 = total negative design strip moment at interior support determined from Direct Design Method (ACI 13.6.6.3)

$= 0.70 M_o = 0.70 \times 351.4 = 246.0$ ft-kips (see Table 3-8)

M_2 = total negative design strip moment at exterior support determined from Direct Design Method (ACI 13.6.6.3)

$= 0.26 M_o = 0.26 \times 351.4 = 91.4$ ft-kips (see Table 3-8)

M_o = total factored static moment in span determined from Eq. 13-3

$$= \frac{w_u \ell_2 \ell_n^2}{8} = \frac{0.260\times 22\times 22.17^2}{8} = 351.4 \text{ ft-kips}$$

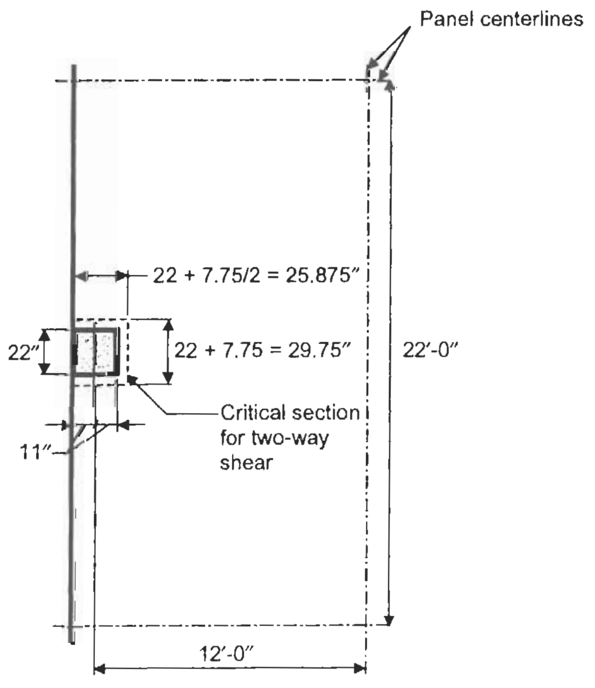

Figure 3-5 Critical Section for Two-way Shear at Exterior Column

The section properties of the critical section are determined as follows [3.7]:

$$A_c = (2b_1 + b_2)d = 631.6 \text{ in.}^2$$

$$\frac{J}{c} = \frac{2b_1^2 d(b_1 + 2b_2) + d^3(2b_1 + b_2)}{6b_1} = 5{,}951 \text{ in.}^3$$

According to ACI 13.6.3.6, gravity load moment to be transferred between the slab and edge column must be set equal to $0.30M_o$ = 105.4 ft-kips. Also, $\gamma_v = 1 - 0.62 = 0.38$. Therefore, the combined factored shear stress at the face of the critical section due to gravity loads is:

$$v_u = \frac{65{,}500}{631.6} + \frac{0.38 \times 105.4 \times 12{,}000}{5{,}951}$$

$$= 103.7 + 80.8 = 184.5 \text{ psi}$$

Design shear strength of nonprestressed slabs is the smallest value obtained from Eqs. 11-35 through 11-37. For a square column, Eq. 11-37 governs:

$$\phi v_c = \frac{\phi V_c}{b_o d} = \phi 4\sqrt{f_c'} = 0.85 \times 4\sqrt{4{,}000} = 215.0 \text{ psi} > 184.5 \text{ psi} \quad \text{O.K.}$$

In addition to the gravity load case, shear strength must be checked for combined gravity and wind loads. From previous calculations, direct shear force on critical section due to gravity loads is $1.4V_D + 1.7V_L$ = 65.5 kips. The shear force due to wind loads V_W = (5.6 + 5.8)/22.17 = 0.5 kip. Therefore, total factored shear force at exterior column is:

$$V_u = 0.75(1.4V_D + 1.7V_L + 1.7V_w) = 0.75[65.5 + (1.7 \times 0.5)] = 49.8 \text{ kips}$$

When wind is considered, shear stress computations can be based on the actual unbalanced moment, rather than on the provisions of ACI 13.6.3.6, which, as shown above, requires the unbalanced moment to be $0.30M_o$. The actual unbalanced moment at the exterior slab-column connection is 75.6 ft-kips from the second load combination (see Table 3-10). The combined shear stress is:

$$v_u = \frac{49{,}800}{631.6} + \frac{0.38 \times 75.6 \times 12{,}000}{5{,}951}$$

$$= 78.9 + 57.9 = 136.8 \text{ psi} < 215.0 \text{ psi} \quad \text{O.K.}$$

First Interior Support – Check for Shear Strength. At this location, the factored shear force V_u due to gravity loads is:

$$V_u = w_u(A_t - b_1 b_2) + \frac{M_1 - M_2}{\ell_n}$$

$$= 0.260\left[528 - \frac{29.75^2}{144}\right] + \frac{246.0 - 91.4}{24 - (22/12)}$$

$$= 135.7 + 7.0 = 142.7 \text{ kips}$$

where $A_t = 24 \times 22 = 528 \text{ ft}^2$

$b_1 = b_2 = 22 + 7.75 = 29.75$ in.

$M_1 = 0.70 M_o = 0.70 \times 351.4 = 246.0$ ft-kips (see Table 3-8)

$M_2 = 0.26 M_o = 0.26 \times 351.4 = 91.4$ ft-kips (see Table 3-8)

The section properties of the critical section are determined as follows [3.7]:

$$A_c = 2(b_1 + b_2)d = 922.3 \text{ in.}^2$$

$$\frac{J}{c} = \frac{b_1 d(b_1 + 3b_2) + d^3}{3} = 9{,}301 \text{ in.}^3$$

The difference between the slab moments acting on opposite faces of the interior support needs to be transferred by shear to the first interior column. From Table 3-8, the exterior moment at the face of the support is $0.70M_o$ = 0.70 × 351.4 = 246.0 ft-kips, and the interior moment at the face of the support is $0.65M_o$ = 228.4 ft-kips. Therefore, the unbalanced moment = 246.0 – 228.4 = 17.6 ft-kips. The combined shear stress is:

$$v_u = \frac{142{,}700}{922.3} + \frac{0.4 \times 17.6 \times 12{,}000}{9{,}301}$$

$$= 154.7 + 9.1 = 163.8 \text{ psi} < 215.0 \text{ psi} \quad \text{O.K.}$$

In addition to the gravity load case, shear strength must be checked for combined gravity and wind loads. From previous calculations, direct shear force on critical section due to gravity loads is $1.4V_D + 1.7V_L$ = 142.7 kips. The shear force due to wind loads V_W = (4.8 + 4.8)/22.17 = 0.4 kip. Therefore, total factored shear force at the interior column is:

$$V_u = 0.75(1.4V_D + 1.7V_L + 1.7V_w) = 0.75[142.7 + (1.7 \times 0.4)] = 107.5 \text{ kips}$$

As shown above, the unbalanced moment at the first interior support due to gravity loads is the difference between the moments acting on the two sides of the support. However, the unbalanced moment due to wind loads is the sum of the moments acting on the two sides of the support. Therefore, the total unbalanced moment is:

$$M_u = 0.75[(246.0 - 228.4) + 1.7(4.8 + 4.8)] = 25.4 \text{ ft-kips}$$

The combined stress is:

$$v_u = \frac{107{,}500}{922.3} + \frac{0.4 \times 25.4 \times 12{,}000}{9{,}301}$$

$$= 116.6 + 13.1 = 129.7 \text{ psi} < 215.0 \text{ psi} \quad \text{O.K.}$$

Reinforcement Details.

Slab reinforcement must conform to the requirements given in ACI 13.3. The provisions in ACI 13.3.8 must also be satisfied for slabs without beams; included are requirements for structural integrity (ACI 13.3.8.5). Since the wind loads have minimal effect on the design of the slab, the minimum bar lengths given in ACI Figure 13.3.8 govern.

Splice lengths for the flexural bars can be determined in accordance with ACI 12.15. A Class A splice is determined for the bottom bars in the column strip.

Using ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.0

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 0.8 for No. 5 bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 0.75 + \dfrac{0.625}{2} = 1.1 \text{ in. (governs)} \\ \\ \dfrac{12}{2} = 6.0 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index = 0

$$\frac{c + K_{tr}}{d_b} = \frac{1.1 + 0}{0.625} = 1.8 < 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.0 \times 1.0 \times 0.8 \times 1.0}{1.8} = 31.6$$

$$\ell_d = 31.6 \times 0.625 = 19.8 \text{ in.} = 1.7 \text{ ft}$$

Class A splice length = 1.0 × 1.7 = 1.7 ft

Use a 1 ft-9 in. splice length. When computing the development length ℓ_d for the top bars in accordance with ACI 12.2.3, the reinforcement location factor α can be taken equal to one, since less than 12 in. of concrete is cast below the splice location. Reinforcement details for the column strip and middle strip are shown in Figure 3-6.

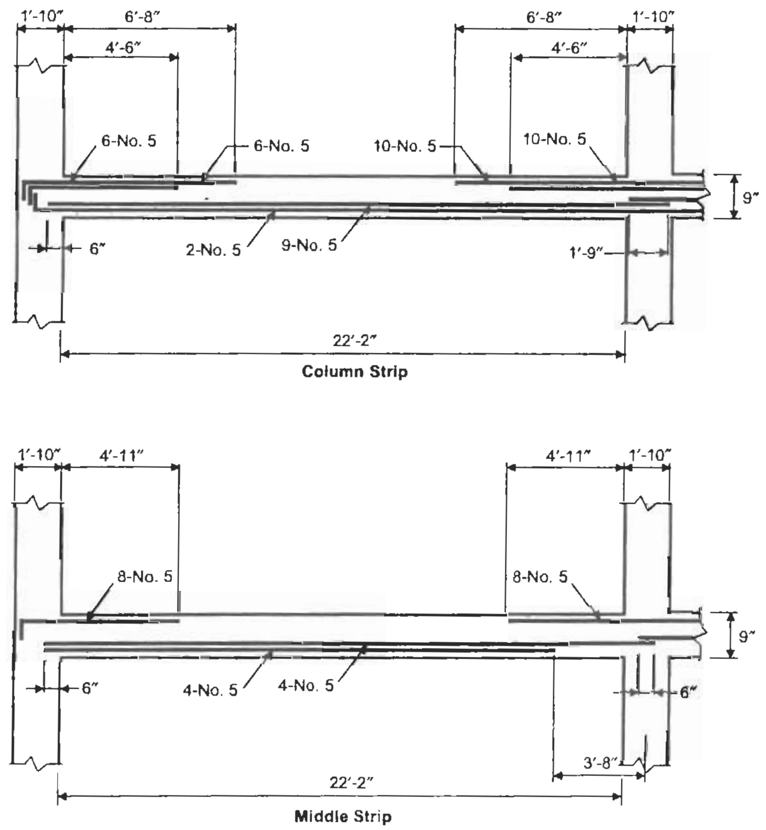

Figure 3-6 Reinforcement Details for Interior Design Strip, Floor Level 4 (SDC A)

3.2.4.3 Design of Column B2

This section outlines the design of column B2 supporting the first floor level. From the analysis it can be shown that reactions on columns due to wind forces in either direction are very small; the shear walls primarily resist the effects from the lateral loads. Therefore, column B2 is designed for the effects of gravity loads only.

Design for axial force.

Service dead and live load axial forces on this member are 714 kips and 103 kips, respectively. Maximum factored axial load = (1.4 × 714) + (1.7 × 103) = 1,175 kips. Based on this factored load combination, a 22 × 22 in. column with 8-No. 10 bars ($\rho_g = 2.10\%$) is adequate for column B2 supporting the first floor level. Slenderness effects need not be considered since P-delta effects were included in the analysis. Also, the provided reinforcement ratio is within the allowable range of 1% and 8% (ACI 10.9.1).

Transverse reinforcement.

Transverse reinforcement requirements must satisfy ACI 7.10.5. With No. 3 lateral ties, the vertical spacing of the ties must not exceed the least of the following:

- 16(smallest longitudinal bar diameter) = 16 × 1.27 = 20.3 in.
- 48(tie bar diameter) = 48 × 0.375 = 18.0 in. (governs)
- Least column dimension = 22 in.

Use No. 3 ties @ 18 in. with the first tie located vertically not more than 18/2 = 9 in. above the top of the slab and not more than 18/2 = 9 in. below the lowest horizontal reinforcement in the slab (ACI 7.10.5.4).

Splice length of longitudinal reinforcement.

ACI 12.17 contains special provisions for splices in columns. It can be shown for this example that the bar stress due to all factored load combinations is compressive. Thus, a compression lap splice computed in accordance with ACI 12.17.2.1 is used.

Compression lap splice length = $0.0005 f_y d_b = 0.0005 \times 60{,}000 \times 1.27 =$ 38.1 in. > 12 in.

Use a 3 ft-3 in. splice length.

Reinforcement details for column B2 are shown in Figure 3-7.

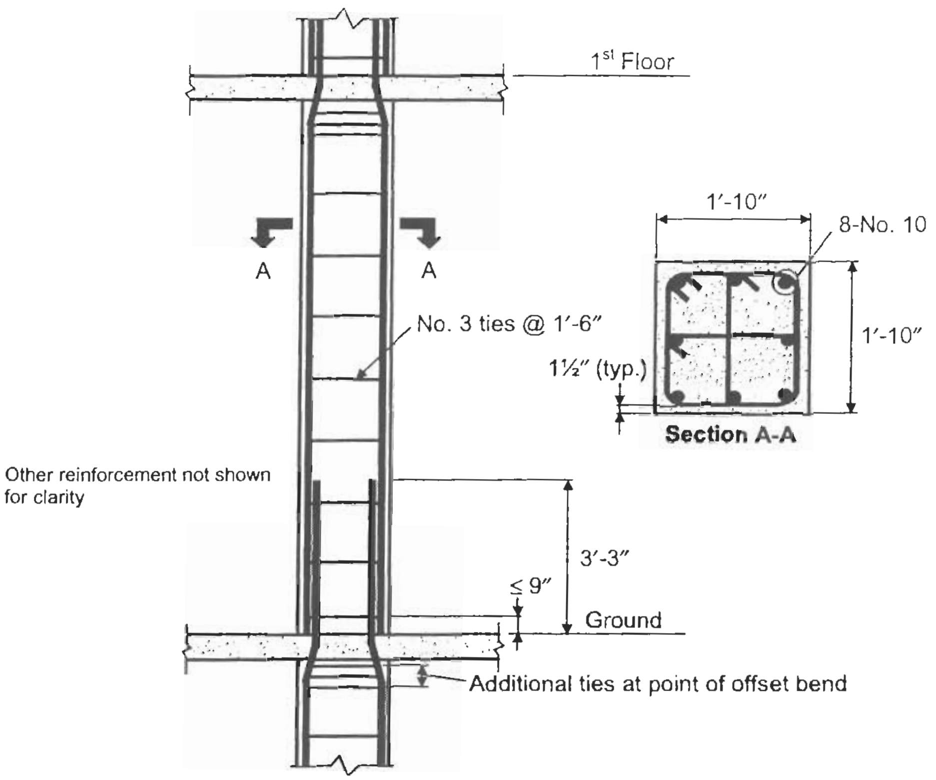

Figure 3-7 Reinforcement Details for Column B2 Supporting the 1st Floor Level (SDC A)

3.2.4.4 Design of Shear Wall on Line 4

Table 3-12 contains a summary of the design axial forces, bending, moments, and shear forces at the base of the wall. Note that wind in the N-S direction causes appreciable reactions in this member; thus, the governing reactions in Table 3-12 are for wind in that direction.

Table 3-12 Summary of Design Axial Forces, Bending Moments, and Shear Forces at Base of Shear Wall on Line 4 (SDC A)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (*D*)	1,549	0	0
Live (*L*)	195	0	0
Wind (*W*)	0	± 12,641	± 222
Load Combinations			
1.4*D* + 1.7*L*	2,500	0	0
0.75(1.4*D* + 1.7*L* + 1.7*W*)	1,875	16,117	283
0.9*D* + 1.3*W*	1,394	-16,433	-289

Design for shear.

The shear strength of the concrete is determined in accordance with ACI 11.10.5 for walls subjected to axial compression:

$$V_c = 2\sqrt{f_c'}hd$$

$$= 2\sqrt{4{,}000} \times 8 \times 220.8 / 1{,}000 = 223.4 \text{ kips}$$

where d is permitted to be taken equal to $0.8\ell_w$ = 0.8 × 276 = 220.8 in. (ACI 11.10.4). The maximum factored shear force is 289 kips from the third load combination (see Table 3-12). Since $\phi V_c = 0.85 \times 223.4 = 189.9 \text{ kips} < V_u = 289 \text{ kips}$, horizontal shear reinforcement shall be provided in accordance with ACI 11.10.9. With 2 layers of No. 4 bars in the web, the required bar spacing is determined by Eq. 11-33:

$$s_2 = \frac{A_v f_y d}{V_s} = \frac{(2 \times 0.20) \times 60 \times 220.8}{(289/0.85) - 223.4} = 45.5 \text{ in.} > 18 \text{ in.}$$

where, according to ACI 11.10.9.3, maximum spacing of horizontal reinforcement shall not exceed $\ell_w/5$ = 276/5 = 55.2 in., $3h$ = 3 × 8 = 24 in., or 18 in. (governs). Try 2-No. 4 horizontal bars @ 18 in.

Ratio of horizontal shear reinforcement ρ_h shall not be less than 0.0025 (ACI 11.10.9.2). For 2-No. 4 horizontal bars spaced at 18 in., the ratio ρ_h of horizontal shear reinforcement area to gross concrete area of vertical section is

$$\rho_h = \frac{2 \times 0.20}{8 \times 18} = 0.0028 > 0.0025 \quad \text{O.K.}$$

Use 2-No. 4 horizontal bars @ 18 in.

The shear strength V_n at any horizontal section must be less than or equal to $10\sqrt{f_c'}hd$ = 1,117 kips (ACI 11.10.3). In this case,

$$V_n = V_c + V_s = 223.4 + \frac{(2 \times 0.20) \times 60 \times 220.8}{18} = 517.8 \text{ kips} < 1{,}117 \text{ kips} \quad \text{O.K.}$$

The ratio of vertical shear reinforcement area to gross concrete area of horizontal section shall not be less than 0.0025 nor the value obtained from Eq. 11-34 (ACI 11.10.9.4):

$$\rho_n = 0.0025 + 0.5\left(2.5 - \frac{h_w}{\ell_w}\right)(\rho_h - 0.0025)$$

$$= 0.0025 + 0.5\left(2.5 - \frac{94}{23}\right)(0.0028 - 0.0025) = 0.0023 < 0.0025$$

Thus, $\rho_n = 0.0025$.

According to ACI 11.10.9.5, spacing of vertical shear reinforcement shall not exceed $\ell_w/3 = 276/3 = 92.0$ in., $3h = 3 \times 8 = 24$ in., or 18 in. (governs). For 2-No. 4 vertical bars spaced at 18 in.:

$$\rho_n = \frac{2\times0.20}{8\times18} = 0.0028 > 0.0025 \quad \text{O.K.}$$

Use 2-No. 4 vertical bars @ 18 in.

The provided vertical and horizontal reinforcement satisfy the requirements of ACI 14.3.2 and 14.3.3 for minimum ratio of vertical and horizontal reinforcement to gross concrete area, respectively, and ACI 14.3.5 for maximum bar spacing.

Design for axial force and bending.

ACI 14.4 requires that walls subjected to axial load or combined flexure and axial load shall be designed as compression members in accordance with ACI 10.2, 10.3, 10.10 through 10.14, 10.17, 14.2, and 14.3 unless the empirical design method of ACI 14.5 or the alternative design method of ACI 14.8 can be used. Clearly, both of these methods cannot be applied in this case, and the wall is designed in accordance with ACI 14.4.

Preliminary design indicates that 2-No. 4 vertical bars @ 18 in. in the web is not sufficient for the load combinations in Table 3-12. Figure 3-8 contains the interaction diagram of the wall reinforced with 2-No. 4 vertical bars @ 12 in. and 4-No. 10 bars at each end of the wall. As seen from the figure, the wall is adequate for the load combinations in Table 3-12.

Splice length of reinforcement.

Class B lap splices are utilized for the vertical bars in the wall. No splices are required for the No. 4 horizontal bars, since full length bars weigh approximately $0.668 \times 23 = 15.4$ lbs. and are easily installed.

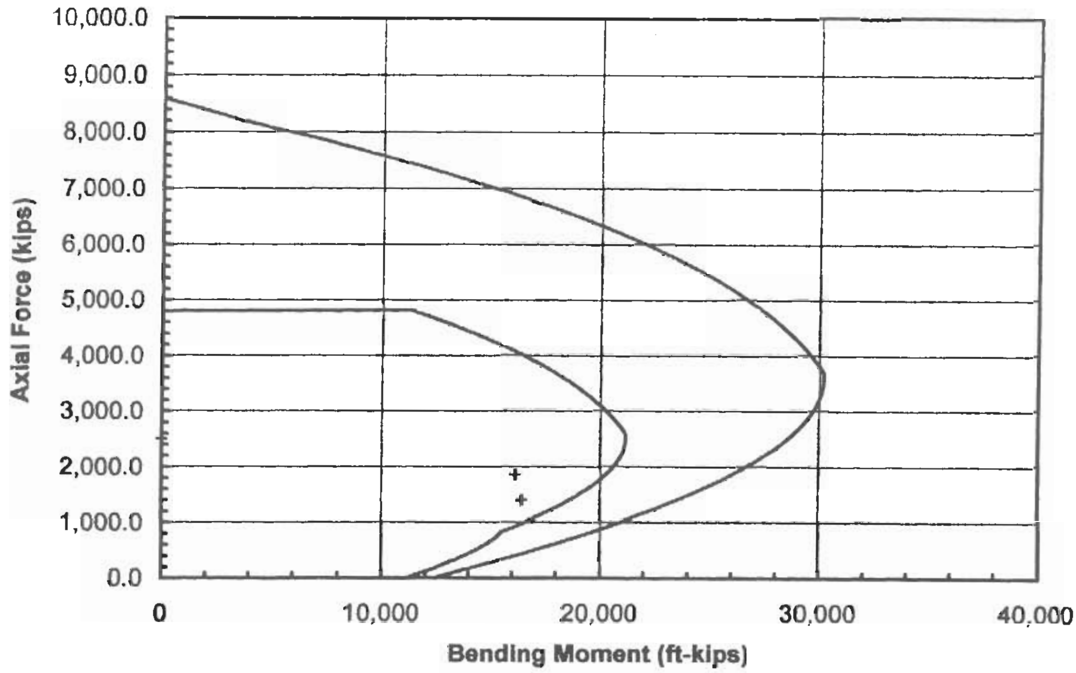

Figure 3-8 Design and Nominal Strength Interaction Diagrams for the Shear Wall Along Line 4 (SDC A)

For the No. 10 vertical bars:

$$\frac{\ell_d}{d_b} = \frac{3}{40}\frac{f_y}{\sqrt{f_c'}}\frac{\alpha\beta\gamma\lambda}{\left(\frac{c+K_{tr}}{d_b}\right)}$$

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 7 and larger bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 0.75 + 0.5 + \frac{1.27}{2} = 1.9 \text{ in.} \quad \text{(governs)} \\ \frac{1}{2} \times 5.2 = 2.6 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index = 0

$$\frac{c+K_{tr}}{d_b}=\frac{1.9+0}{1.27}=1.5<2.5$$

Therefore,

$$\frac{\ell_d}{d_b}=\frac{3}{40}\times\frac{60{,}000}{\sqrt{4{,}000}}\times\frac{1.0\times1.0\times1.0\times1.0}{1.5}=47.4$$

$$\ell_d=47.4\times1.27=60.2\text{ in.}=5.0\text{ ft}$$

Class B splice length = $1.3\ell_d$ = 6.5 ft

Use a 6 ft-6 in. splice length for the No. 10 bars. In lieu of lap splices for these large bars, mechanical or welded splices can be used (ACI 12.14.3).

For the No. 4 bars:

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 0.8 for No. 6 and smaller bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$=\begin{cases}0.75+0.5+\dfrac{0.5}{2}=1.5\text{ in.} & \text{(governs)}\\ \\ \dfrac{1}{2}\times12=6.0\text{ in.}\end{cases}$$

K_{tr} = transverse reinforcement index = 0

$$\frac{c+K_{tr}}{d_b}=\frac{1.5+0}{0.5}=3.0>2.5,\ \text{use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.0 \times 1.0 \times 0.8 \times 1.0}{2.5} = 22.8$$

$$\ell_d = 22.8 \times 0.5 = 11.4 \text{ in.} < 12 \text{ in., use } 12 \text{ in.}$$

$$\text{Class B splice length} = 1.3\ell_d = 1.3 \text{ ft}$$

Use a 1 ft-4 in. splice length for the No. 4 bars.

The No. 4 horizontal bars are developed by providing standard 90-degree hooks at the ends of the bars.

Reinforcement details for the shear wall along line 4 are shown in Figure 3-9.

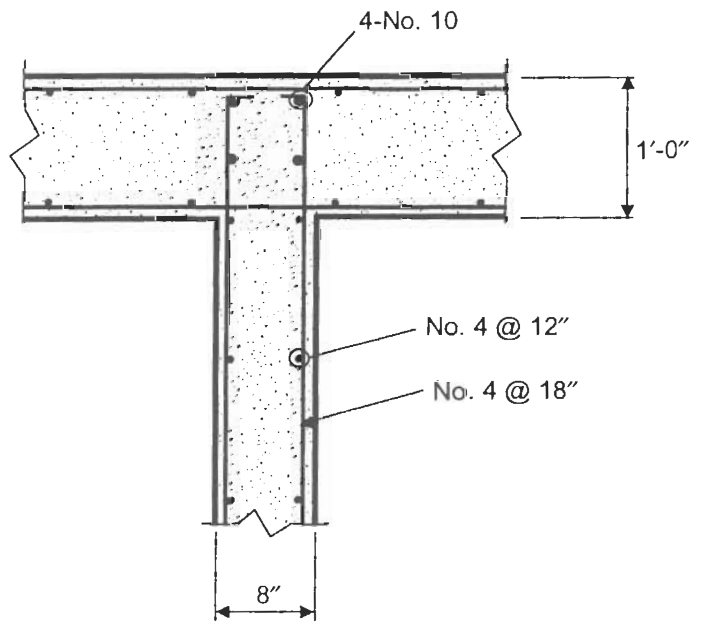

Figure 3-9 Reinforcement Details for Shear Wall Along Line 4 (SDC A)

3.3 DESIGN FOR SDC B

To illustrate the design requirements for Seismic Design Category (SDC) B, the residential building in Figure 3-1 is assumed to be located in Atlanta, GA. Typical structural members are designed and detailed for combined effects of gravity, wind, and seismic forces.

3.3.1 Design Data

- Building Location: Atlanta, GA (zip code 30350)

- Material Properties

  Concrete: $f'_c = 4{,}000$ psi, $w_c = 150$ pcf
  Reinforcement: $f_y = 60{,}000$ psi

- Service Loads

  Live loads: roof = 20 psf
  floor = 50 psf

  Superimposed dead loads: roof = 10 psf + 200 kips for penthouse
  floor = 30 psf (20 psf permanent partitions + 10 psf ceiling, etc.)

- Seismic Design Data

  For zip code 30350: $S_S = 0.276g, S_1 = 0.117g$ [3.1]
  Site Class C (very dense soil / soft rock soil profile; IBC Table 1615.1.1)
  For Category (Seismic Use Group) I occupancy, $I_E = 1.0$ (IBC Table 1604.5)

- Wind Design Data

  Basic wind speed = 90 mph (IBC Figure 1609)
  Exposure B (IBC 1609.4)
  For Category I occupancy, $I_W = 1.0$ (IBC Table 1604.5)

- Member Dimensions

  Slab: 9 in.
  Columns: 22 × 22 in.
  Wall thickness: 8 in. in N-S direction, 12 in. in E-W direction

3.3.2 Seismic Load Analysis

3.3.2.1 Seismic Design Category (SDC)

Analysis procedures for seismic design are given in IBC 1616.6. The appropriate procedure to use depends on the Seismic Design Category (SDC), which is determined in accordance with IBC 1616.3. Structures are assigned to a SDC based on their Seismic Use Group and the design spectral response acceleration parameters S_{DS} and S_{D1}.

These parameters can be computed from Eqs. 16-18 and 16-19 in IBC 1615.1.3 or can be obtained from the provisions of IBC 1615.2.5 where site-specific procedures are used as required or permitted by IBC 1615.

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1-second period S_{M1} are determined from IBC Eqs. 16-16 and 16-17, respectively:

$$S_{MS} = F_a S_S = 1.2 \times 0.276 = 0.33g$$

$$S_{M1} = F_v S_1 = 1.68 \times 0.117 = 0.20g$$

where F_a and F_v are the values of site coefficients as a function of site class and of mapped spectral response acceleration at short periods S_S and at 1-second period S_1, respectively. The values of these coefficients for Site Class C in the equations above are contained in IBC Table 1615.1.2(1) for F_a and Table 1615.1.2(2) for F_v. Straight-line interpolation was used to determine F_v.

Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from Eqs. 16-18 and 16-19:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 0.33 = 0.22g$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.20 = 0.13g$$

Tables 1616.3(1) and 1616.3(2) are utilized to determine the SDC. From Table 1616.3(1) for Seismic Use Group I and S_{DS} = 0.22*g*, the SDC is B. Similarly, from Table 1616.3(2), the SDC is B for $S_{D1} = 0.13g$. Thus, the SDC is B for this building.

3.3.2.2 Seismic Forces

According to IBC 1616.6.2, the equivalent lateral force procedure in IBC 1617.4 may be used to compute the seismic base shear V for structures assigned to SDC B. In a given direction, V is determined from Eq. 16-34:

$$V = C_s W$$

where C_s is the seismic response coefficient determined in accordance with IBC 1617.4.1.1 and W is the effective weight of the structure defined in IBC 1617.4. For the member sizes and superimposed dead loads given above, W = 14,836 kips (see Table 3-13 below).

In both directions, a shear wall-frame interactive system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls is utilized, which is permitted for structures assigned to SDC B without any limitations (see IBC Table 1617.6 and IBC 1910.3). The response modification coefficient R = 5.5 and the deflection amplification factor $C_d = 5$ (IBC Table 1617.6).

Approximate period (T_a). The fundamental period of the building T is determined in accordance with IBC 1617.4.2. In lieu of a more exact analysis, an approximate fundamental period T_a is computed from Eq. 16-39:

Building height $h_n = 94$ ft

Building period coefficient $C_T = 0.02$

Period $T_a = C_T (h_n)^{3/4} = 0.020 \times (94)^{3/4} = 0.60$ sec

No further refinement of the period is made as permitted by IBC 1617.4.2.

Seismic base shear (V). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.13}{\left(\frac{5.5}{1.0}\right) \times 0.60} = 0.039$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{0.22}{\left(\frac{5.5}{1.0}\right)} = 0.040$$

Also, C_s must not be less than the following value from Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 0.22 \times 1.0 = 0.010$$

Thus, the value of C_s from Eq. 16-36 governs so that the base shear V in the N-S and E-W directions is:

$$V = C_s W = 0.039 \times 14{,}836 = 579 \text{ kips}$$

Vertical distribution of seismic forces. The total base shear is distributed over the height of the building in conformance with Eqs. 16-41 and 16-42:

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

where F_x is the lateral force induced at level x, w_x and w_i are the portions of W assigned to levels x or i, and k is the distribution exponent defined in IBC 1617.4.3. For $T = 0.60$ sec, $k = 1.05$ from linear interpolation. The lateral forces F_x and the story shears V_x are contained in Table 3-13.

Table 3-13 Seismic Forces and Story Shears (SDC B)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
9	1,533	94	180,853	107	107
8	1,656	84	173,602	103	210
7	1,656	74	151,969	90	300
6	1,656	64	130,482	78	378
5	1,656	54	109,163	65	443
4	1,656	44	88,041	52	495
3	1,656	34	67,160	40	535
2	1,656	24	46,589	28	563
1	1,711	14	27,333	16	579
Σ	14,836		975,192	579	

3.3.2.3 Method of Analysis

A three-dimensional analysis of the building was performed in both the N-S and E-W directions for the seismic forces contained in Table 3-13 using SAP2000 [3.4]. In the model, rigid diaphragms were assigned at each floor level, and rigid-end offsets were defined at the ends of the horizontal members so that results were automatically obtained at the faces of the supports. The stiffness properties of the members were input assuming cracked sections. In lieu of a more accurate analysis, the following cracked section properties were used:

- Slabs: $I_{eff} = I_g$
- Columns: $I_{eff} = 0.7 I_g$

- Shear walls: $I_{eff} = 0.7I_g$

where I_g is the gross moment of inertia of the section. P-delta effects were also considered in the analysis. As discussed in Section 3.2.4 of this publication, the effective beam width of the slab was set equal to one-fourth and one-eighth of the bay width in the direction perpendicular to the direction of analysis for interior and exterior frames, respectively. These effective widths include a reduction in stiffness for cracking.

In accordance with IBC 1617.4.4.4, the center of mass was displaced each way from its actual location a distance equal to 5 percent of the building dimension perpendicular to the applied forces to account for accidental torsion in seismic design. Torsional effects need not be amplified, since the building is assigned to SDC B (IBC 1617.4.4.5).

3.3.2.4 Story Drift and P-delta Effects

Story drift determination. Table 3-14 contains the displacements δ_{xe} in the N-S direction obtained from the 3-D static, elastic analysis using the design seismic forces, including accidental torsional effects. The table also contains the design earthquake displacement δ_x computed by IBC Eq. 16-46:

$$\delta_x = \frac{C_d \delta_{xe}}{I_E}$$

where C_d is the deflection amplification factor in Table 1617.6, which depends on the seismic-force-resisting system. As noted above, C_d is equal to 5 for this system.

Table 3-14 Lateral Displacements and Interstory Drifts due to Seismic Forces in N-S Direction (SDC B)

Story	δ_{xe} (in.)	δ_x (in.)	Δ (in.)
9	0.41	2.05	0.25
8	0.36	1.80	0.25
7	0.31	1.55	0.25
6	0.26	1.30	0.25
5	0.21	1.05	0.25
4	0.16	0.80	0.25
3	0.11	0.55	0.20
2	0.07	0.35	0.20
1	0.03	0.15	0.15

The interstory drifts Δ computed from the δ_x are also contained in the table. For this structure that does not have plan irregularity Type 1a or 1b of Table 1616.5.1, the drift at story level x is determined by subtracting the design earthquake displacement at the center of mass at the bottom of the story from the design earthquake displacement at the center of mass at the top of the story (IBC 1617.4.6.1):

$$\Delta = \delta_x - \delta_{x-1}$$

The design story drifts Δ must not exceed the allowable story drift Δ_a from Table 1617.3 (IBC 1617.3). For Seismic Use Group I, $\Delta_a = 0.020h_{sx}$ where h_{sx} is the story height below level x. Thus, for the 10-ft story heights, Δ_a = 0.020 × 10 × 12 = 2.40 in., and for the 14-ft story height at the first level, Δ_a = 3.36 in. It is evident from Table 3-14 that for all stories, the lateral drifts obtained from the prescribed lateral forces in the N-S direction are less than the limiting values.

Similar calculations for seismic forces in the E-W direction also show that the lateral drifts are less than the allowable values.

P-delta effects. As noted above, P-delta effects were automatically considered in the analysis using SAP2000. The provisions of IBC 1617.4.6.2 are illustrated in Sections 3.5.2.4 and 3.6.2.4 of this publication for SDC D.

3.3.3 Wind Load Analysis

3.3.3.1 Wind Forces

According to IBC 1609.1.1, wind forces shall be determined in accordance with Section 6 of ASCE 7 [3.2]. Since the building has a mean roof height greater than 30 ft, the simplified procedure (Method 1) given in ASCE 6.4 cannot be used to determine the wind forces. Similarly, the simplified procedure of IBC 1609.6 must not be used, since the building is taller than 60 ft. The analytical procedure (Method 2) of ASCE 6.5 may be used to determine the wind forces.

Details on how to compute the wind forces in both the N-S and E-W directions are given in Section 3.2.3 of this publication. A summary of the design wind forces in both directions is contained in Table 3-15. Once again it is important to note that the wind directionality factor K_d has been taken equal to 1.0 (see Exception 1 in IBC 1605.2.1).

3.3.3.2 Method of Analysis

Similar to the seismic analysis, a three-dimensional analysis of the building was performed in the N-S and E-W directions for the wind forces contained in Table 3-15 using SAP2000. The modeling assumptions utilized for the seismic analysis were also used for the wind analysis.

According to ASCE 6.5.12.3, main wind-force-resisting systems of buildings with mean roof height h greater than 60 ft must be designed for the full and partial wind load cases of Figure 6-9 (Cases 1 through 4). These four cases were considered in the three-dimensional analysis.

Table 3-15 Design Wind Forces in N-S and E-W Directions (V = 90 mph)

Level	Height above ground level, z (ft)	Total Design Wind Force N-S (kips)	Total Design Wind Force E-W (kips)
9	94	16.2	6.3
8	84	31.8	12.3
7	74	31.2	12.0
6	64	30.4	11.7
5	54	29.5	11.3
4	44	28.6	10.8
3	34	27.4	10.3
2	24	26.0	9.6
1	14	28.9	10.5
	Σ	250.0	94.8

3.3.4 Design for Combined Load Effects

3.3.4.1 Load Combinations

Basic load combinations for strength design are given in IBC 1605.2.1. As noted above, the first exception in this section requires that the non-seismic load combinations of ACI 9.2 be used for concrete structures. Thus, the following load combinations are utilized in the design of the structural members:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)
4. $1.2D + f_1 L + 1.0E$ (Formula 16-5)
5. $0.9D + 1.0E$ (Formula 16-6)

where D, L, W, and E are the effects due to dead, live, wind, and seismic loads, respectively, and f_1 is equal to either 1.0 for places of public assembly, for live loads in excess of 100 psf, and for parking garage live load, or is equal to 0.5 for other live loads.

The seismic load effect E for use in Formula 16-5, which is the combined effect of horizontal and vertical earthquake-induced forces, is computed from Eq. 16-28 where the effects of gravity and seismic ground motion are additive:

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of horizontal seismic forces

ρ = redundancy coefficient

= 1.0 for structures assigned to SDC A, B, or C (IBC 1617.2.1)

Similarly, E for use in Formula 16-6 is computed from Eq. 16-29 where the effects of gravity and seismic ground motion counteract:

$$E = \rho Q_E - 0.2 S_{DS} D$$

Substituting $S_{DS} = 0.22g$ and $\rho = 1.0$ into the equations for E, and then substituting E along with $f_1 = 0.5$ into load combinations 4 and 5 above results in the following:

4a. $1.2D + 0.5L + 1.0Q_E + (0.2 \times 0.22)D = 1.24D + 0.5L + Q_E$

4b. $1.2D + 0.5L + 1.0Q_E - (0.2 \times 0.22)D = 1.16D + 0.5L + Q_E$

5a. $0.9D + 1.0Q_E + (0.2 \times 0.22)D = 0.94D + Q_E$

5b. $0.9D + 1.0Q_E - (0.2 \times 0.22)D = 0.86D + Q_E$

Values of Q_E are obtained from the structural analysis for both sidesway to the left and to the right based on the code-prescribed horizontal seismic forces acting on the building. Also, the special seismic load combinations in IBC 1605.4 are not applicable in this example.

3.3.4.2 Slab Design

Preliminary Slab Thickness.

Calculations given in Section 3.2.4.2 of this publication show that a 9-in. thick slab is adequate for serviceability and shear strength requirements for gravity loads. A more refined check for shear strength is made at a later stage for gravity loads and combined gravity and seismic loads.

Design for Flexure.

It was determined in Section 3.2.4.2 of this publication that the Direct Design Method could be utilized to compute the design bending moments in the slab due to gravity loads. Table 3-9 contains a summary of the service gravity load bending moments in the column strip and middle strip of an end span and an interior span.

Comparing the seismic forces in Table 3-13 to the wind forces in Table 3-15, it is clear that seismic effects will govern the design in both directions. Thus, a summary of the governing design bending moments due to gravity and seismic load effects is contained in Table 3-16 at the fourth floor level where the bending moments due to seismic forces are the largest in the slab for the E-W direction.

As in the case for SDC A, gravity load effects govern the flexural design of the slab for SDC B. Therefore, the same slab reinforcement in the column strips and middle strips that was provided for SDC A can be used for SDC B, including the additional reinforcing bars at the supports required for moment transfer as determined in Section 3.2.4.2 of this publication (see Figures 3-4 and 3-6).

Table 3-16 Summary of Slab Design Bending Moments (ft-kips) at the 4th Floor Level (SDC B)

Load Case	Location	End Span Column Strip	End Span Middle Strip	Interior Span Column Strip	Interior Span Middle Strip
Dead (D)	Ext. neg.	-50.3	0		
	Positive	59.9	40.6	40.6	27.1
	Int. neg.	-102.5	-32.9	-94.7	-30.9
Live (L)	Ext. neg.	-12.3	0		
	Positive	14.7	9.9	9.9	6.6
	Int. neg.	-25.1	-8.0	-23.2	-7.6
Seismic (Q_E)	Ext. neg.	**± 14.4**			
	Positive				
	Int. neg.	**± 14.0**		**± 18.0**	
Load Combinations					
1.4D + 1.7L	Ext. neg.	-91.3	0		
	Positive	108.9	73.7	**73.7**	**49.2**
	Int. neg.	-186.2	-59.7	**-172.0**	**-56.2**
1.24D + 0.5L + Q_E	Ext. neg.	-82.9	0		
	Positive	81.6	55.3	**55.3**	**36.9**
	Int. neg.	-153.7	-44.8	**-147.0**	-42.1
0.86D + Q_E	Ext. neg.	-28.9	0		
	Positive	51.5	34.9	34.9	23.3
	Int. neg.	-74.2	-28.3	-63.4	-26.6

Design for Shear.

Shear strength needs to be checked at the slab-column connections at the end support and the first interior support for gravity forces and for gravity plus seismic forces.

End Support – Check for Shear Strength. At this location, the factored shear stress v_u due to gravity loads is the same as that computed for SDC A, i.e., v_u = 184.5 psi < ϕv_c = 215 psi O.K. (see Section 3.2.4.2).

Shear strength must also be checked for combined gravity and seismic loads. Direct shear forces on the critical section are as follows:

$$V_D = w_D(A_t - b_1 b_2) - \frac{M_{1D} - M_{2D}}{\ell_n}$$

$$= 0.143\left[284.2 - \frac{25.875 \times 29.75}{144}\right] - \frac{(0.70 \times 193.3) - (0.26 \times 193.3)}{22.17}$$

$$= 39.9 - 3.8 = 36.1 \text{ kips}$$

$$V_L = w_L(A_t - b_1 b_2) - \frac{M_{1L} - M_{2L}}{\ell_n}$$

$$= 0.035\left[284.2 - \frac{25.875 \times 29.75}{144}\right] - \frac{(0.70 \times 47.3) - (0.26 \times 47.3)}{22.17}$$

$$= 9.8 - 0.9 = 8.9 \text{ kips}$$

$$V_E = \frac{14.4 + 14.0}{22.17} = 1.3 \text{ kips}$$

Therefore, total factored shear force at exterior column is:

$$V_u = 1.24V_D + 0.5V_L + V_E = (1.24 \times 36.1) + (0.5 \times 8.9) + 1.3 = 50.5 \text{ kips}$$

When seismic effects are considered, shear stress computations can be based on the actual unbalanced moment, rather than on the provisions of ACI 13.6.3.6, which, as shown above in Section 3.2.4.2, requires the unbalanced moment to be $0.30M_o$. The actual unbalanced moment at the exterior slab-column connection is 82.9 ft-kips from the second load combination (see Table 3-16). The combined shear stress is:

$$v_u = \frac{50{,}500}{631.6} + \frac{0.38 \times 82.9 \times 12{,}000}{5{,}951}$$

$$= 80.0 + 63.5 = 143.5 \text{ psi} < 215.0 \text{ psi} \quad \text{O.K.}$$

where the section properties of the critical section A_c and J/c were determined in Section 3.2.4.2.

First Interior Support – Check for Shear Strength. At this location, the factored shear stress v_u due to gravity loads is the same as that computed for SDC A, i.e., v_u = 163.8 psi < ϕv_c = 215 psi O.K. (see Section 3.2.4.2).

The shear strength due to gravity plus seismic loads must also be checked. Direct shear forces on the critical section are as follows:

$$V_D = w_D(A_t - b_1 b_2) + \frac{M_{1D} - M_{2D}}{\ell_n}$$

$$= 0.143\left[528 - \frac{29.75^2}{144}\right] + \frac{(0.70\times 193.3) - (0.26\times 193.3)}{22.17}$$

$$= 74.6 + 3.8 = 78.4 \text{ kips}$$

$$V_L = w_L(A_t - b_1 b_2) + \frac{M_{1L} - M_{2L}}{\ell_n}$$

$$= 0.035\left[528 - \frac{29.75^2}{144}\right] + \frac{(0.70\times 47.3) - (0.26\times 47.3)}{22.17}$$

$$= 18.3 + 0.9 = 19.2 \text{ kips}$$

$$V_E = \frac{18.0 + 18.0}{22.17} = 1.6 \text{ kips}$$

Therefore, total factored shear force is:

$$V_u = 1.24V_D + 0.5V_L + V_E = (1.24\times 78.4) + (0.5\times 19.2) + 1.6 = 108.4 \text{ kips}$$

The unbalanced moment at the first interior support due to gravity loads is the difference between the moments acting on the two sides of the support, while the unbalanced moment due to seismic forces is the sum of the moments acting on the two sides of the support:

$$M_u = 1.24[(0.70\times 193.3) - (0.65\times 193.3)] + 0.5[(0.70\times 47.3) - (0.65\times 47.3)]$$

$$+ (18.0 + 14.0)$$

$$= 45.2 \text{ ft - kips}$$

The combined shear stress is:

$$v_u = \frac{108{,}400}{922.3} + \frac{0.4 \times 45.2 \times 12{,}000}{9{,}301}$$

$$= 117.5 + 23.3 = 140.8 \text{ psi} < 215.0 \text{ psi} \quad \text{O.K.}$$

Reinforcement Details.

Slab reinforcement must conform to the requirements given in ACI 13.3. The provisions in ACI 13.3.8 must also be satisfied for slabs without beams; included are requirements for structural integrity (ACI 13.3.8.5).

Bar cutoff points for the top bars in the column strip were computed in accordance with ACI 12.10 through 12.12 based on the third load combination, since this resulted in the longest bar lengths. The minimum bar lengths in ACI Figure 13.3.8 were longer than those computed from analysis, except for the top bars at the first interior support in the end span where a length of $0.32\,\ell_n$ is required (compared to $0.30\,\ell_n$ in Figure 13.3.8). Therefore, the reinforcement details in Figure 3-6 can also be used for SDC B with the one modification noted above.

3.3.4.3 Design of Column B2

Table 3-17 contains a summary of the design axial forces, bending moments, and shear forces on column B2 for gravity and seismic loads.

Table 3-17 Summary of Design Axial Forces, Bending Moments, and Shear Forces on Column B2 between Ground and 1st Floor Levels (SDC B)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	714	0	0
Live (L)*	103	0	0
Seismic (Q_E)	± 3	± 18	± 13
Load Combinations			
$1.4D + 1.7L$	1,175	0	0
$1.24D + 0.5L + Q_E$	940	18	13
$0.86D + Q_E$	611	-18	-13

* Live load reduced per IBC 1607.9

Design for axial force and bending.

Based on the governing load combinations in Table 3-17, a 22 × 22 in. column with 8-No. 10 bars ($\rho_g = 2.10\%$) is adequate for column B2 supporting the first floor level. As noted above, slenderness effects need not be considered since P-delta effects were included in the analysis. Also, the provided reinforcement ratio is within the allowable range of 1% and 8% (ACI 10.9.1).

Transverse reinforcement.

Transverse reinforcement requirements must satisfy ACI 7.10.5. The same transverse reinforcement determined in Section 3.2.4.3 for SDC A can be used for SDC B.

Note that since the clear column height to maximum plan dimension of the column is $[(14 \times 12) - 9]/22 = 7.2 > 5$, the column need not be designed for shear in accordance with ACI 21.10.3 (IBC 1910.3.1).

Reinforcement details for the column are the same as for SDC A (see Figure 3-7).

3.3.4.4 Design of Shear Wall on Line 4

Table 3-18 contains a summary of the design axial forces, bending moments, and shear forces at the base of the wall.

Table 3-18 Summary of Design Axial Forces, Bending Moments, and Shear Forces at Base of Shear Wall on Line 4 (SDC B)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	1,549	0	0
Live (L)	195	0	0
Seismic (Q_E)	0	± 14,465	± 204
Load Combinations			
$1.4D + 1.7L$	2,500	0	0
$1.24D + 0.5L + Q_E$	2,018	14,465	204
$0.86D + Q_E$	1,332	-14,465	-204

Design for shear.

Calculations for shear strength for SDC B are similar to those given in Section 3.2.4.4 for SDC A. It is determined that 2-No. 4 horizontal and vertical bars spaced at 18 in. satisfy the shear strength requirements of ACI 11.10 for SDC B, as well as the requirements of ACI 14.3.2 and 14.3.3 for minimum ratio of vertical and horizontal reinforcement to gross concrete area, respectively, and ACI 14.3.5 for maximum bar spacing.

Design for axial force and bending.

Preliminary design indicates that 2-No. 4 vertical bars @ 18 in. in the web is not sufficient for the load combinations in Table 3-18.

Figure 3-8 contains the interaction diagram of the wall reinforced with 2-No. 4 vertical bars @ 12 in. and 4-No. 10 bars at each end of the wall, which was determined to be adequate for SDC A. As seen from the figure, the wall is also adequate for the load combinations in Table 3-18 for SDC B.

The splice lengths of the reinforcement and the reinforcement details determined for SDC A can be used for SDC B as well (see Figure 3-9).

3.4 DESIGN FOR SDC C

To illustrate the design requirements for Seismic Design Category (SDC) C, the residential building in Figure 3-10 is assumed to be located in New York, NY. Typical structural members are designed and detailed for combined effects of gravity, wind, and seismic forces.

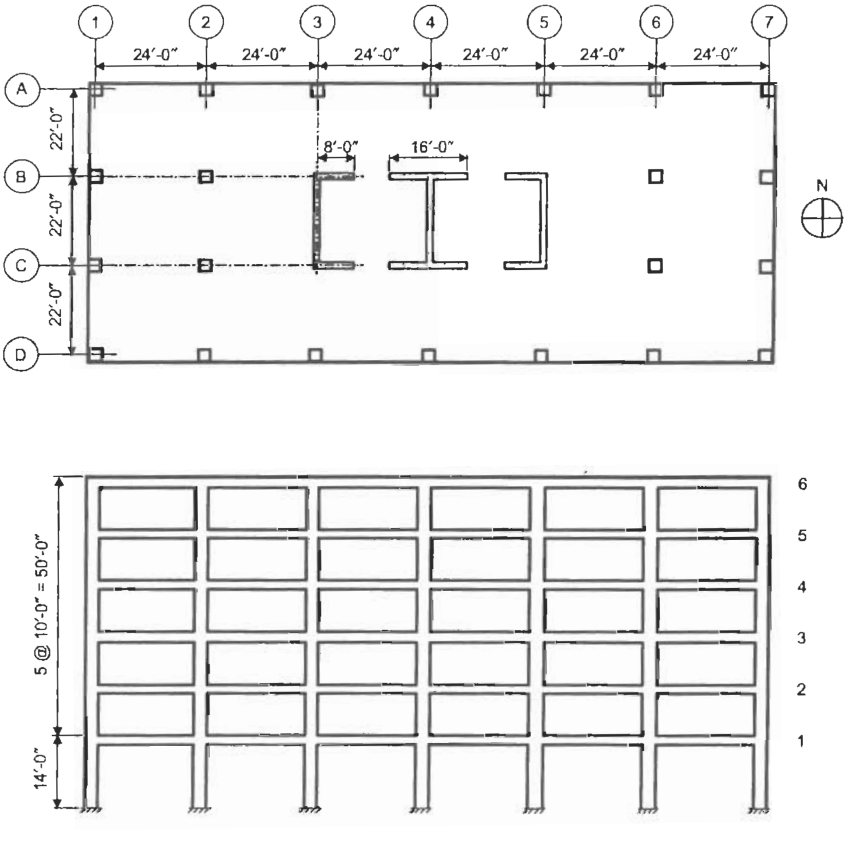

Figure 3-10 Typical Plan and Elevation of Example Building (SDC C)

3.4.1 Design Data

- Building Location: New York, NY (zip code 10013)

- Material Properties

  Concrete: $f'_c = 4{,}000$ psi, $w_c = 150$ pcf
  Reinforcement: $f_y = 60{,}000$ psi

- Service Loads

  Live loads: roof = 20 psf
  floor = 50 psf

  Superimposed dead loads: roof = 10 psf + 200 kips for penthouse
  floor = 30 psf (20 psf permanent partitions + 10 psf ceiling, etc.)

- Seismic Design Data

  For zip code 10013: $S_S = 0.424g, S_1 = 0.094g$ [3.1]
  Site Class D (stiff soil profile; IBC Table 1615.1.1)
  For Category (Seismic Use Group) I occupancy, $I_E = 1.0$ (IBC Table 1604.5)

- Wind Design Data

  Basic wind speed = 110 mph (IBC Figure 1609)
  Exposure B (IBC 1609.4)
  For Category I occupancy, $I_W = 1.0$ (IBC Table 1604.5)

- Member Dimensions

  Slab: 9 in.
  Columns: 24 × 24 in.
  Wall thickness: 8 in. in N-S direction, 12 in. in E-W direction

3.4.2 Seismic Load Analysis

3.4.2.1 Seismic Design Category (SDC)

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1-second period S_{M1} are determined from IBC Eqs. 16-16 and 16-17, respectively:

$$S_{MS} = F_a S_S = 1.46 \times 0.424 = 0.62g$$

$$S_{M1} = F_v S_1 = 2.4 \times 0.094 = 0.23g$$

where F_a and F_v are the values of site coefficients as a function of site class and of mapped spectral response acceleration at short periods S_S and at 1-second period S_1, respectively. The values of these coefficients for Site Class D in the equations above are contained in IBC Table 1615.1.2(1) for F_a and Table 1615.1.2(2) for F_v. Straight-line interpolation was used to determine F_a.

Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from Eqs. 16-18 and 16-19:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 0.62 = 0.41g$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.23 = 0.15g$$

Tables 1616.3(1) and 1616.3(2) are utilized to determine the SDC. From Table 1616.3(1) for Seismic Use Group I and S_{DS} = 0.41g, the SDC is C. Similarly, from Table 1616.3(2), the SDC is C for $S_{D1} = 0.15g$. Thus, the SDC is C for this building.

3.4.2.2 Seismic Forces

According to IBC 1616.6.2, the equivalent lateral force procedure in IBC 1617.4 may be used to compute the seismic base shear V for structures assigned to SDC C. In a given direction, V is determined from Eq. 16-34:

$$V = C_s W$$

where C_s is the seismic response coefficient determined in accordance with IBC 1617.4.1.1 and W is the effective weight of the structure defined in IBC 1617.4. For the member sizes and superimposed dead loads given above, W = 10,040 kips (see Table 3-19 below).

In both directions, a building frame system with ordinary reinforced concrete shear walls is utilized, which is permitted for structures assigned to SDC C without any limitations (see IBC Table 1617.6 and IBC 1910.4). The response modification coefficient $R = 5$ and the deflection amplification factor $C_d = 4.5$ (IBC Table 1617.6).

Approximate period (T_a). The fundamental period of the building T is determined in accordance with IBC 1617.4.2. In lieu of a more exact analysis, an approximate fundamental period T_a is computed from Eq. 16-39:

Building height h_n = 64 ft

Building period coefficient $C_T = 0.02$

Period $T_a = C_T (h_n)^{3/4} = 0.020 \times (64)^{3/4} = 0.45$ sec

Seismic base shear (V). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.15}{\left(\frac{5}{1.0}\right)\times 0.45} = 0.067$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{0.41}{\left(\frac{5}{1.0}\right)} = 0.082$$

Also, C_s must not be less than the following value from Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 0.41 \times 1.0 = 0.018$$

Thus, the value of C_s from Eq. 16-36 governs so that the base shear V in the N-S and E-W directions is:

$$V = C_s W = 0.067 \times 10{,}040 = 673 \text{ kips}$$

Vertical distribution of seismic forces. The total base shear is distributed over the height of the building in conformance with Eqs. 16-41 and 16-42:

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

where F_x is the lateral force induced at level x, w_x and w_i are the portions of W assigned to levels x or i, and k is the distribution exponent defined in IBC 1617.4.3. For $T = 0.45$ sec, $k = 1.0$. The lateral forces F_x and the story shears V_x are contained in Table 3-19.

Table 3-19 Seismic Forces and Story Shears (SDC C)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
6	1,552	64	99,328	173	173
5	1,686	54	91,044	158	331
4	1,686	44	74,184	129	460
3	1,686	34	57,324	100	560
2	1,686	24	40,464	70	630
1	1,744	14	24,416	43	673
Σ	10,040		386,760	673	

3.4.2.3 Method of Analysis

In a building frame system, the shear walls are designed to carry the total lateral forces, and the frames provide support for the gravity forces. However, since the shear walls and beam-column frames form one structural system and will laterally deflect by virtually the same amount, it is important to ensure that compatibility requirements be addressed in the design of the frame members. In general, the elements of the structure that are not designated to be part of the seismic-force-resisting system (SFRS) must be designed and detailed to maintain support of the design dead and live forces when subjected to the expected deformations caused by seismic forces.

Structures that are assigned to SDC C and that have a building frame system as the SFRS are not required to satisfy the deformational compatibility requirements described above (see IBC 1617.6.4). However, it is prudent to satisfy the provisions of IBC 1617.6.4.3 for such structures, considering the important function frame members have when subjected to earthquake-induced displacements. Application of these provisions is illustrated below for the example building.

The reactions in members that are part of the SFRS were determined from a three-dimensional analysis of a structural model comprised of the shear walls subjected to the seismic forces in Table 3-19. In accordance with IBC 1617.4.4.4, the center of mass was displaced each way from its actual location a distance equal to 5 percent of the building dimension perpendicular to the applied forces to account for accidental torsion. Torsional effects need not be amplified, since the building does not possess Type 1a or 1b plan torsional irregularities as defined in Table 1616.5.1 (IBC 1617.4.4.5). Also, 70% of the gross section properties of the walls were used to account for cracking. P-delta effects were also considered in the analysis. The deflections δ_{xe} in the N-S direction from this analysis are contained in Table 3-20.

IBC 1617.6.4.3 requires that reinforced concrete members not designed as part of the SFRS comply with ACI 21.9. According to ACI 21.9.1, such members are to be detailed according to ACI 21.9.2 or 21.9.3 depending on the magnitude of bending moments in those members when subjected to the design displacements δ_x where $\delta_x = C_d \delta_{xe} / I_E$

and δ_{xe} are the displacements calculated by an elastic analysis of the SFRS subjected to the code-prescribed seismic forces. The approximate method outlined in Reference 3.8 can be used to determine the reactions in these members due to the earthquake-induced displacements. In this method, the required member force F_M induced by δ_x can be determined from the following equation:

$$F_M = C_d \left(\frac{\delta_x}{\delta'_x} \right) F'_M$$

where F'_M is the member force due to the code-prescribed seismic forces applied to the entire structure (which includes stiffening effects of members that are not part of the SFRS), $\delta'_x = C_d \delta'_{xe}$, and δ'_{xe} are the corresponding displacements of the entire structure. A three-dimensional analysis similar to the one described above was performed on the entire structure subjected to the seismic forces contained in Table 3-19. As discussed in Section 3.2.4 of this publication, the effective beam width of the slab was set equal to one-fourth and one-eighth of the bay width in the direction perpendicular to the direction of analysis for interior and exterior frames, respectively. These effective widths include a reduction in stiffness for cracking. An effective moment of inertia equal to 70% of the gross moment of inertia was taken for the columns. The displacements δ'_{xe} determined from this analysis are also contained in Table 3-20.

Table 3-20 Lateral Displacements (in.) due to Seismic Forces in N-S Direction (SDC C)

Story	δ_{xe} (in.)	δ_x (in.)	Δ (in.)	δ'_{xe} (in.)	δ'_x (in.)	δ_x / δ'_x
6	0.32	1.44	0.27	0.25	1.13	1.27
5	0.26	1.17	0.27	0.21	0.95	1.23
4	0.20	0.90	0.27	0.16	0.72	1.25
3	0.14	0.63	0.22	0.12	0.54	1.17
2	0.09	0.41	0.23	0.07	0.32	1.28
1	0.04	0.18	0.18	0.03	0.14	1.29

3.4.2.4 Story Drift and P-delta Effects

Story drift determination. Table 3-20 contains the displacements δ_{xe} of the SFRS in the N-S direction obtained from the 3-D static, elastic analysis using the design seismic forces, including accidental torsional effects, and the design earthquake displacement $\boldsymbol{\delta_x}$ computed by IBC Eq. 16-46. As noted above, C_d is equal to 4.5 for this system. The interstory drifts $\boldsymbol{\Delta}$ computed from the δ_x are also contained in the table.

The design story drifts Δ must not exceed the allowable story drift Δ_a from Table 1617.3 (IBC 1617.3). For Seismic Use Group I, $\Delta_a = 0.020 h_{sx}$ where h_{sx} is the story height below level x. Thus, for the 10-ft story heights, $\Delta_a = 0.020 \times 10 \times 12 = 2.40$ in., and for the 14-ft story height at the first level, $\Delta_a = 3.36$ in. It is evident from Table 3-20 that for all stories, the lateral drifts obtained from the prescribed lateral forces in the N-S direction are less than the limiting values.

Similar calculations for seismic forces in the E-W direction also show that the lateral drifts are less than the allowable values.

P-delta effects. As noted above, P-delta effects were automatically considered in the analysis using SAP2000. The provisions of IBC 1617.4.6.2 are illustrated in Sections 3.5.2.4 and 3.6.2.4 of this publication for SDC D.

3.4.3 Wind Load Analysis

3.4.3.1 Wind Forces

According to IBC 1609.1.1, wind forces shall be determined in accordance with Section 6 of ASCE 7 [3.2]. Since the building has a mean roof height greater than 30 ft, the simplified procedure (Method 1) given in ASCE 6.4 cannot be used to determine the wind forces. Similarly, the simplified procedure of IBC 1609.6 must not be used, since the building is taller than 60 ft. The analytical procedure (Method 2) of ASCE 6.5 may be used to determine the wind forces.

Details on how to compute the wind forces are given in Section 3.2.3 of this publication. A summary of the design wind forces in both directions at all floor levels is contained in Table 3-21. Once again it is important to note that the wind directionality factor K_d has been taken equal to 1.0 (see Exception 1 in IBC 1605.2.1).

Table 3-21 Design Wind Forces in N-S and E-W Directions (V = 110 mph)

Level	Height above ground level, z (ft)	Total Design Wind Force N-S (kips)	Total Design Wind Force E-W (kips)
6	64	21.7	8.5
5	54	42.1	16.4
4	44	40.7	15.7
3	34	39.0	14.9
2	24	36.9	13.9
1	14	37.9	15.3
	Σ	218.3	84.7

3.4.3.2 Method of Analysis

A three-dimensional analysis of the building was performed in the N-S and E-W directions for the wind forces contained in Table 3-21 using SAP2000 [3.4]. The modeling assumptions utilized for the seismic analysis were also used for the wind analysis.

According to ASCE 6.5.12.3, main wind-force-resisting systems of buildings with mean roof height h greater than 60 ft must be designed for the full and partial wind load cases of Figure 6-9 (Cases 1 through 4). These four cases were considered in the three-dimensional analysis.

3.4.4 Design for Combined Load Effects

3.4.4.1 Load Combinations

Basic load combinations for strength design are given in IBC 1605.2.1. As noted above, the first exception in this section requires that the non-seismic load combinations of ACI 9.2 be used for concrete structures. Thus, the following load combinations are applicable to structural members that are part of the SFRS:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)
4. $1.2D + f_1 L + 1.0E$ (Formula 16-5)
5. $0.9D + 1.0E$ (Formula 16-6)

where D, L, W, and E are the effects due to dead, live, wind, and seismic loads, respectively, and f_1 is equal to either 1.0 for places of public assembly, for live loads in excess of 100 psf, and for parking garage live load, or is equal to 0.5 for other live loads.

The seismic load effect E for use in Formula 16-5, which is the combined effect of horizontal and vertical earthquake-induced forces, is computed from Eq. 16-28 where the effects of gravity and seismic ground motion are additive:

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of horizontal seismic forces
ρ = redundancy coefficient
= 1.0 for structures assigned to SDC A, B, or C (IBC 1617.2.1)

Similarly, E for use in Formula 16-6 is computed from Eq. 16-29 where the effects of gravity and seismic ground motion counteract:

$$E = \rho Q_E - 0.2 S_{DS} D$$

Substituting S_{DS} = 0.41g and ρ = 1.0 into the equations for E, and then substituting E along with f_1 = 0.5 into load combinations 4 and 5 above results in the following:

4a. $1.2D + 0.5L + 1.0Q_E + (0.2\times 0.41)D = 1.28D + 0.5L + Q_E$

4b. $1.2D + 0.5L + 1.0Q_E - (0.2\times 0.41)D = 1.12D + 0.5L + Q_E$

5a. $0.9D + 1.0Q_E + (0.2\times 0.41)D = 0.98D + Q_E$

5b. $0.9D + 1.0Q_E - (0.2\times 0.41)D = 0.82D + Q_E$

Values of Q_E are obtained from the structural analysis for both sidesway to the left and to the right based on the code-prescribed horizontal seismic forces acting on the building.

The exception in IBC 1617.6.4.3 requires that reinforced concrete members that are not part of the SFRS comply with ACI 21.9. The load combinations for these members are (ACI 21.9.2):

- $1.05D + 1.28L + E$
- $0.9D + E$

where the values of E are the forces in the frame members due to the lateral displacements δ_x being applied to the frames. When the approximate method outlined above is utilized, $E = F_M$.

Also, the special seismic load combinations in IBC 1605.4 are not applicable in this example.

3.4.4.2 Slab Design

Preliminary Slab Thickness.

As noted above, the flat plate is not part of the SFRS. Thus, the slab must satisfy the requirements of ACI 21.9 for frame members not proportioned to resist forces induced by earthquake motions. Since factored gravity axial forces on the slab are less than $A_g f'_c/10$, either ACI 21.9.2 or 21.9.3 apply, depending on whether or not the earthquake-induced bending moments and shear forces due to the design displacements exceed the design moment and shear strength of the slab. In the Supplement to the 2000 IBC [3.9], ACI 21.9.2.1 and 21.9.3.2 have been modified to include specific requirements for two-way slabs. In particular, stirrups need not be provided in two-way slabs with column-line beams that are the same thickness as the slab. Shear reinforcement in the slab is to be provided in accordance with ACI 11.12 where required.

Calculations given in Section 3.2.4.2 of this publication show that a 9-in. thick slab and 22-in. square columns are adequate for serviceability and shear strength requirements for gravity loads. A more refined check for shear strength is made at a later stage in this example for a 9-in. thick slab and 24-in. square columns for effects from gravity loads and combined effects from gravity loads and earthquake-induced displacements.

Design for Flexure.

It was determined in Section 3.2.4.2 of this publication that the Direct Design Method can be utilized to compute the design bending moments in the slab due to gravity loads. Table 3-22 contains a summary of the service gravity load bending moments in the column strip and middle strip of an end span and an interior span of an interior N-S design strip. The total static service dead load and live load moments for spans in an interior design strip in the N-S direction are computed as follows:

$$M_o = \frac{w\ell_n^2 \ell_2}{8}$$

$$(M_o)_D = \frac{0.143 \times 20^2 \times 24}{8} = 171.6 \text{ ft-kips}$$

$$(M_o)_L = \frac{0.035 \times 20^2 \times 24}{8} = 42.0 \text{ ft-kips}$$

Table 3-22 Service Dead and Live Load Bending Moments (ft-kips) in N-S Interior Design Strip (SDC C)

Location		Moment	M_D	M_L
End Span				
Column Strip	Exterior Negative	$0.26M_o$	-44.6	-10.9
	Positive	$0.31M_o$	53.2	13.0
	Interior Negative	$0.53M_o$	-91.0	-22.3
Middle Strip	Exterior Negative	0	0	0
	Positive	$0.21M_o$	36.0	8.8
	Interior Negative	$0.17M_o$	-29.2	-7.1
Interior Span				
Column Strip	Positive	$0.21M_o$	36.0	8.8
	Interior Negative	$0.49M_o$	-84.1	-20.6
Middle Strip	Positive	$0.14M_o$	24.0	5.9
	Interior Negative	$0.16M_o$	-27.5	-6.7

The reactions in the slab due to seismic displacements δ_x are determined using the approximate method outlined above. Table 3-23 contains the bending moments M'_E in the slab at the fourth floor level obtained from a 3-D analysis of the shear walls and frames acting together subjected to the seismic forces in Table 3-19. The table also contains the approximate compatibility bending moments $M_E = C_d(\delta_x / \delta'_x)M'_E$, where

C_d = 4.5 and the ratio (δ_x / δ'_x) at the fourth floor level is equal to 1.25 (see Table 3-20).

Table 3-23 Compatibility Bending Moments (ft-kips) in N-S Interior Design Strip (SDC C)

Location		M'_E	M_E
End Span	Exterior Negative	± 11.8	± 66.4
	Interior Negative	± 11.6	± 65.3
Interior Span	Interior Negative	± 11.4	± 64.1

A summary of the governing design bending moments at the fourth floor level is contained in Table 3-24.

Table 3-24 Summary of Slab Design Bending Moments (ft-kips) at Floor Level 4 (SDC C)

Load Case	Location	End Span		Interior Span	
		Column Strip	Middle Strip	Column Strip	Middle Strip
Dead (*D*)	Ext. neg.	-44.6	0		
	Positive	53.2	36.0	36.0	24.0
	Int. neg.	-91.0	-29.2	-84.1	-27.5
Live (*L*)	Ext. neg.	-10.9	0		
	Positive	13.0	8.8	8.8	5.9
	Int. neg.	-22.3	-7.1	-20.6	-6.7
Seismic (*E*)	Ext. neg.	± 66.4			
	Positive				
	Int. neg.	± 65.3		± 64.1	
Load Combinations					
1.4*D* + 1.7*L*	Ext. neg.	-81.0	0		
	Positive	96.6	65.4	65.4	43.6
	Int. neg.	-165.3	-53.0	-152.8	-49.9
1.05*D* + 1.28*L* + *E*	Ext. neg.	-127.2	0		
	Positive	72.5	49.1	49.1	32.8
	Int. neg.	-189.4	-39.8	-178.8	-37.5
0.9*D* + *E*	Ext. neg.	26.3	0		
	Positive	47.9	32.4	32.4	21.6
	Int. neg.	-16.6	-26.3	-11.6	-24.8

The required flexural reinforcement is contained in Table 3-25. The provided areas of steel are greater than the minimum required (ACI 13.3.1). Also, the provided spacing is less than the maximum allowed according to ACI 13.3.2.

The amount of slab reinforcement needs to be checked at the end support and the first interior support to ensure that the moment transfer requirements in ACI 13.5.3 are satisfied.

Table 3-25 Required Slab Reinforcement at Floor Level 4 (SDC C)

Location			M_u (ft-kips)	b (in.)	A_s^* (in.^{2})	Reinforcement*
End Span	Column strip	Ext. neg.	-127.2	132	3.79	13-No. 5
		Positive	96.6	132	2.86	10-No. 5
		Int. neg.	-189.4	132	5.73	19-No. 5
	Middle strip	Ext. neg.	0	156	2.53	9-No. 5
		Positive	65.4	156	2.53	9-No. 5
		Int. neg.	-53.0	156	2.53	9-No. 5
Interior Span	Column strip	Positive	65.4	132	2.14	8-No. 5
		Negative	-178.8	132	5.42	18-No. 5
	Middle strip	Positive	43.6	156	2.53	9-No. 5
		Negative	-49.9	156	2.53	9-No. 5

* Minimum $A_s = 0.0018bh = 0.0018 \times 132 \times 9 = 2.14$ in.^{2} (ACI 13.3.1)
$= 0.0018 \times 156 \times 9 = 2.53$ in.^{2}
Maximum spacing = $2h$ = 18 in. For b = 132 in., 132/18 = 7.3 spaces, say 8 bars
For b = 156 in., 156/18 = 8.6 spaces, say 9 bars

End Support – Additional Flexural Reinforcement Required for Moment Transfer. At this location, the maximum unbalanced moment at the slab-column connection is 127.2 ft-kips (see Table 3-25). A fraction of this moment $\gamma_f M_u$ must be transferred over an effective width equal to $c_2 + 3h = 24 + (3 \times 9) = 51$ in. (ACI 13.5.3.2).

The fraction of unbalanced moment transferred by flexure is calculated from Eq. 13-1:

$$\gamma_f = \frac{1}{1+(2/3)\sqrt{b_1/b_2}} = \frac{1}{1+(2/3)\sqrt{[24+(7.75/2)]/(24+7.75)}} = 0.62$$

For edge columns bending perpendicular to the edge, the value of γ_f computed from Eq. 13-1 is permitted to be increased by up to 25% provided that $V_u \leq 0.4\phi V_c$ (ACI 13.5.3.3). No adjustment to γ_f is made in this example.

Unbalanced moment transferred by flexure = $\gamma_f M_u$ = 0.62 × 127.2 = 78.9 ft-kips. The required area of steel to resist this moment in the 51-in. wide strip is A_s = 2.37 in.^{2}, which is equivalent to 8-No. 5 bars.

Provide the 8-No. 5 bars by concentrating 8 of the column strip bars (13-No. 5) within the 51 in. width over the column. For symmetry, add another bar in the column strip and check bar spacing:

- For 8-No. 5 within 51 in. width: 51/8 = 6.4 in. < 18 in. O.K.
- For 6-No. 5 within 132 – 51 = 81 in. width: 81/6 = 13.5 in. < 18 in. O.K.

Figure 3-11 shows the reinforcement detail for the top bars at the exterior column.

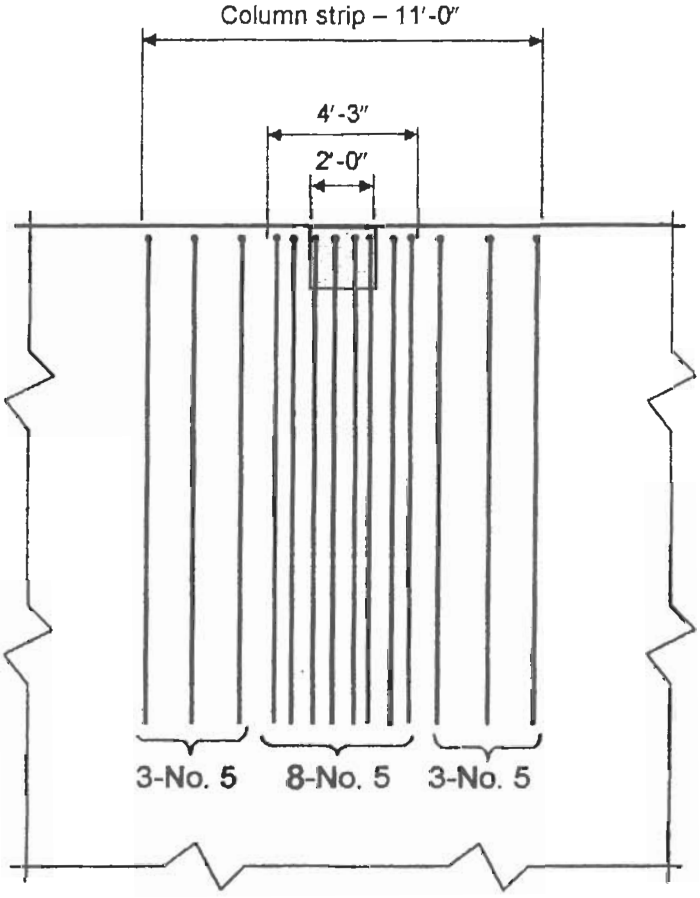

Figure 3-11 Reinforcement Detail at Exterior Column (SDC C)

End Support – Design for Positive Moment. From the third load combination, a positive moment of 26.3 ft-kips occurs in the column strip at the face of the exterior support (see Table 3-24). The 10-No. 5 bars required for the positive moment in the column strip in the end span are sufficient to resist this moment. Two of these bottom bars, which must be anchored into the column in accordance with ACI 13.3.8.5, are sufficient to transfer 62% of this moment by flexure in the 51-in. slab width.

First Interior Support – Additional Flexural Reinforcement Required for Moment Transfer. The maximum factored moment in the column strip at the first interior support is 189.4 ft-kips (see Table 3-25). At an interior column, $b_1 = b_2 = 24 + 7.75 = 31.75$ in. Therefore, from Eq. 13-1, $\gamma_f = 0.60$, and $\gamma_f M_u = 0.60 \times 189.4 = 113.6$ ft-kips.

Required area of steel to resist this moment in the 51-in. wide strip is A_s = 3.52 in.^{2}, which is equivalent to 12-No. 5 bars.

Provide the 12-No. 5 bars by concentrating 12 of the column strip bars (19-No. 5) within the 51 in. width over the column. For symmetry, add another bar in the column strip and check bar spacing:

- For 12-No. 5 within 51 in. width: 51/12 = 4.3 in. < 18 in. O.K.
- For 8-No. 5 within 132 – 51 = 81 in. width: 81/8 = 10.1 in. < 18 in. O.K.

Design for Shear.

Shear strength is checked at the slab-column connections at the end support and the first interior support for the effects from gravity loads and combined effects from gravity loads and earthquake-induced displacements.

End Support – Check for Shear Strength. At this location, the factored shear force V_u due to gravity loads is:

$$V_u = w_u(A_t - b_1 b_2) - \frac{M_1 - M_2}{\ell_n}$$

$$= 0.260\left[288 - \frac{27.875 \times 31.75}{144}\right] - \frac{218.4 - 81.1}{20}$$

$$= 73.3 - 6.9 = 66.4 \text{ kips}$$

where A_t = tributary area of column

$$= \left(\frac{22}{2} + \frac{12}{12}\right) \times 24 = 288 \text{ ft}^2$$

b_1 = length of critical section perimeter in direction of analysis

= 24 + (7.75/2) = 27.875 in. (see Figure 3-12)

b_2 = length of critical section perimeter perpendicular to direction of analysis

= 24 + 7.75 = 31.75 in.

M_1 = total negative design strip moment at interior support determined from Direct Design Method (ACI 13.6.6.3)

$= 0.70\, M_o = 0.70 \times 312 = 218.4$ ft-kips

M_2 = total negative design strip moment at exterior support determined from Direct Design Method (ACI 13.6.6.3)

$= 0.26 M_o = 0.26 \times 312 = 81.1$ ft-kips

M_o = total factored static moment in span determined from Eq. 13-3

$$= \frac{w_u \ell_2 \ell_n^2}{8} = \frac{0.260 \times 24 \times 20^2}{8} = 312 \text{ ft-kips}$$

The section properties of the critical section are determined as follows [3.7]:

$$A_c = (2b_1 + b_2)d = 678.1 \text{ in.}^2$$

$$\frac{J}{c} = \frac{2b_1^2 d(b_1 + 2b_2) + d^3(2b_1 + b_2)}{6b_1} = 6{,}824 \text{ in.}^3$$

According to ACI 13.6.3.6, gravity load moment to be transferred between the slab and edge column must be set equal to $0.30M_o$ = 93.6 ft-kips. Also, $\gamma_v = 1 - 0.62 = 0.38$.

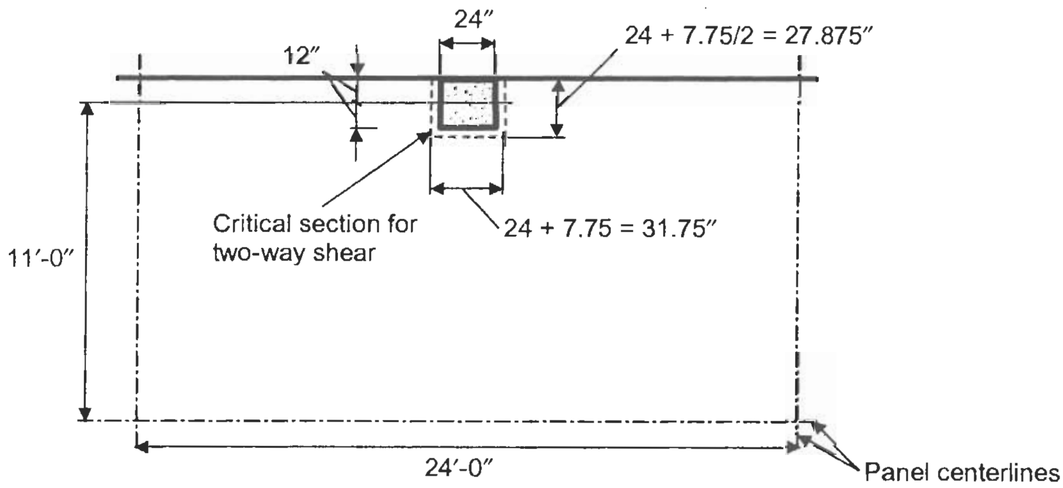

Figure 3-12 Critical Section for Two-way Shear at Exterior Column

Therefore, the combined factored shear stress at the face of the critical section due to gravity loads is:

$$v_u = \frac{66{,}400}{678.1} + \frac{0.38 \times 93.6 \times 12{,}000}{6{,}824}$$

$$= 97.9 + 62.6 = 160.5 \text{ psi}$$

Design shear strength of nonprestressed slabs is the smallest value obtained from Eqs. 11-35 through 11-37. For a square column, Eq. 11-37 governs:

$$\phi v_c = \frac{\phi V_c}{b_o d} = \phi 4\sqrt{f_c'} = 0.85 \times 4\sqrt{4{,}000} = 215.0 \text{ psi} > 160.5 \text{ psi} \quad \text{O.K.}$$

In addition to the gravity load case, shear strength must be checked for the combined effects from gravity loads and earthquake-induced displacements. Direct shear forces on critical section due to gravity loads are:

$$V_D = w_D(A_t - b_1 b_2) - \frac{M_{1D} - M_{2D}}{\ell_n}$$

$$= 0.143\left[288 - \frac{27.875 \times 31.75}{144}\right] - \frac{(0.7 \times 171.6) - (0.26 \times 171.6)}{20}$$

$$= 40.3 - 3.8 = 36.5 \text{ kips}$$

$$V_L = w_L(A_t - b_1 b_2) - \frac{M_{1L} - M_{2L}}{\ell_n}$$

$$= 0.035\left[288 - \frac{27.875 \times 31.75}{144}\right] - \frac{(0.7 \times 42.0) - (0.26 \times 42.0)}{20}$$

$$= 9.9 - 0.9 = 9.0 \text{ kips}$$

The shear force V_E due to the compatibility moments = (66.4 + 65.3)/20 = 6.6 kips. Therefore, total factored shear force at exterior column is:

$$V_u = 1.05V_D + 1.28V_L + V_E = (1.05 \times 36.5) + (1.28 \times 9.0) + 6.6 = 56.5 \text{ kips}$$

When compatibility moments due to seismic displacements are considered, shear stress computations can be based on the actual unbalanced moment, rather than on the provisions of ACI 13.6.3.6, which, as shown above, requires the unbalanced moment to be $0.30M_o$. The actual unbalanced moment at the exterior slab-column connection is 127.2 ft-kips from the second load combination (see Table 3-24). The combined shear stress is:

$$v_u = \frac{56{,}500}{678.1} + \frac{0.38 \times 127.2 \times 12{,}000}{6{,}824}$$

$$= 83.3 + 85.0 = 168.3 \text{ psi} < 215.0 \text{ psi} \quad \text{O.K.}$$

First Interior Support – Check for Shear Strength. At this location, the factored shear force V_u due to gravity loads is:

$$V_u = w_u(A_t - b_1 b_2) + \frac{M_1 - M_2}{\ell_n}$$

$$= 0.260\left[528 - \frac{31.75^2}{144}\right] + \frac{218.4 - 81.1}{20}$$

$$= 135.5 + 6.9 = 142.4 \text{ kips}$$

where $A_t = 24 \times 22 = 528 \text{ ft}^2$

$b_1 = b_2 = 24 + 7.75 = 31.75$ in.

$M_1 = 0.70\, M_o = 0.70 \times 312 = 218.4$ ft-kips

$M_2 = 0.26\, M_o = 0.26 \times 312 = 81.1$ ft-kips

The section properties of the critical section are determined as follows [3.7]:

$$A_c = 2(b_1 + b_2)d = 984.3 \text{ in.}^2$$

$$\frac{J}{c} = \frac{b_1 d(b_1 + 3b_2) + d^3}{3} = 10{,}572 \text{ in.}^3$$

The difference between the slab moments acting on opposite faces of the interior support needs to be transferred by shear to the first interior column. From Table 3-8, the exterior moment at the face of the support is $0.70M_o$ = 0.70 × 312 = 218.4 ft-kips, and the interior moment at the face of the support is $0.65M_o$ = 202.8 ft-kips. Therefore, the unbalanced moment = 218.4 – 202.8 = 15.6 ft-kips. The combined shear stress is:

$$v_u = \frac{142{,}400}{984.3} + \frac{0.4 \times 15.6 \times 12{,}000}{10{,}572}$$

$$= 144.7 + 7.1 = 151.8 \text{ psi} < 215.0 \text{ psi} \quad \text{O.K.}$$

In addition to the gravity load case, shear strength must be checked for the combined effects from gravity loads and earthquake-induced displacements. Direct shear forces on critical section due to gravity loads are:

$$V_D = w_D(A_t - b_1 b_2) + \frac{M_{1D} - M_{2D}}{\ell_n}$$

$$= 0.143\left[528 - \frac{31.75^2}{144}\right] + \frac{(0.7 \times 171.6) - (0.26 \times 171.6)}{20}$$

$$= 74.5 + 3.8 = 78.3 \text{ kips}$$

$$V_L = w_L(A_t - b_1 b_2) + \frac{M_{1L} - M_{2L}}{\ell_n}$$

$$= 0.035\left[528 - \frac{31.75^2}{144}\right] + \frac{(0.7\times 42.0) - (0.26\times 42.0)}{20}$$

$$= 18.2 + 0.9 = 19.1 \text{ kips}$$

The shear force V_E due to the compatibility moments = (64.1 + 64.1)/20 = 6.4 kips. Therefore, total factored shear force at the interior column is:

$$V_u = 1.05V_D + 1.28V_L + V_E = (1.05\times 78.3) + (1.28\times 19.1) + 6.4 = 113.1 \text{ kips}$$

As shown above, the unbalanced moment at the first interior support due to gravity loads is the difference between the moments acting on the two sides of the support. However, the unbalanced moment due to the earthquake-induced displacements is the sum of the moments acting on the two sides of the support. Therefore, the total unbalanced moment is:

$$M_u = 1.05[(0.7\times 171.6) - (0.65\times 171.6)] + 1.28[(0.7\times 42.0) - (0.65\times 42.0)]$$

$$+ (65.3 + 64.1)$$

$$= 141.1 \text{ ft-kips}$$

The combined stress is:

$$v_u = \frac{113{,}100}{984.3} + \frac{0.4\times 141.1\times 12{,}000}{10{,}572}$$

$$= 114.9 + 64.1 = 179.0 \text{ psi} < 215.0 \text{ psi} \quad \text{O.K.}$$

Reinforcement Details.

Slab reinforcement must conform to the requirements given in ACI 13.3. The provisions in ACI 13.3.8 must also be satisfied for slabs without beams; included are requirements for structural integrity (ACI 13.3.8.5).

Bar cutoff points for the top bars in the column strip were computed in accordance with ACI 12.10 through 12.12 based on the third load combination, since this resulted in the longest bar lengths. Due to the positive moment at the exterior column, a portion of the top bars must be continuous. Reinforcement details are shown in Figure 3-13.

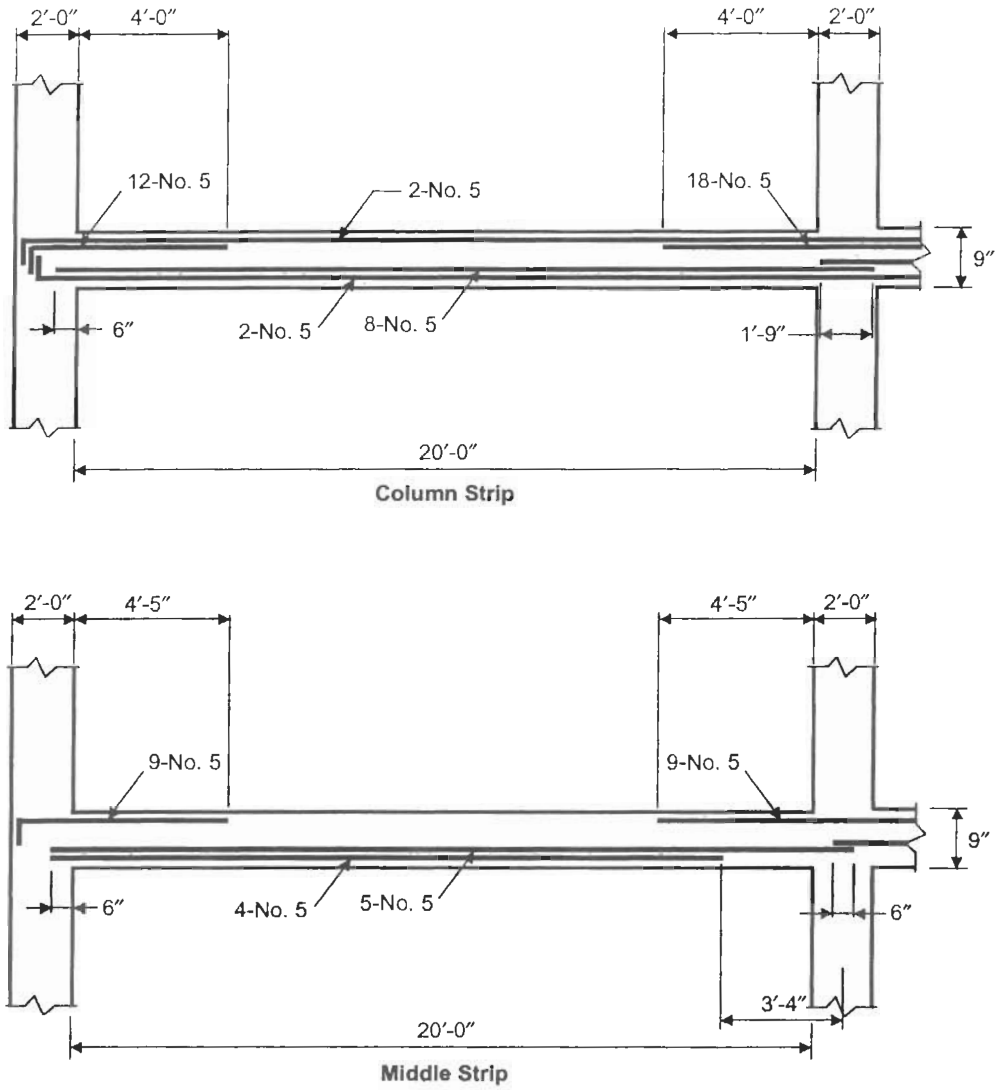

Figure 3-13 Reinforcement Details for Interior Design Strip, Floor Level 4 (SDC C)

3.4.4.3 Design of Column B2

This section outlines the design of column B2 supporting the first floor level. Like the slab, this member is not part of the SFRS and must comply with the requirements in ACI 21.9 (IBC 1617.6.4.3). The reactions on the column obtained from the analysis of the shear walls and frames acting together are multiplied by $C_d\ (\delta_x / \delta'_x) = 4.5 \times 1.29 = 5.8$ to determine the reactions induced by the earthquake-induced displacements, where (δ_x / δ'_x) at the first floor level is given in Table 3-20. Table 3-26 contains a summary of the design axial forces, bending moments, and shear forces on this column from gravity loads and earthquake-induced displacements.

Table 3-26 Summary of Design Axial Forces, Bending Moments, and Shear Forces on Column B2 between Ground and 1st Floor Levels (SDC C)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (*D*)	479	0	0
Live (*L*)*	72	0	0
Seismic (*E*)	± 8	± 151	± 12
Load Combinations			
1.4*D* + 1.7*L*	793	0	0
1.05*D* + 1.28*L* + *E*	603	151	12
0.9*D* + *E*	423	-151	-12

* Live load reduced per IBC 1607.9

Design for axial force and bending.

Based on the governing load combinations in Table 3-26, a 24 × 24 in. column with 8-No. 8 bars ($\rho_g = 1.10\%$) is adequate for column B2 supporting the first floor level. As noted above, slenderness effects need not be considered since P-delta effects were included in the analysis. Since the maximum factored gravity axial force exceeds $A_g f_c' / 10$ = 230 kips, longitudinal reinforcement must satisfy the provisions in ACI 21.4.3.1 (see IBC 1908.1.11, which modifies ACI 21.9.2.2). The provided amount of longitudinal reinforcement is greater than 1% and less than 6% (ACI 21.4.3.1).

Transverse reinforcement.

Transverse reinforcement requirements also depend on the magnitude of the factored gravity axial force. As noted above, the maximum factored gravity axial force exceeds $A_g f_c' / 10$. However, it is less than $0.35 P_o = 0.35[0.85 f_c'(A_g - A_{st}) + f_y A_{st}]$ = 811 kips, where P_o = the nominal axial load strength at zero eccentricity. Therefore, transverse reinforcement requirements in ACI 21.9.2.2 must be satisfied.

According to ACI 21.9.2.2, transverse reinforcement shall not be spaced greater than 6 times the diameter of the smallest longitudinal bar = 6 × 1.0 = 6 in. or 6 in. for the full height of the column.

Since the factored axial forces are greater than $A_g f_c' / 20$ = 115 kips, the shear strength of the concrete may be used (ACI 21.4.5.2). The shear capacity of the column is checked using ACI Eq. 11-4 for members subjected to axial compression, assuming No. 3 hoops spaced at 6 in.:

$$V_c = 2\left(1+\frac{N_u}{2{,}000A_g}\right)\sqrt{f'_c}\,b_w d$$

$$= 2\left(1+\frac{423{,}000}{2{,}000\times 24^2}\right)\sqrt{4{,}000}\times 24\times 17.8/1{,}000 = 73.9 \text{ kips}$$

$$V_s = \frac{A_v f_y d}{s} = \frac{(3\times 0.11)\times 60\times 17.8}{6} = 58.7 \text{ kips}$$

$$\phi V_n = \phi(V_c + V_s) = 0.85\times(73.9+58.7) = 112.7 \text{ kips} > V_u = 12 \text{ kips} \quad \text{O.K.}$$

where N_u = 423 kips is the smallest axial force on the section (see Table 3-26) and d = 17.8 in. was obtained from a strain compatibility analysis.

Splice length of longitudinal reinforcement.

As noted above, IBC 1908.1.11 requires that the longitudinal reinforcement satisfy ACI 21.4.3.1, whereas ACI 21.9.2.2 requires that ACI 21.4.3 be satisfied. This means that in the case of lap splices, the IBC does not require them to conform to ACI 21.4.3.2. However, it is prudent to locate the lap splices within the center half of the member length, to design them as tension lap splices, and to confine them over the entire lap length with transverse reinforcement conforming to ACI 21.4.4.2 and 21.4.4.3. Satisfying ACI 21.4.3.2 locates lap splices outside of potential plastic hinge regions and helps ensure that that they perform as intended. In lieu of lap splices, mechanical splices conforming to ACI 21.2.6 and welded splices conforming to ACI 21.2.7.1 may be utilized. Requirements for lap splices are illustrated below.

Since all of the bars are to be spliced at the same location, a Class B splice is required (ACI 12.15.1, 12.15.2):

Class B splice length = $1.3\,\ell_d$

where the development length ℓ_d is computed from ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 7 and larger bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5 + 0.375 + \dfrac{1.0}{2} = 2.375 \text{ in.} \quad \text{(governs)} \\ \dfrac{24 - 2(1.5 + 0.375) - 1.0}{2 \times 2} = 4.8 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index

$$= \frac{A_{tr} f_{yt}}{1{,}500 sn} = \frac{3 \times 0.11 \times 60{,}000}{1{,}500 \times 5 \times 3} = 0.88$$

$$\frac{c + K_{tr}}{d_b} = \frac{2.375 + 0.88}{1.0} = 3.3 > 2.5, \quad \text{use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.0 \times 1.0 \times 1.0 \times 1.0}{2.5} = 28.5$$

$$\ell_d = 28.5 \times 1.0 = 28.5 \text{ in.}$$

Class B splice length = 1.3 × 28.5 = 37.1 in.

Use a 3 ft-2 in. splice length.

Lap slices are to be enclosed with transverse reinforcement conforming to ACI 21.4.4.2 and 21.4.4.3. Transverse reinforcement shall not be spaced greater than the smallest of (ACI 21.4.4.2):

- Minimum member dimension/4 = 24/4 = 6.0 in.
- 6(diameter of longitudinal reinforcement) = 6 × 1.0 = 6.0 in.
- $s_x = 4 + \left(\dfrac{14 - h_x}{3}\right) = 4 + \left(\dfrac{14 - 11}{3}\right) = 5.0 \text{ in.}$ > 4 in. (governs)

  < 6 in.

where h_x = maximum horizontal spacing of hoop or crosstie legs on all faces of the 24 × 24 in. column

$$= \frac{24 - 2(1.5 + 0.375) - 1.0}{2} + 1.0 + 0.375 = 11.0 \text{ in.} < 14 \text{ in.} \qquad \text{(ACI 21.4.4.3)}$$

Reinforcement details for column B2 are shown in Figure 3-14.

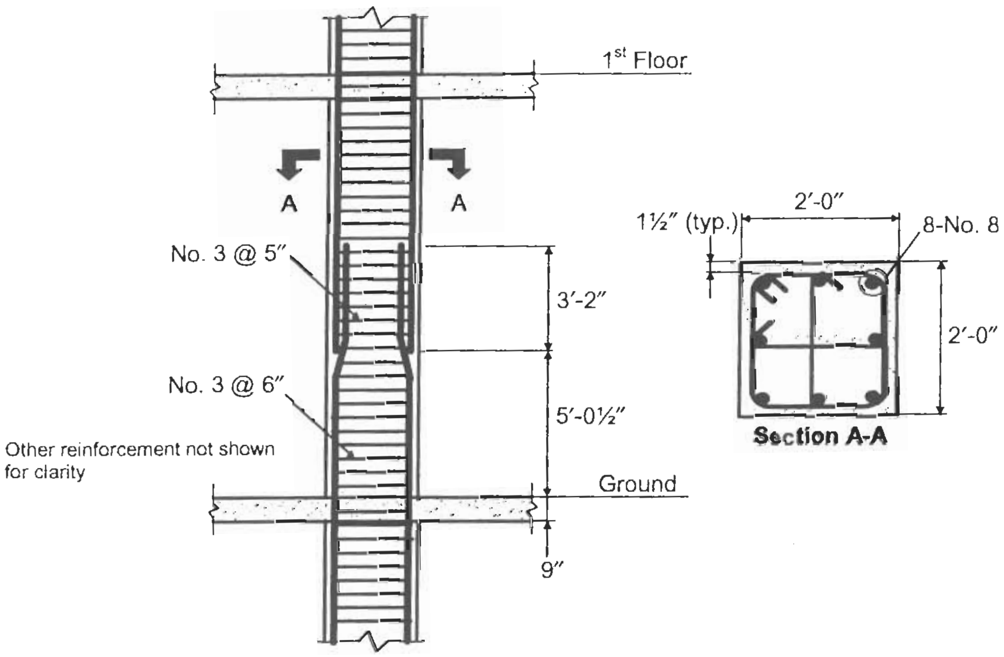

Figure 3-14 Reinforcement Details for Column B2 Supporting the 1st Floor Level (SDC C)

3.4.4.4 Design of Shear Wall on Line 4

Table 3-27 contains a summary of the design axial forces, bending moments, and shear forces at the base of the wall. As noted above, this ordinary reinforced concrete shear wall, as well as all of the other shear walls in the building, is part of the SFRS.

Design for shear.

Calculations for shear strength for SDC C are similar to those given in Section 3.2.4.4 for SDC A. It is determined that 2-No. 4 horizontal and vertical bars spaced at 18 in. satisfy the shear strength requirements of ACI 11.10 for SDC C, as well as the requirements of ACI 14.3.2 and 14.3.3 for minimum ratio of vertical and horizontal reinforcement to gross concrete area, respectively, and ACI 14.3.5 for maximum bar spacing.

Table 3-27 Summary of Design Axial Forces, Bending Moments, and Shear Forces at Base of Shear Wall on Line 4 (SDC C)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	1,029	0	0
Live (L)	132	0	0
Seismic (Q_E)	0	± 11,842	± 226
Load Combinations			
$1.4D + 1.7L$	1,665	0	0
$1.28D + 0.5L + Q_E$	1,383	11,842	226
$0.82D + Q_E$	844	-11,842	-226

Design for axial force and bending.

Figure 3-8 contains the interaction diagram of the wall reinforced with 2-No. 4 vertical bars @ 12 in. and 4-No. 10 bars at each end of the wall, which was determined to be adequate for SDC A and B. As seen from the figure, the wall is also adequate for the load combinations in Table 3-27 for SDC C.

The splice lengths of the reinforcement and the reinforcement details determined for SDC A can be used for SDC C as well (see Figure 3-9).

3.5 DESIGN FOR SDC D – SOUTHEASTERN U.S.

To illustrate the design requirements for Seismic Design Category (SDC) D, the 7-story residential building in Figure 3-15 is assumed to be located in Atlanta, GA on a site with a soft soil profile. Typical structural members are designed and detailed for combined effects of gravity, wind, and seismic forces.

3.5.1 Design Data

- Building Location: Atlanta, GA (zip code 30350)
- Material Properties

  Concrete: $f'_c = 4{,}000$ psi, $w_c = 150$ pcf

  Reinforcement: $f_y = 60{,}000$ psi

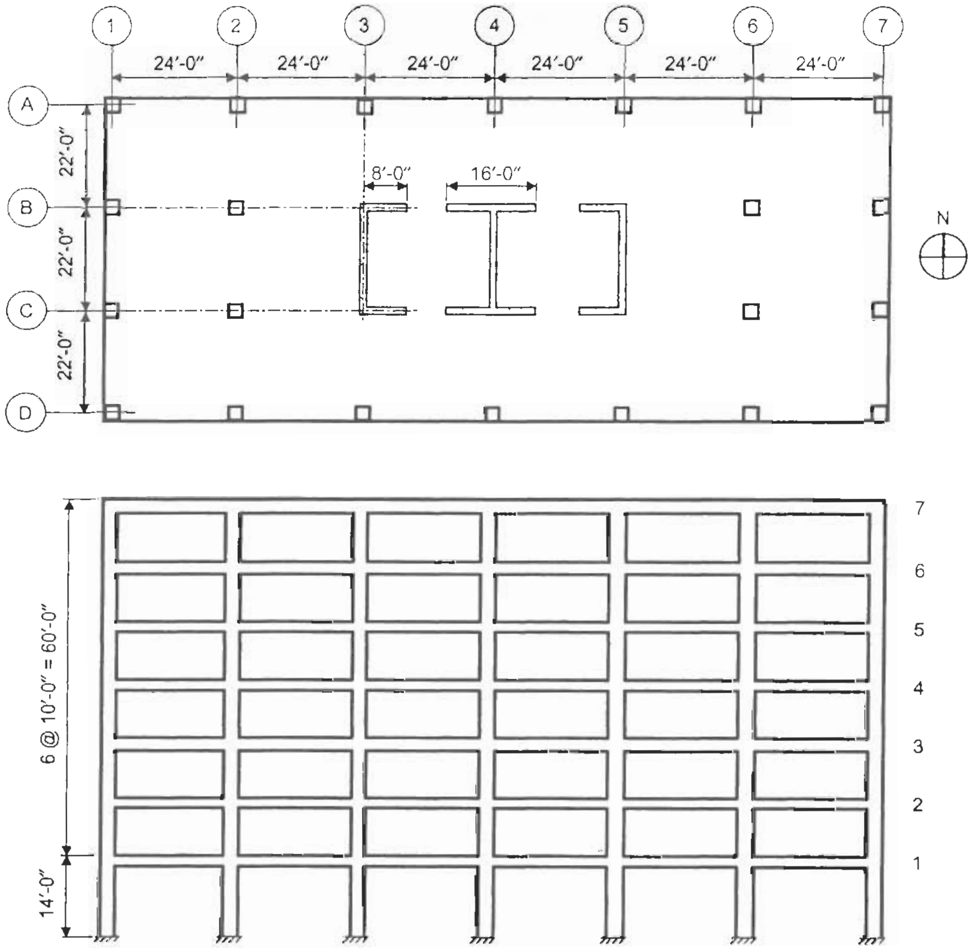

Figure 3-15 Typical Plan and Elevation of Example Building (SDC D)

- Service Loads

Live loads: roof = 20 psf
floor = 50 psf

Superimposed dead loads: roof = 10 psf + 200 kips for penthouse
floor = 30 psf (20 psf permanent partitions + 10 psf ceiling, etc.)

- Seismic Design Data

For zip code 30350: $S_S = 0.276g, S_1 = 0.117g$ [3.1]
Site Class E (soft soil profile; IBC Table 1615.1.1)
For Category (Seismic Use Group) I occupancy, $I_E = 1.0$ (IBC Table 1604.5)

- Wind Design Data

  Basic wind speed = 90 mph (IBC Figure 1609)
  Exposure B (IBC 1609.4)
  For Category I occupancy, $I_W = 1.0$ (IBC Table 1604.5)

- Member Dimensions

  Slab: 9 in.
  Columns: 28 × 28 in.
  Wall thickness: 12 in. in both directions

3.5.2 Seismic Load Analysis

3.5.2.1 Seismic Design Category (SDC)

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1-second period S_{M1} are determined from IBC Eqs. 16-16 and 16-17, respectively:

$$S_{MS} = F_a S_S = 2.42 \times 0.276 = 0.67g$$

$$S_{M1} = F_v S_1 = 3.45 \times 0.117 = 0.40g$$

where F_a and F_v are the values of site coefficients as a function of site class and of mapped spectral response acceleration at short periods S_S and at 1-second period S_1, respectively. The values of these coefficients for Site Class E in the equations above are contained in IBC Table 1615.1.2(1) for F_a and Table 1615.1.2(2) for F_v. Straight-line interpolation was used to determine both F_a and F_v.

Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from Eqs. 16-18 and 16-19:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 0.67 = 0.45g$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.40 = 0.27g$$

Tables 1616.3(1) and 1616.3(2) are utilized to determine the SDC. From Table 1616.3(1) for Seismic Use Group I and S_{DS} = 0.45g, the SDC is C. Similarly, from Table 1616.3(2), the SDC is D for S_{D1} = 0.27g. The most severe SDC is assigned to the structure in accordance with IBC 1616.3. Thus, the SDC is D for this building.

3.5.2.2 Seismic Forces

Since the building does not have plan irregularity Type 1a, 1b, or 4 of Table 1616.5.1 or vertical irregularity Type 1a, 1b, 4, or 5 of Table 1616.5.2, it can be considered regular (IBC 1616.6.3). For this regular building that is less than 240 ft in height, Table 1616.6.3 allows the equivalent lateral force procedure in IBC 1617.4 to be used to compute the seismic base shear V (see Eq. 16-34):

$$V = C_s W$$

where C_s is the seismic response coefficient determined in accordance with IBC 1617.4.1.1 and W is the effective weight of the structure defined in IBC 1617.4. For the member sizes and superimposed dead loads given above, W = 12,291 kips (see Table 3-28 below).

In both directions, a building frame system with special reinforced concrete shear walls is utilized, which is permitted for structures assigned to SDC D with a height less than or equal to 160 ft (see IBC Table 1617.6 and IBC 1910.5). The response modification coefficient $R = 6$ and the deflection amplification factor $C_d = 5$ (IBC Table 1617.6).

Approximate period (T_a). The fundamental period of the building T is determined in accordance with IBC 1617.4.2. In lieu of a more exact analysis, an approximate fundamental period T_a is computed from Eq. 16-39:

Building height $h_n = 74$ ft

Building period coefficient $C_T = 0.02$

Period $T_a = C_T (h_n)^{3/4} = 0.020 \times (74)^{3/4} = 0.50$ sec

Seismic base shear (V). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.27}{\left(\frac{6}{1.0}\right) \times 0.50} = 0.090$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{0.44}{\left(\frac{6}{1.0}\right)} = 0.073$$

Also, C_s must not be less than the following value from Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 0.44 \times 1.0 = 0.019$$

Thus, the value of C_s from Eq. 16-35 governs so that the base shear V in the N-S and E-W directions is:

$$V = C_s W = 0.073 \times 12{,}291 = 897 \text{ kips}$$

Vertical distribution of seismic forces. The total base shear is distributed over the height of the building in conformance with Eqs. 16-41 and 16-42:

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

where F_x is the lateral force induced at level x, w_x and w_i are the portions of W assigned to levels x or i, and k is the distribution exponent defined in IBC 1617.4.3. For T= 0.50 sec, k = 1.0. The lateral forces F_x and the story shears V_x are contained in Table 3-28.

Table 3-28 Seismic Forces and Story Shears (SDC D)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
7	1,597	74	118,178	199	199
6	1,770	64	113,280	191	390
5	1,770	54	95,580	161	551
4	1,770	44	77,880	131	682
3	1,770	34	60,180	101	783
2	1,770	24	42,480	71	854
1	1,844	14	25,816	43	897
Σ	12,291		533,394	897	

3.5.2.3 Method of Analysis

As discussed in Section 3.4.2.3 of this publication, shear walls in a building frame system are designed to carry the total lateral loads, and frames provide support for the gravity loads. Also, frame members, which are not designated to be part of the SFRS, must be designed and detailed to maintain support of design dead and live loads when subjected to expected deformations caused by seismic forces.

The reactions in members that are part of the SFRS were determined from a three-dimensional analysis of a structural model comprised of the shear walls subjected to the seismic forces in Table 3-28. In accordance with IBC 1617.4.4.4, the center of mass was displaced each way from its actual location a distance equal to 5 percent of the building dimension perpendicular to the applied forces to account for accidental torsion. Torsional effects need not be amplified, since the building does not possess Type 1a or 1b plan torsional irregularities as defined in Table 1616.5.1 (IBC 1617.4.4.5). Also, 70% of the gross section properties of the walls were used to account for cracking. P-delta effects were also considered in the analysis. The deflections δ_{xe} in the N-S direction from this analysis are contained in Table 3-29.

Table 3-29 Lateral Displacements (in.) due to Seismic Forces in N-S Direction (SDC D)

Story	δ_{xe} (in.)	δ_x (in.)	Δ (in.)	δ'_{xe} (in.)	δ'_x (in.)	δ_x / δ'_x
7	0.43	2.15	0.30	0.36	1.80	1.19
6	0.37	1.85	0.35	0.31	1.55	1.19
5	0.30	1.50	0.35	0.26	1.30	1.15
4	0.23	1.15	0.35	0.20	1.00	1.15
3	0.16	0.80	0.30	0.14	0.70	1.14
2	0.10	0.50	0.25	0.09	0.45	1.11
1	0.05	0.25	0.25	0.04	0.20	1.25

For all structures assigned to SDC D and higher, IBC 1620.3.5 requires that orthogonal effects of the seismic forces be considered for design and detailing of the components of the SFRS. In the 2002 supplement to the 2000 IBC [3.9], the orthogonal combination procedure is required only for columns or walls that form part of two or more intersecting seismic-force-resisting systems and that are subjected to axial load due to seismic forces greater than or equal to 20% of the axial load design strength. Orthogonal effects can be neglected for the shear walls in this example building.

IBC 1617.6.4.3 requires that reinforced concrete members not designed as part of the SFRS comply with ACI 21.9. According to ACI 21.9.1, such members are to be detailed according to ACI 21.9.2 or 21.9.3 depending on the magnitude of bending moments in those members when subjected to the design displacements δ_x where $\delta_x = C_d \delta_{xe} / I_E$ and δ_{xe} are the displacements calculated by an elastic analysis of the SFRS subjected to the prescribed seismic forces (IBC 1617.6.4.3). The approximate method outlined in Reference 3.8, which was first introduced in Section 3.4.2.3 of this publication, is used to determine these reactions. A three-dimensional analysis similar to the one described above was performed on the entire structure subjected to the seismic forces contained in Table 3-28. The effective beam width of the slab was set equal to one-fourth and one-eighth of the bay width in the direction perpendicular to the direction of analysis for interior and exterior frames, respectively. These effective widths include a reduction in stiffness for cracking. An effective moment of inertia equal to 70% of the gross moment of inertia was taken for the columns. The displacements δ'_{xe} determined from this analysis are also contained in Table 3-29.

3.5.2.4 Story Drift and P-delta Effects

Story drift determination. Table 3-29 contains the displacements δ_{xe} of the SFRS in the N-S direction obtained from the 3-D static, elastic analysis using the design seismic forces, including accidental torsional effects, and the design earthquake displacement δ_x computed by IBC Eq. 16-46. As noted above, C_d is equal to 5 for this system. The interstory drifts Δ computed from the δ_x are also contained in the table.

The design story drifts Δ must not exceed the allowable story drift Δ_a from Table 1617.3 (IBC 1617.3). For the 10-ft story heights, $\Delta_a = 0.020 \times 10 \times 12 = 2.40$ in. and for the 14-ft story height at the first level, $\Delta_a = 3.36$ in. It is evident from Table 3-29 that for all stories, the lateral drifts obtained from the prescribed lateral forces in the N-S direction are less than the limiting values.

Similar calculations for seismic forces in the E-W direction also show that the lateral drifts are less than the allowable values.

P-delta effects. As noted above, P-delta effects were automatically considered in the analysis using SAP2000. However, for illustration purposes, the following procedure can be used to determine whether P-delta effects need to be considered or not in accordance with IBC 1617.4.6.2.

P-delta effects need not be considered when the stability coefficient θ determined by Eq. 16-47 is less than or equal to 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d}$$

where P_x = total unfactored vertical design load at and above level x

Δ = design story drift occurring simultaneously with V_x

V_x = seismic shear force acting between level x and $x - 1$

h_{sx} = story height below level x

C_d = deflection amplification factor

The stability coefficient θ must not exceed θ_{max} determined from Eq. 16-48:

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25$$

where β is the ratio of shear demand to shear capacity between level x and $x - 1$, which may be taken equal to 1.0 when it is not calculated.

Table 3-30 contains the calculations for the N-S direction. It is clear that P-delta effects need not be considered at any of the floor levels. Note that θ_{max} is equal to 0.100 in the N-S direction using $\beta = 1.0$. Similar calculations for the E-W direction show that P-delta effects may also be neglected.

Table 3-30 P-delta Effects (SDC D)

Level	h_{sx} (ft)	P_x (kips)	V_x (kips)	Δ (in.)	θ
7	10	1,797	199	0.30	0.0045
6	10	3,767	390	0.35	0.0056
5	10	5,737	551	0.35	0.0061
4	10	7,707	682	0.35	0.0066
3	10	9,677	783	0.30	0.0062
2	10	11,647	854	0.25	0.0057
1	14	13,691	897	0.25	0.0045

3.5.3 Wind Load Analysis

3.5.3.1 Wind Forces

The analytical procedure (Method 2) of ASCE 6.5 may be used to determine the wind forces.

Details on how to compute the wind forces are given in Section 3.2.3 of this publication. A summary of the design wind forces in the N-S direction at all floor levels is contained in Table 3-31. Once again it is important to note that the wind directionality factor K_d has been taken equal to 1.0 (see Exception 1 in IBC 1605.2.1).

3.5.3.2 Method of Analysis

A three-dimensional analysis of the building was performed in the N-S and E-W directions using SAP2000. The modeling assumptions utilized for the seismic analysis were also used for the wind analysis.

According to ASCE 6.5.12.3, main wind-force-resisting systems of buildings with mean roof height h greater than 60 ft must be designed for the full and partial wind load cases of Figure 6-9 (Cases 1 through 4). These four cases were considered in the three-dimensional analysis.

Table 3-31 Design Wind Forces in N-S Directions (V = 90 mph)

Level	Height above ground level, z (ft)	Total Design Wind Force (kips)
7	74	15.2
6	64	29.6
5	54	28.8
4	44	27.8
3	34	26.7
2	24	25.3
1	14	28.0
	Σ	181.4

3.5.4 Design for Combined Load Effects

3.5.4.1 Load Combinations

Basic load combinations for strength design are given in IBC 1605.2.1. As noted above, the first exception in this section requires that the non-seismic load combinations of ACI 9.2 be used for concrete structures. Thus, the following load combinations are applicable to structural members that are part of the SFRS:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)
4. $1.2D + f_1 L + 1.0E$ (Formula 16-5)
5. $0.9D + 1.0E$ (Formula 16-6)

where D, L, W, and E are the effects due to dead, live, wind, and seismic loads, respectively, and f_1 is equal to either 1.0 for places of public assembly, for live loads in excess of 100 psf, and for parking garage live load, or is equal to 0.5 for other live loads.

The seismic load effect E for use in Formula 16-5, which is the combined effect of horizontal and vertical earthquake-induced forces, is computed from Eq. 16-28 where the effects of gravity and seismic ground motion are additive:

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of horizontal seismic forces

ρ = redundancy coefficient determined in accordance with IBC 1617.2.2 for SDC D, E, or F

Similarly, E for use in Formula 16-6 is computed from Eq. 16-29 where the effects of gravity and seismic ground motion counteract:

$$E = \rho Q_E - 0.2 S_{DS} D$$

According to IBC 1617.2.2, the redundancy coefficient ρ, which shall not be less than 1.0 and need not exceed 1.5, is the largest of the values of ρ_i calculated at each story i from Equation 16-32:

$$\rho_i = 2 - \frac{20}{r_{\max_i}\sqrt{A_i}}$$

For shear walls:

$r_{\max_i}$ = (maximum wall shear × $10/\ell_w$)/total story shear

ℓ_w = length of the wall in feet

For the building in Figure 3-15, the shear wall on line 3 or 5 will have the largest shear force at its base, depending on which direction the center of mass is displaced. Therefore,

$$r_{\max_1} = \frac{336 \times \frac{10}{23}}{897} = 0.16$$

$$\rho_{\max} = 2 - \frac{20}{0.16\sqrt{146.33 \times 68.33}} = 0.75 < 1.0$$

Use $\rho = 1.0$.

Substituting $S_{DS} = 0.44g$ and $\rho = 1.0$ into the equations for E, and then substituting E along with $f_1 = 0.5$ into load combinations 4 and 5 above results in the following:

4a. $1.2D + 0.5L + 1.0Q_E + (0.2 \times 0.44)D = 1.29D + 0.5L + Q_E$

4b. $1.2D + 0.5L + 1.0Q_E - (0.2 \times 0.44)D = 1.11D + 0.5L + Q_E$

5a. $0.9D + 1.0Q_E + (0.2 \times 0.44)D = 0.99D + Q_E$

5b. $0.9D + 1.0Q_E - (0.2 \times 0.44)D = 0.81D + Q_E$

Values of Q_E are obtained from the structural analysis for both sidesway to the left and to the right based on the code-prescribed horizontal seismic forces acting on the building.

The exception in IBC 1617.6.4.3 requires that reinforced concrete members that are not part of the SFRS comply with ACI 21.9. Therefore, the load combinations for these members are:

- $1.05D + 1.28L + E$
- $0.9D + E$

where the values of E are the forces in the frame members due to the lateral displacements δ_x being applied to the frames. When the approximate method outlined in Section 3.4.2.3 is utilized, $E = F_M$.

Also, the special seismic load combinations in IBC 1605.4 are not applicable in this example.

3.5.4.2 Slab Design

Preliminary Slab Thickness.

As noted above, the flat plate is not part of the SFRS. Thus, the requirements of ACI 21.9 for frame members not proportioned to resist forces induced by earthquake motions must be satisfied (IBC 1617.6.4.3). Since factored gravity axial forces on the slab are less than $A_g f'_c / 10$, either ACI 21.9.2 or 21.9.3 apply, depending on whether or not the earthquake-induced bending moments and shear forces due to the design displacements exceed the design moment and shear strength of the slab. In the supplement to the 2000 IBC [3.9], ACI 21.9.2.1 and 21.9.3.2 were modified to include specific requirements for two-way slabs. In particular, stirrups need not be provided in two-way slabs with column-line beams that are the same thickness as the slab. Shear reinforcement in the slab is to be provided in accordance with ACI 11.12 where required.

Calculations given in Section 3.2.4.2 of this publication show that a 9-in. thick slab and 22-in. square columns are adequate for serviceability and shear strength requirements for gravity loads. A more refined check for shear strength is made at a later stage in this example for a 9-in. thick slab and 28-in. square columns for effects from gravity loads and combined effects from gravity loads and earthquake-induced displacements.

Design for Flexure.

It was determined in Section 3.2.4.2 of this publication that the Direct Design Method can be utilized to compute the design bending moments in the slab due to gravity loads. Table 3-32 contains a summary of the service gravity load bending moments in the column strip and middle strip of an end span and an interior span of an interior N-S design strip. The total static service dead load and live load moments for spans in an interior design strip in the N-S direction are computed as follows:

$$M_o = \frac{w\ell_n^2\ell_2}{8}$$

$$(M_o)_D = \frac{0.143 \times 19.67^2 \times 24}{8} = 166.0 \text{ ft - kips}$$

$$(M_o)_L = \frac{0.035 \times 19.67^2 \times 24}{8} = 40.6 \text{ ft - kips}$$

The reactions in the slab due to seismic displacements δ_x are determined using the approximate method outlined above. Table 3-33 contains the bending moments M'_E in the slab at the sixth floor level obtained from a 3-D analysis of the shear walls and frames acting together subjected to the seismic forces in Table 3-28. The table also contains the approximate compatibility bending moments $M_E = C_d(\delta_x / \delta'_x)M'_E$, where the ratio (δ / δ'_x) at the sixth floor level is equal to 1.19 (see Table 3-29).

Table 3-32 Service Dead and Live Load Bending Moments (ft-kips) in N-S Interior Design Strip (SDC D)

Location		Moment	M_D	M_L
End Span				
Column Strip	Exterior Negative	$0.26M_O$	-43.2	-10.6
	Positive	$0.31M_O$	51.5	12.6
	Interior Negative	$0.53M_O$	-88.0	-21.5
Middle Strip	Exterior Negative	0	0	0
	Positive	$0.21M_O$	34.9	8.5
	Interior Negative	$0.17M_O$	-28.2	-6.9
Interior Span				
Column Strip	Positive	$0.21M_O$	34.9	8.5
	Interior Negative	$0.49M_O$	-81.3	-19.9
Middle Strip	Positive	$0.14M_O$	23.2	5.7
	Interior Negative	$0.16M_O$	-26.6	-6.5

Table 3-33 Compatibility Bending Moments (ft-kips) in N-S Interior Design Strip (SDC D)

Location		M'_E	M_E
End Span	Exterior Negative	± 23.7	± 141.0
	Interior Negative	± 19.7	± 117.2
Interior Span	Interior Negative	± 19.5	± 116.0

A summary of the governing design bending moments at the 6th floor level is contained in Table 3-34.

Table 3-34 Summary of Slab Design Bending Moments (ft-kips) at Floor Level 6 (SDC D)

Load Case	Location	End Span		Interior Span	
		Column Strip	Middle Strip	Column Strip	Middle Strip
Dead (*D*)	Ext. neg.	-43.2	0		
	Positive	51.5	34.9	34.9	23.2
	Int. neg.	-88.0	-28.2	-81.3	-26.6
Live (*L*)	Ext. neg.	-10.6	0		
	Positive	12.6	8.5	8.5	5.7
	Int. neg.	-21.5	-6.9	-19.9	-6.5
Seismic (*E*)	Ext. neg.	± 141.0			
	Positive				
	Int. neg.	± 117.2		± 116.0	
Load Combinations					
1.4*D* + 1.7*L*	Ext. neg.	-78.5	0		
	Positive	93.5	63.3	63.3	42.2
	Int. neg.	-159.8	-51.2	-147.7	-48.3
1.05*D* + 1.28*L* + *E*	Ext. neg.	-199.9	0		
	Positive	70.2	47.5	47.5	31.7
	Int. neg.	-237.1	-38.4	-226.8	-36.3
0.9*D* + *E*	Ext. neg.	102.1	0		
	Positive	46.4	31.4	31.4	20.9
	Int. neg.	38.0	-25.4	42.8	-23.9

The required flexural reinforcement is contained in Table 3-35. The provided areas of steel are greater than the minimum required (ACI 13.3.1). Also, the provided spacing is less than the maximum allowed according to ACI 13.3.2.

The amount of slab reinforcement needs to be checked at the end support and the first interior support to ensure that the moment transfer requirements in ACI 13.5.3 are satisfied.

End Support – Additional Flexural Reinforcement Required for Moment Transfer. At this location, the maximum unbalanced moment at the slab-column connection is 199.9 ft-kips (see Table 3-35). A fraction of this moment $\gamma_f M_u$ must be transferred over an effective width equal to $c_2 + 3h = 28 + (3 \times 9) = 55$ in. (ACI 13.5.3.2).

The fraction of unbalanced moment transferred by flexure is calculated from Eq. 13-1:

$$\gamma_f = \frac{1}{1+(2/3)\sqrt{b_1/b_2}} = \frac{1}{1+(2/3)\sqrt{[28+(7.75/2)]/(28+7.75)}} = 0.61$$

Table 3-35 Required Slab Reinforcement at Floor Level 6 (SDC D)

Location			M_u (ft-kips)	b (in.)	A_s^* (in.^{2})	Reinforcement*
End Span	Column strip	Ext. neg.	-199.9	132	6.04	20-No. 5
		Positive	93.5	132	2.74	9-No. 5
		Int. neg.	-237.1	132	7.25	24-No. 5
	Middle strip	Ext. neg.	0	156	2.53	9-No. 5
		Positive	63.3	156	2.53	9-No. 5
		Int. neg.	-51.2	156	2.53	9-No. 5
Interior Span	Column strip	Positive	63.3	132	2.14	8-No. 5
		Negative	-226.8	132	6.92	23-No. 5
	Middle strip	Positive	42.2	156	2.53	9-No. 5
		Negative	-48.2	156	2.53	9-No. 5

* Minimum $A_s = 0.0018bh = 0.0018 \times 132 \times 9 = 2.14$ in.^{2} (ACI 13.3.1)
$= 0.0018 \times 156 \times 9 = 2.53$ in.^{2}
Maximum spacing = $2h$ = 18 in. For b = 132 in., 132/18 = 7.3 spaces, say 8 bars
For b = 156 in., 156/18 = 8.6 spaces, say 9 bars

For edge columns bending perpendicular to the edge, the value of γ_f computed from Eq. 13-1 is permitted to be increased by up to 25% provided that $V_u \le 0.4\phi V_c$ (ACI 13.5.3.3). No adjustment to γ_f was made in this example.

Unbalanced moment transferred by flexure = $\gamma_f M_u = 0.61 \times 199.9 = 121.9$ ft-kips. The required area of steel to resist this moment in the 55-in. wide strip is A_s = 3.82 in.^{2}, which is equivalent to 13-No. 5 bars.

Provide the 13-No. 5 bars by concentrating 13 of the column strip bars (20-No. 5) within the 55 in. width over the column. For symmetry, add another bar in the column strip and check bar spacing:

- For 13-No. 5 within 55 in. width: 55/13 = 4.2 in. < 18 in. O.K.
- For 8-No. 5 within 132 – 55 = 77 in. width: 77/8 = 9.6 in. < 18 in. O.K.

The reinforcement detail for the top bars at the exterior column is similar to that depicted in Figure 3-11 for SDC C.

End Support – Design for Positive Moment. From the third load combination, a positive moment of 102.1 ft-kips occurs in the column strip at the face of the exterior support (see Table 3-34), which requires 3.01 in.^{2} of reinforcement. The 9-No. 5 bars required for the positive moment in the column strip in the end span are not sufficient to resist this moment. Therefore, add 2 additional No. 5 bars at the bottom of the section (one additional bar is required to increase the flexural strength and the other to satisfy the spacing requirements in ACI 13.3.2). The 55-in. slab width in the column strip must be able to resist 61% of this moment, which is equal to 62.3 ft-kips. This requires 6-No. 5 bars, with at least 2 of these bars anchored into the column to satisfy the integrity requirements in ACI 13.3.8.5.

First Interior Support – Additional Flexural Reinforcement Required for Moment Transfer. The maximum factored moment in the column strip at the first interior support is 237.1 ft-kips (see Table 3-35). At an interior column, $b_1 = b_2 = 28 + 7.75 = 35.75$ in. Therefore, from Eq. 13-1, $\gamma_f = 0.60$, and $\gamma_f M_u = 0.60 \times 237.1 = 142.3$ ft-kips.

Required area of steel to resist this moment in the 55-in. wide strip is $A_s = 4.50$ in.^{2}, which is equivalent to 15-No. 5 bars.

Provide the 15-No. 5 bars by concentrating 15 of the column strip bars (24-No. 5) within the 55 in. width over the column. For symmetry, add another bar in the column strip and check bar spacing:

- For 15-No. 5 within 55 in. width: 55/15 = 3.7 in. < 18 in. O.K.
- For 10-No. 5 within 132 – 55 = 77 in. width: 77/10 = 7.7 in. < 18 in. O.K.

First Interior Support – Design for Positive Moment. From the third load combination, a positive moment of 42.8 ft-kips occurs in the column strip at the face of the exterior support (see Table 3-34), which requires 1.24 in.^{2} of reinforcement. The 11-No. 5 bars required for the positive moment in the column strip are sufficient to resist this moment. The 55-in. slab width in the column strip must be able to resist 60% of this moment, which is equal to 25.7 ft-kips. This requires 3-No. 5 bars, with at least 2 of these bars anchored into the column to satisfy the integrity requirements in ACI 13.3.8.5.

Design for Shear.

Shear strength is checked at the slab-column connections at the end support and the first interior support for the effects from gravity loads and combined effects from gravity loads and earthquake-induced displacements.

End Support – Check for Shear Strength. At this location, the factored shear force V_u due to gravity loads is:

$$V_u = w_u (A_t - b_1 b_2) - \frac{M_1 - M_2}{\ell_n}$$

$$= 0.260\left[292 - \frac{31.875 \times 35.75}{144}\right] - \frac{210.9 - 78.3}{19.67}$$

$$= 73.9 - 6.7 = 67.2 \text{ kips}$$

where A_t = tributary area of column

$$= \left(\frac{22}{2} + \frac{14}{12}\right) \times 24 = 292 \text{ ft}^2$$

b_1 = length of critical section perimeter in direction of analysis

$= 28 + (7.75/2) = 31.875$ in.

b_2 = length of critical section perimeter perpendicular to direction of analysis

$= 28 + 7.75 = 35.75$ in.

M_1 = total negative design strip moment at interior support determined from Direct Design Method (ACI 13.6.6.3)

$= 0.70 M_o = 0.70 \times 301.3 = 210.9$ ft-kips

M_2 = total negative design strip moment at exterior support determined from Direct Design Method (ACI 13.6.6.3)

$= 0.26 M_o = 0.26 \times 301.3 = 78.3$ ft-kips

M_o = total factored static moment in span determined from Eq. 13-3

$$= \frac{w_u \ell_2 \ell_n^2}{8} = \frac{0.260 \times 24 \times 19.67^2}{8} = 301.3 \text{ ft-kips}$$

The section properties of the critical section are determined as follows [3.7]:

$$A_c = (2b_1 + b_2)d = 771.1 \text{ in.}^2$$

$$\frac{J}{c} = \frac{2b_1^2 d(b_1 + 2b_2) + d^3(2b_1 + b_2)}{6b_1} = 8{,}754 \text{ in.}^3$$

According to ACI 13.6.3.6, gravity load moment to be transferred between the slab and edge column must be set equal to $0.30 M_o$ = 90.4 ft-kips. Also, $\gamma_v = 1 - 0.61 = 0.39$. Therefore, the combined factored shear stress at the face of the critical section due to gravity loads is:

$$v_u = \frac{67{,}200}{771.1} + \frac{0.39 \times 90.4 \times 12{,}000}{8{,}754}$$

$$= 87.2 + 48.3 = 135.5 \text{ psi}$$

Design shear strength of nonprestressed slabs is the smallest value obtained from Eqs. 11-35 through 11-37. For a square column, Eq. 11-37 governs:

$$\phi v_c = \frac{\phi V_c}{b_o d} = \phi 4\sqrt{f_c'} = 0.85 \times 4\sqrt{4{,}000} = 215.0 \text{ psi} > 135.5 \text{ psi} \quad \text{O.K.}$$

In addition to the gravity load case, shear strength must be checked for the combined effects from gravity loads and earthquake-induced displacements. Direct shear forces on critical section due to gravity loads are:

$$V_D = w_D(A_t - b_1 b_2) - \frac{M_{1D} - M_{2D}}{\ell_n}$$

$$= 0.143\left[292 - \frac{31.875 \times 35.75}{144}\right] - \frac{(0.7 \times 165.9) - (0.26 \times 165.9)}{19.67}$$

$$= 40.6 - 3.7 = 36.9 \text{ kips}$$

$$V_L = w_L(A_t - b_1 b_2) - \frac{M_{1L} - M_{2L}}{\ell_n}$$

$$= 0.035\left[292 - \frac{31.875 \times 35.75}{144}\right] - \frac{(0.7 \times 40.6) - (0.26 \times 40.6)}{19.67}$$

$$= 9.9 - 0.9 = 9.0 \text{ kips}$$

The shear force V_E due to the compatibility moments = (141.0 + 117.2)/19.67 = 13.1 kips. Therefore, total factored shear force at exterior column is:

$$V_u = 1.05V_D + 1.28V_L + V_E = (1.05 \times 36.9) + (1.28 \times 9.0) + 13.1 = 63.4 \text{ kips}$$

When compatibility moments due to seismic displacements are considered, shear stress computations can be based on the actual unbalanced moment. The actual unbalanced moment at the exterior slab-column connection is 199.9 ft-kips from the second load combination (see Table 3-34). The combined shear stress is:

$$v_u = \frac{63{,}400}{771.1} + \frac{0.39 \times 199.9 \times 12{,}000}{8{,}754}$$

$$= 82.2 + 106.9 = 189.1 \text{ psi} < 215.0 \text{ psi} \quad \text{O.K.}$$

First Interior Support – Check for Shear Strength. At this location, the factored shear force V_u due to gravity loads is:

$$V_u = w_u(A_t - b_1 b_2) + \frac{M_1 - M_2}{\ell_n}$$

$$= 0.260\left[528 - \frac{35.75^2}{144}\right] + \frac{210.9 - 78.3}{19.67}$$

$$= 135.0 + 6.7 = 141.7 \text{ kips}$$

where $A_t = 24 \times 22 = 528$ ft^2

$b_1 = b_2 = 28 + 7.75 = 35.75$ in.

$M_1 = 0.70 M_o = 0.70 \times 301.3 = 210.9$ ft-kips

$M_2 = 0.26 M_o = 0.26 \times 301.3 = 78.3$ ft-kips

The section properties of the critical section are determined as follows [3.7]:

$$A_c = 2(b_1 + b_2)d = 1{,}108.3 \text{ in.}^2$$

$$\frac{J}{c} = \frac{b_1 d(b_1 + 3b_2) + d^3}{3} = 13{,}362 \text{ in.}^3$$

The difference between the slab moments acting on opposite faces of the interior support needs to be transferred by shear to the first interior column. The exterior moment at the face of the support is $0.70M_o = 0.70 \times 301.3 = 210.9$ ft-kips, and the interior moment at the face of the support is $0.65M_o = 195.9$ ft-kips. Therefore, the unbalanced moment = $210.9 - 195.9 = 15.0$ ft-kips. The combined shear stress is:

$$v_u = \frac{141{,}700}{1{,}108.3} + \frac{0.4 \times 15.0 \times 12{,}000}{13{,}362}$$

$$= 127.9 + 5.4 = 133.3 \text{ psi} < 215.0 \text{ psi} \quad \text{O.K.}$$

In addition to the gravity load case, shear strength must be checked for the combined effects from gravity loads and earthquake-induced displacements. Direct shear forces on critical section due to gravity loads are:

$$V_D = w_D(A_t - b_1 b_2) + \frac{M_{1D} - M_{2D}}{\ell_n}$$

$$= 0.143\left[528 - \frac{35.75^2}{144}\right] + \frac{(0.7\times165.9) - (0.26\times165.9)}{19.67}$$

$$= 74.2 + 3.7 = 77.9 \text{ kips}$$

$$V_L = w_L(A_t - b_1 b_2) + \frac{M_{1L} - M_{2L}}{\ell_n}$$

$$= 0.035\left[528 - \frac{35.75^2}{144}\right] + \frac{(0.7\times40.6) - (0.26\times40.6)}{19.67}$$

$$= 18.2 + 0.9 = 19.1 \text{ kips}$$

The shear force V_E due to the compatibility moments = (116.0 + 116.0)/19.67 = 11.8 kips. Therefore, total factored shear force at the interior column is:

$$V_u = 1.05V_D + 1.28V_L + V_E = (1.05\times77.9) + (1.28\times19.1) + 11.8 = 118.0 \text{ kips}$$

As shown above, the unbalanced moment at the first interior support due to gravity loads is the difference between the moments acting on the two sides of the support. However, the unbalanced moment due to the earthquake-induced displacements is the sum of the moments acting on the two sides of the support. Therefore, the total unbalanced moment is:

$$M_u = 1.05[(0.7\times165.9) - (0.65\times165.9)] + 1.28[(0.7\times40.6) - (0.65\times40.6)]$$

$$+ (117.2 + 116.0)$$

$$= 244.5 \text{ ft-kips}$$

The combined stress is:

$$v_u = \frac{118{,}000}{1{,}108.3} + \frac{0.4\times244.5\times12{,}000}{13{,}362}$$

$$= 106.5 + 87.8 = 194.3 \text{ psi} < 215.0 \text{ psi} \quad \text{O.K.}$$

Reinforcement Details.

Slab reinforcement must conform to the requirements given in ACI 13.3. The provisions in ACI 13.3.8 must also be satisfied for slabs without beams; included are requirements for structural integrity (ACI 13.3.8.5).

Bar cutoff points for the top bars in the column strip were computed in accordance with ACI 12.10 through 12.12 based on the third load combination, since this resulted in the longest bar lengths. Reinforcement details are shown in Figure 3-16.

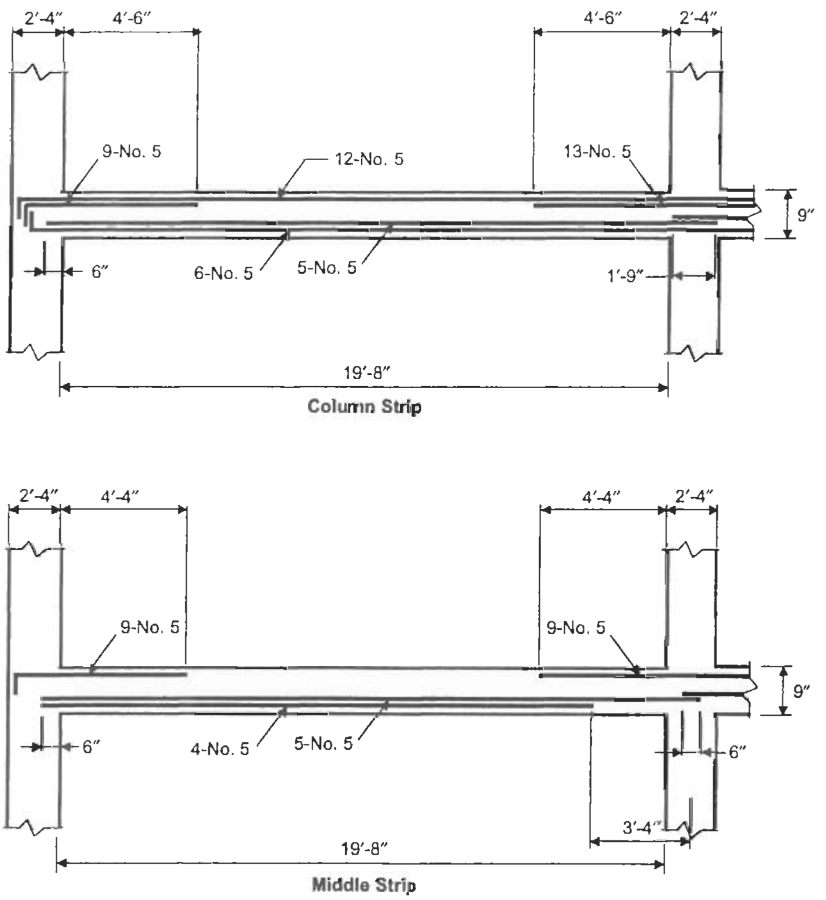

Figure 3-16 Reinforcement Details for Interior Design Strip, Floor Level 6 (SDC D)

3.5.4.3 Design of Column B2

As noted previously, column B2 is not part of the SFRS. The design of this member supporting the first floor level for SDC D is very similar to the design for SDC C presented in Section 3.4.4.3 of the publication. It can be shown that a 28 × 28 in. column with 8-No. 9 bars ($\rho_g = 1.02\%$) is adequate for the load combinations. Reinforcement details for the transverse reinforcement are similar to those depicted in Figure 3-14.

3.5.4.4 Design of Shear Wall on Line 4

Table 3-36 contains a summary of the design axial forces, bending moments, and shear forces at the base of the wall. As noted above, this special reinforced concrete shear wall, as well as all of the other shear walls in the building, is part of the SFRS.

Table 3-36 Summary of Design Axial Forces, Bending Moments, and Shear Forces at Base of Shear Wall on Line 4 (SDC D)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (*D*)	1,288	0	0
Live (*L*)	153	0	0
Seismic (Q_E)	0	± 20,153	± 330
Load Combinations			
1.4*D* + 1.7*L*	2,063	0	0
1.29*D* + 0.5*L* + Q_E	1,738	20,153	330
0.81*D* + Q_E	1,043	-20,153	-330

Design for shear.

Reinforcement requirements. Special reinforced structural walls are to be provided with reinforcement in two orthogonal directions in the plane of the wall in accordance with ACI 21.6.2. The minimum reinforcement ratio in both directions is 0.0025, unless the design shear force is less than or equal to $A_{cv}\sqrt{f'_c}$, where A_{cv} is the gross area of concrete bounded by the web thickness and the length of wall in the direction of analysis. In such cases, minimum reinforcement in accordance with ACI 14.3 for ordinary walls must be provided. For the wall in this example, $A_{cv} = 12 \times 276 = 3{,}312$ in.^{2}, so that

$$A_{cv}\sqrt{f'_c} = 3{,}312 \times \sqrt{4{,}000}/1{,}000 = 210 \text{ kips} < V_u = 330 \text{ kips}$$

Therefore, the minimum reinforcement ratio is 0.0025 and the maximum spacing is 18 in. (ACI 21.6.2.1).

Two curtains of reinforcement are required in a wall when the in-plane factored shear force exceeds $2A_{cv}\sqrt{f'_c} = 2 \times 210 = 420$ kips. In this case, two curtains are not required, since 330 kips < 420 kips. However, two curtains will be provided.

The minimum required reinforcement in each direction per foot of wall is equal to $0.0025 \times 12 \times 12 = 0.36$ in.^{2} Assuming No. 4 bars in two curtains, required spacing s is

$$s = \frac{2 \times 0.20}{0.36} \times 12 = 13.3 \text{ in.} < 18 \text{ in.}$$

Try 2 curtains of No. 4 bars spaced at 12 in.

Shear strength requirements. ACI Eq. 21-7 is used to determine nominal shear strength V_n of structural walls:

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

where $\alpha_c = 2$ for ratio of wall height to length $h_w / \ell_w =$ 74/23 = 3.2 > 2 (ACI 21.6.4.1).

For 2 curtains of No. 4 horizontal bars spaced at 12 in. ($\rho_n = 0.40/(12 \times 12) = 0.0028$):

$$\phi V_n = 0.85 \times 3{,}312 \times [2\sqrt{4{,}000} + (0.0028 \times 60{,}000)]/1{,}000$$

$$= 829 \text{ kips} > V_u = 330 \text{ kips} \qquad \text{O.K.}$$

where $\phi = 0.85$ for walls with $h_w / \ell_w > 2$ (ACI 9.3.4(a)). Therefore, use 2 curtains of No. 4 bars @ 12 in. on center in horizontal direction. Note that V_n = 975 kips is less than the upper limit on shear strength, which is $8A_{cv}\sqrt{f_c'} = 8 \times 210 = 1{,}680$ kips (ACI 21.6.4.4).

Reinforcement ratio ρ_v for the vertical reinforcement must not be less than ρ_n when $h_w / \ell_w \leq 2.0$ (ACI 21.6.4.3). Since $h_w / \ell_w =$ 3.2 > 2, use minimum reinforcement ratio of 0.0025.

Use 2 curtains of No. 4 bars spaced at 12 in. on center in the vertical direction (ρ_v = 0.0028 > 0.0025).

Design for axial force and bending.

Figure 3-17 contains the interaction diagram of the wall reinforced with 2-No. 4 vertical bars @ 12 in. and 6-No. 10 bars at each end of the wall. As seen from the figure, the wall is adequate for the load combinations in Table 3-36.

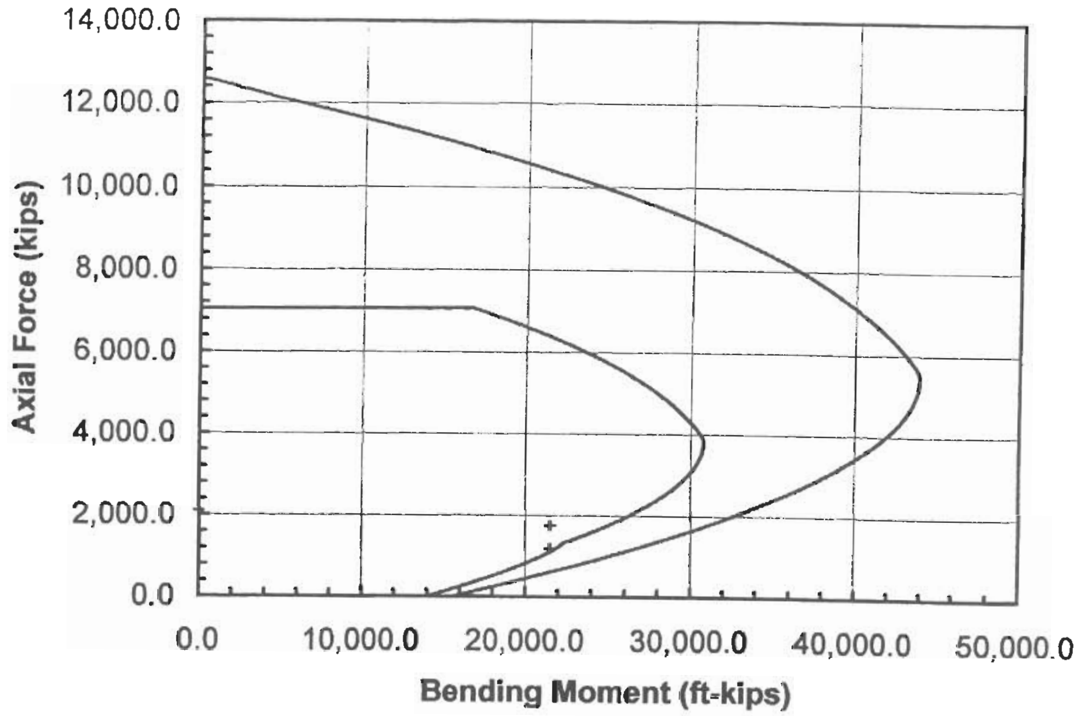

Figure 3-17 Design and Nominal Strength Interaction Diagrams for the Shear Wall Along Line 4 (SDC D)

Special boundary elements.

The need for special boundary elements at the edges of structural walls is evaluated in accordance with ACI 21.6.6.2 or 21.6.6.3. The displacement-based approach in ACI 21.6.6.2 is utilized in this example. In this method, the wall is displaced at the top an amount equal to the expected design displacement; special boundary elements are required to confine the concrete when the strain in the extreme compression fiber of the wall exceeds a critical value. This method is applicable to walls or wall piers that are essentially continuous in cross-section over the entire height and designed to have one critical section for flexure and axial loads.

Compression zones are to be reinforced with special boundary elements where (Eq. 21-8):

$$c \geq \frac{\ell_w}{600(\delta_u / h_w)}, \quad \delta_u / h_w \geq 0.007$$

where c = distance from extreme compression fiber to the neutral axis per ACI 10.2.7 calculated for the factored axial force and nominal moment strength, consistent with the design displacement δ_u, resulting in the largest neutral axis depth

ℓ_w = length of entire wall or segment of wall considered in the direction of the shear force

δ_u = design displacement

= total lateral displacement expected for the design-basis earthquake as specified by the governing code

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

The lower limit on the quantity δ_u / h_w is specified to require moderate wall deformation capacity for stiff buildings.

In this example, ℓ_w = 23 ft = 276 in., h_w = 74 ft = 888 in., δ_u is equal to δ_x from Table 3-29, which is 2.15 in. at the top of the wall, and $\delta_u / h_w = 0.0024 < 0.007$ (use 0.007). Therefore, special boundary elements are required if c is greater than or equal to $276/(600 \times 0.007) = 65.7$ in.

The distance c to be used in Eq. 21-8 is the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with the design displacement δ_u. From a strain compatibility analysis, the largest c is equal to 60.0 in. corresponding to a factored axial load of 1,738 kips and nominal moment strength of 30,739 ft-kips, which is less than 65.7 in. Therefore, special boundary elements are not required.

Where special boundary elements are not required according to ACI 21.6.6.2, the provisions of ACI 21.6.5.5 must be satisfied. These provisions require boundary transverse reinforcement for walls with moderate amounts of boundary longitudinal reinforcement to help prevent buckling of the longitudinal bars.

Boundary transverse reinforcement in accordance with ACI 21.4.4.1(c), 21.4.4.3, and 21.6.6.4(a) must be provided at the ends of walls where the longitudinal reinforcement ratio at the wall boundary is greater than $400/f_y$. In this example, the reinforcement layout is similar to that shown in the upper portion of ACI Figure R21.6.6.5, i.e., larger longitudinal bars concentrated at the ends of the wall. The longitudinal reinforcement ratio ρ at the wall boundary is (see Figure 3-18):

$$\rho = \frac{6 \times 1.27}{12[10 + (2 \times 1.89)]} = 0.0461 > \frac{400}{60,000} = 0.0067$$

Therefore, provide transverse reinforcement in accordance with ACI 21.4.4.1(c), 21.4.4.3, and 21.6.6.4(a) at ends of wall. The maximum spacing of the transverse reinforcement is 8 in. (ACI 21.6.6.5(a)).

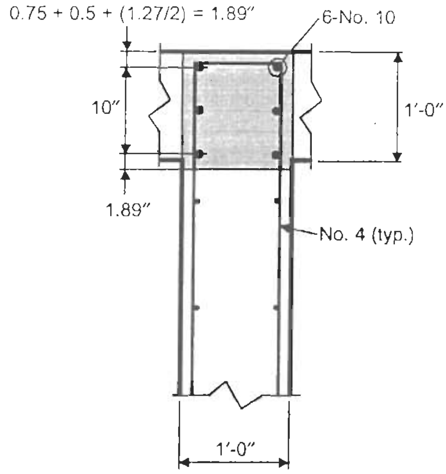

Figure 3-18 Longitudinal Reinforcement Ratio at Wall Boundary

The boundary elements must extend horizontally from the extreme compression fiber a distance not less than the larger of the following (ACI 21.6.6.4):

- $c - 0.1\ell_w = 60.0 - (0.1 \times 276) = 32.4$ in. (governs)
- $c/2 = 60.0/2 = 30.0$ in.

Based on the 12 in. spacing of the No. 4 vertical bars in the web, provide confinement over a length of 42 in. at each end of the wall.

Since $V_u = 330$ kips > $A_{cv}\sqrt{f_c'} = 210$ kips, horizontal reinforcement terminating at the edges of the wall must have a standard hook engaging the edge reinforcement or the edge reinforcement must be enclosed in U-stirrups that are spliced to the horizontal reinforcement; the U-stirrups must have the same size and spacing as the horizontal reinforcement (ACI 21.6.6.5(b)).

Splice length of reinforcement.

Class B lap splices are utilized for the longitudinal reinforcement at the ends of the wall and the vertical bars in the web. Mechanical connectors may be considered as an alternative to lap splices for the large bars at the ends of the wall. No splices are required for the No. 4 horizontal bars in the web, since full length bars weigh approximately $0.668 \times 23 = 15$ lbs. and are easily installed.

Vertical bars at ends of wall. For the No. 10 vertical bars at the ends of the wall, ℓ_d is determined from Eq. (12-1):

$$\frac{\ell_d}{d_b} = \frac{3}{40}\frac{f_y}{\sqrt{f_c'}}\frac{\alpha\beta\gamma\lambda}{\left(\frac{c+K_{tr}}{d_b}\right)}$$

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 7 and larger bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 0.75 + 0.50 + \dfrac{1.27}{2} = 1.89 \text{ in.} & \text{(governs)} \\ \dfrac{5}{2} = 2.5 \text{ in.} & \end{cases}$$

K_{tr} = transverse reinforcement index = 0 (conservative)

$$\frac{c+K_{tr}}{d_b} = \frac{1.89+0}{1.27} = 1.5 < 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.0 \times 1.0 \times 1.0 \times 1.0}{1.5} = 47.4$$

$$\ell_d = 47.4 \times 1.27 = 60.2 \text{ in.}$$

Class B splice length = $1.3\ell_d$ = 1.3 × 60.2 = 78.3 in.

Use a 6 ft-7 in. splice length for the No. 10 bars.

Vertical bars in wall web.

$$\frac{\ell_d}{d_b} = \frac{3}{40}\frac{f_y}{\sqrt{f_c'}}\frac{\alpha\beta\gamma\lambda}{\left(\frac{c+K_{tr}}{d_b}\right)}$$

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 0.8 for No. 6 and smaller bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 0.75 + 0.5 + \dfrac{0.5}{2} = 1.5 \text{ in.} & \text{(governs)} \\ \\ \dfrac{12}{2} = 6.0 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index = 0

$$\frac{c + K_{tr}}{d_b} = \frac{1.5 + 0}{0.5} = 3.0 > 2.5, \quad \text{use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{0.8 \times 1.0 \times 1.0 \times 1.0}{2.5} = 22.8$$

$$\ell_d = 22.8 \times 0.5 = 11.4 \text{ in.} < 12.0 \text{ in.}, \quad \text{use } 12.0 \text{ in.}$$

Class B splice length = $1.3\ell_d$ = 1.3 × 12.0 = 15.6 in.

Use a 1 ft-4 in. splice length for the No. 4 vertical bars in the web.

Reinforcement details for the shear wall along line 4 are shown in Figure 3-19.

3.6 DESIGN FOR SDC D – CALIFORNIA

The 5-story residential building in Figure 3-20 is assumed to be located in San Francisco. Typical structural members are designed and detailed for combined effects of gravity, wind, and seismic forces.

3.6.1 Design Data

- Building Location: San Francisco, CA (zip code 94105)
- Material Properties

  Concrete: $f'_c = 4{,}000$ psi, $w_c = 150$ pcf
  Reinforcement: $f_y = 60{,}000$ psi

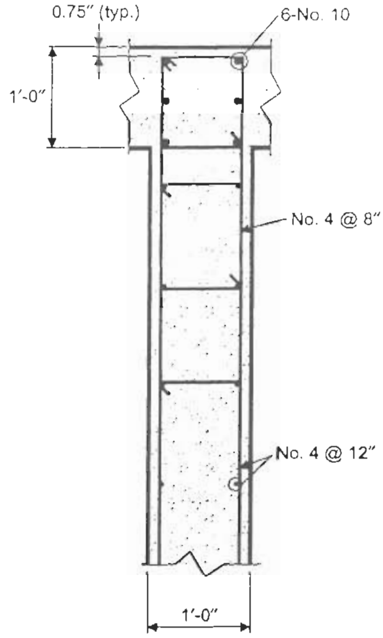

Figure 3-19 Reinforcement Details for Shear Wall Along Line 4 (SDC D)

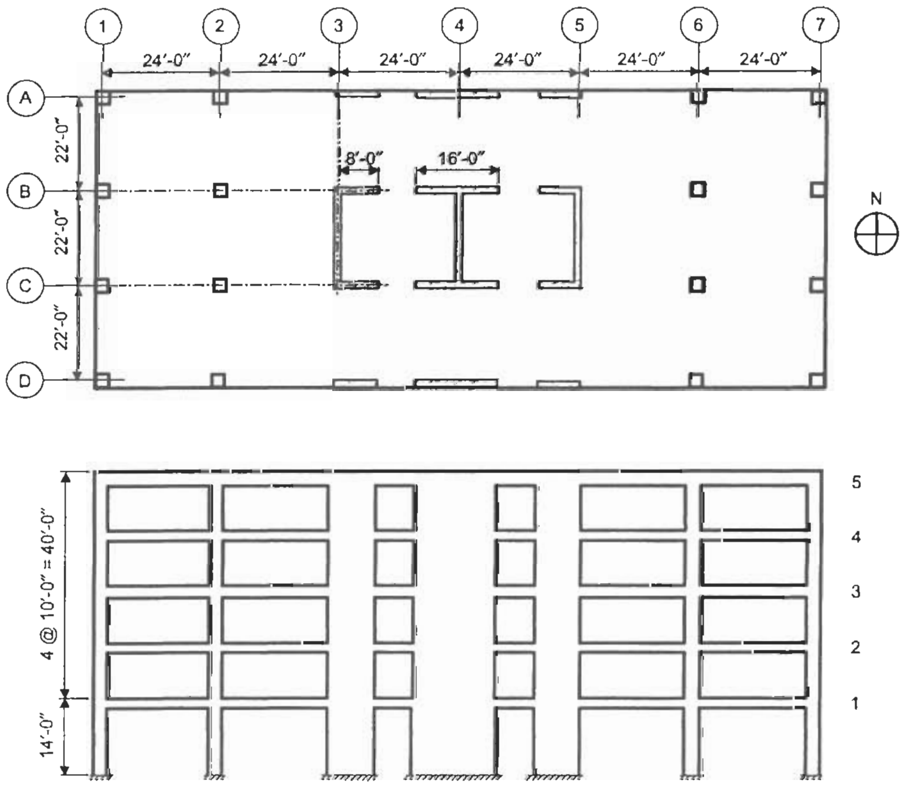

Figure 3-20 Typical Plan and Elevation of Example Building (SDC D)

- Service Loads

  Live loads: roof = 20 psf
  floor = 50 psf

  Superimposed dead loads: roof = 10 psf + 200 kips for penthouse
  floor = 30 psf (20 psf permanent partitions + 10 psf ceiling, etc.)

- Seismic Design Data

  For zip code 94105: $S_S = 1.50g, S_1 = 0.61g$ [3.1]
  Site Class D (stiff soil profile; IBC Table 1615.1.1)
  For Category (Seismic Use Group) I occupancy, $I_E = 1.0$ (IBC Table 1604.5)

- Wind Design Data

  Basic wind speed = 85 mph (IBC Figure 1609)
  Exposure B (IBC 1609.4)
  For Category I occupancy, $I_W = 1.0$ (IBC Table 1604.5)

- Member Dimensions

  Slab: 9 in.
  Columns: 28 × 28 in.
  Wall thickness: 12 in. in both directions

3.6.2 Seismic Load Analysis

3.6.2.1 Seismic Design Category (SDC)

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1-second period S_{M1} are determined from IBC Eqs. 16-16 and 16-17, respectively. According to IBC 1616.6.3, for regular structures 5 stories or less having a period determined in accordance with IBC 1617.4.2 of 0.5 sec or less, S_{DS} and S_{D1} need not exceed the values calculated using values of S_S = 1.50g and S_1 = 0.60g, respectively. The 5-story example building is regular according to IBC 1616.6.3, and it is shown below that its period is less than 0.5 sec. Therefore,

$$S_{MS} = F_a S_S = 1.0 \times 1.50 = 1.50g$$

$$S_{M1} = F_v S_1 = 1.5 \times 0.60 = 0.90g$$

where F_a and F_v are contained in IBC Table 1615.1.2(1) and Table 1615.1.2(2), respectively.

Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from Eqs. 16-18 and 16-19:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 1.50 = 1.00g$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.90 = 0.60g$$

Tables 1616.3(1) and 1616.3(2) are utilized to determine the SDC. From Table 1616.3(1) for Seismic Use Group I and S_{DS} = 1.00*g*, the SDC is D. Similarly, from Table 1616.3(2), the SDC is D for $S_{D1} = 0.60g$. Thus, the SDC is D for this building.

3.6.2.2 Seismic Forces

Since the building does not have plan irregularity Type 1a, 1b, or 4 of Table 1616.5.1 or vertical irregularity Type 1a, 1b, 4, or 5 of Table 1616.5.2, it can be considered regular (IBC 1616.6.3). For this regular building that is less than 240 ft in height, Table 1616.6.3 allows the equivalent lateral force procedure in IBC 1617.4 to be used to compute the seismic base shear V (see Eq. 16-34):

$$V = C_s W$$

where C_s is the seismic response coefficient determined in accordance with IBC 1617.4.1.1 and W is the effective weight of the structure defined in IBC 1617.4. For the member sizes and superimposed dead loads given above, W = 8,970 kips (see Table 3-37 below).

In both directions, a building frame system with special reinforced concrete shear walls is utilized, which is permitted for structures assigned to SDC D with a height less than or equal to 160 ft (see IBC Table 1617.6 and IBC 1910.5). The response modification coefficient R = 6 and the deflection amplification factor C_d = 5 are found in IBC Table 1617.6.

Approximate period (T_a). The fundamental period of the building T is determined in accordance with IBC 1617.4.2. In lieu of a more exact analysis, an approximate fundamental period T_a is computed from Eq. 16-39:

Building height h_n = 54 ft

Building period coefficient C_T = 0.02

Period $T_a = C_T(h_n)^{3/4} = 0.020 \times (54)^{3/4} = 0.40$ sec

Seismic base shear (*V*). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.60}{\left(\frac{6}{1.0}\right) \times 0.40} = 0.250$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{1.00}{\left(\frac{6}{1.0}\right)} = 0.167$$

Also, C_s must not be less than the following value from Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 1.00 \times 1.0 = 0.044$$

For buildings assigned to SDC E or F and for those buildings for which $S_1 \geq 0.6g$, C_s shall not be taken less than that computed from Eq. 16-38. Since $S_1 = 0.60g$, Eq. 16-38 is applicable, even though the SDC is D:

$$C_s = \frac{0.5 S_1}{R / I_E} = \frac{0.5 \times 0.60}{6/1.0} = 0.050$$

In this case, the lower limit is 0.050 from Eq. 16-38.

Thus, the value of C_s from Eq. 16-35 governs so that the base shear *V* in the N-S and E-W directions is:

$$V = C_s W = 0.167 \times 8{,}970 = 1{,}498 \text{ kips}$$

Vertical distribution of seismic forces. The total base shear is distributed over the height of the building in conformance with Eqs. 16-41 and 16-42:

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

where F_x is the lateral force induced at level x, w_x and w_i are the portions of W assigned to levels x or i, and k is the distribution exponent defined in IBC 1617.4.3. For $T = 0.40$ sec, $k = 1.0$. The lateral forces F_x and the story shears V_x are contained in Table 3-37.

Table 3-37 Seismic Forces and Story Shears (SDC D)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
5	1,621	54	87,534	438	438
4	1,816	44	79,904	400	838
3	1,816	34	61,744	309	1,147
2	1,816	24	43,584	218	1,365
1	1,901	14	26,614	133	1,498
Σ	8,970		299,380	1,498	

3.6.2.3 Method of Analysis

The method of analysis described in Section 3.5.2.3 of this publication is the same method of analysis utilized here. The shear walls are part of the SFRS and the slab and columns are not. Lateral displacements δ_{xe} and δ'_{xe} are contained in Table 3-38.

Table 3-38 Lateral Displacements (in.) due to Seismic Forces in N-S Direction (SDC D)

Story	δ_{xe} (in.)	δ_x (in.)	Δ (in.)	δ'_{xe} (in.)	δ'_x (in.)	δ_x / δ'_x
5	0.39	1.95	0.40	0.35	1.75	1.11
4	0.31	1.55	0.45	0.28	1.40	1.11
3	0.22	1.10	0.40	0.20	1.00	1.10
2	0.14	0.70	0.40	0.13	0.65	1.08
1	0.06	0.30	0.30	0.06	0.30	1.00

3.6.2.4 Story Drift and P-delta Effects

Story drift determination. Table 3-38 contains the displacements δ_{xe} of the SFRS in the N-S direction obtained from the 3-D static, elastic analysis using the design seismic forces, including accidental torsional effects, and the design earthquake displacement δ_x

computed by IBC Eq. 16-46. As noted above, C_d is equal to 5 for this system. The interstory drifts Δ computed from the δ_x are also contained in the table.

The design story drifts Δ must not exceed the allowable story drift Δ_a from Table 1617.3 (IBC 1617.3). For the 10-ft story heights, $\Delta_a = 0.020 \times 10 \times 12 = 2.40$ in. and for the 14-ft story height at the first level, $\Delta_a = 3.36$ in. It is evident from Table 3-38 that for all stories, the lateral drifts obtained from the prescribed lateral forces in the N-S direction are less than the limiting values.

Similar calculations for seismic forces in the E-W direction also show that the lateral drifts are less than the allowable values.

P-delta effects. As noted above, P-delta effects were automatically considered in the analysis using SAP2000. Section 3.5.2.4 of this publication illustrates the procedure to determine whether P-delta effects need to be considered or not in accordance with IBC 1617.4.6.2. Similar calculations for this example show that P-delta effects need not be considered.

3.6.3 Wind Load Analysis

In this example, the wind velocity is 85 mph, which produces wind forces that are significantly smaller than the seismic forces computed above. Thus, wind forces are not considered in this example.

3.6.4 Design for Combined Load Effects

3.6.4.1 Load Combinations

The following load combinations are applicable to structural members that are part of the SFRS:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)
4. $1.2D + f_1 L + 1.0E$ (Formula 16-5)
5. $0.9D + 1.0E$ (Formula 16-6)

where D, L, W, and E are the effects due to dead, live, wind, and seismic loads, respectively, and f_1 is equal to either 1.0 for places of public assembly, for live loads in excess of 100 psf, and for parking garage live load, or is equal to 0.5 for other live loads.

The seismic load effect E for use in Formula 16-5, which is the combined effect of horizontal and vertical earthquake-induced forces, is computed from Eq. 16-28 where the effects of gravity and seismic ground motion are additive:

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of horizontal seismic forces
ρ = redundancy coefficient determined in accordance with IBC 1617.2.2 for SDC D, E, or F

Similarly, E for use in Formula 16-6 is computed from Eq. 16-29 where the effects of gravity and seismic ground motion counteract:

$$E = \rho Q_E - 0.2 S_{DS} D$$

According to IBC 1617.2.2, the redundancy coefficient ρ, which shall not be less than 1.0 and need not exceed 1.5, is the largest of the values of ρ_i calculated at each story i from Equation 16-32:

$$\rho_i = 2 - \frac{20}{r_{\max_i} \sqrt{A_i}}$$

For shear walls:

$r_{\max_i}$ = (maximum wall shear × $10/\ell_w$)/total story shear

ℓ_w = length of the wall in feet

For the building in Figure 3-20, the shear wall on line 3 or 5 will have the largest shear force at its base, depending on which direction the center of mass is displaced. Therefore,

$$r_{\max_1} = \frac{530 \times \frac{10}{23}}{1{,}498} = 0.15$$

$$\rho_{\max} = 2 - \frac{20}{0.15\sqrt{146.33 \times 68.33}} = 0.67 < 1.0$$

Use ρ = 1.0.

Substituting S_{DS} = 1.00g and ρ = 1.0 into the equations for E, and then substituting E along with f_1 = 0.5 into load combinations 4 and 5 above results in the following:

4a. $1.2D + 0.5L + 1.0Q_E + (0.2 \times 1.00)D = 1.4D + 0.5L + Q_E$

4b. $1.2D + 0.5L + 1.0Q_E - (0.2 \times 1.00)D = D + 0.5L + Q_E$

5a. $0.9D + 1.0Q_E + (0.2 \times 1.00)D = 1.1D + Q_E$

5b. $0.9D + 1.0Q_E - (0.2 \times 1.00)D = 0.7D + Q_E$

Values of Q_E are obtained from the structural analysis for both sidesway to the left and to the right based on the code-prescribed horizontal seismic forces acting on the building.

The exception in IBC 1617.6.4.3 requires that reinforced concrete members that are not part of the SFRS comply with ACI 21.9. Therefore, the load combinations for these members are:

- $1.05D + 1.28L + E$
- $0.9D + E$

where the values of E are the forces in the frame members due to the lateral displacements δ_x being applied to the frames. When the approximate method outlined in Section 3.4.2.3 is utilized, $E = F_M$.

Also, the special seismic load combinations in IBC 1605.4 are not applicable in this example.

3.6.4.2 Slab Design

Preliminary Slab Thickness.

As noted above, the flat plate is not part of the SFRS. Thus, the requirements of ACI 21.9 for frame members not proportioned to resist forces induced by earthquake motions must be satisfied (IBC 1617.6.4.3). Since factored gravity axial forces on the slab are less than $A_g f_c' / 10$, either ACI 21.9.2 or 21.9.3 apply, depending on whether or not the earthquake-induced bending moments and shear forces due to the design displacements exceed the design moment and shear strength of the slab. In the supplement to the 2000 IBC [3.9], ACI 21.9.2.1 and 21.9.3.2 were modified to include specific requirements for two-way slabs. In particular, stirrups need not be provided in two-way slabs with column-line beams that are the same thickness as the slab. Shear reinforcement in the slab is to be provided in accordance with ACI 11.12 where required.

Calculations given in Section 3.2.4.2 of this publication show that a 9-in. thick slab and 22-in. square columns are adequate for serviceability and shear strength requirements for gravity loads. A more refined check for shear strength is made at a later stage in this example for a 9-in. thick slab and 28-in. square columns for effects from gravity loads and combined effects from gravity loads and earthquake-induced displacements.

Design for Flexure.

In this example, gravity load moments computed from the Direct Design Method are the same as those given in Table 3-32.

The reactions in the slab due to seismic displacements δ_x are determined using the approximate method outlined previously. Table 3-39 contains the bending moments M'_E in the slab at the fourth floor level obtained from a 3-D analysis of the shear walls and frames acting together subjected to the seismic forces in Table 3-37. The table also contains the approximate compatibility bending moments $M_E = C_d(\delta_x / \delta'_x)M'_E$, where the ratio (δ / δ'_x) at the fourth floor level is equal to 1.11 (see Table 3-38).

Table 3-39 Compatibility Bending Moments (ft-kips) in N-S Interior Design Strip (SDC D)

Location		M'_E	M_E
End Span	Exterior Negative	± 24.4	± 135.4
	Interior Negative	± 24.2	± 134.3
Interior Span	Interior Negative	± 24.0	± 133.2

A summary of the governing design bending moments at the fourth floor level is contained in Table 3-40.

The required flexural reinforcement is contained in Table 3-41. The provided areas of steel are greater than the minimum required (ACI 13.3.1). Also, the provided spacing is less than the maximum allowed according to ACI 13.3.2.

The amount of slab reinforcement needs to be checked at the end support and the first interior support to ensure that the moment transfer requirements in ACI 13.5.3 are satisfied. It can be shown that 1 additional No. 5 top bar is required in the column strip at the end support to satisfy moment transfer requirements. Also, 2 additional No. 5 bottom bars are required in the column strip in the end span to resist the 96.6 ft-kip positive moment at the end support (see Table 3-40).

Design for Shear.

Shear strength needs to be checked at the slab-column connections at the end support and the first interior support for the effects from gravity loads and combined effects from gravity loads and earthquake-induced displacements. Once again, calculations for this example are very similar to those in Section 3.5.4.2 and are not repeated here. It can be shown that the shear strength requirements for effects from gravity loads and combined effects from gravity loads and earthquake-induced displacements are satisfied at the end and interior columns. The largest shear stress is due to the combined effects from gravity loads and earthquake-induced displacements and occurs at the interior support. This shear stress is equal to 208.2 psi, which is less than the allowable shear stress of 215 psi.

Table 3-40 Summary of Slab Design Bending Moments (ft-kips) at Floor Level 4 (SDC D)

Load Case	Location	End Span Column Strip	End Span Middle Strip	Interior Span Column Strip	Interior Span Middle Strip
Dead (D)	Ext. neg.	-43.2	0		
	Positive	51.5	34.9	34.9	23.2
	Int. neg.	-88.0	-28.2	-81.3	-26.6
Live (L)	Ext. neg.	-10.6	0		
	Positive	12.6	8.5	8.5	5.7
	Int. neg.	-21.5	-6.9	-19.9	-6.5
Seismic (E)	Ext. neg.	± 135.4			
	Positive				
	Int. neg.	± 134.3		± 133.2	
Load Combinations					
$1.4D + 1.7L$	Ext. neg.	-78.5	0		
	Positive	93.5	63.3	63.3	42.2
	Int. neg.	-159.8	-51.2	-147.7	-48.3
$1.05D + 1.28L + E$	Ext. neg.	-194.2	0		
	Positive	70.2	47.5	47.5	31.7
	Int. neg.	-254.0	-38.4	-244.0	-36.3
$0.9D + E$	Ext. neg.	96.6	0		
	Positive	46.4	31.4	31.4	20.9
	Int. neg.	55.0	-25.4	60.0	-23.9

Table 3-41 Required Slab Reinforcement at Floor Level 4 (SDC D)

Location			M_u (ft-kips)	b (in.)	A_s^* (in.^{2})	Reinforcement*
End Span	Column strip	Ext. neg.	-194.2	132	5.87	19-No. 5
		Positive	93.5	132	2.74	9-No. 5
		Int. neg.	-254.0	132	7.81	26-No. 5
	Middle strip	Ext. neg.	0	156	2.53	9-No. 5
		Positive	63.3	156	2.53	9-No. 5
		Int. neg.	-51.2	156	2.53	9-No. 5
Interior Span	Column strip	Positive	63.3	132	2.14	8-No. 5
		Negative	-244.0	132	7.48	25-No. 5
	Middle strip	Positive	42.2	156	2.53	9-No. 5
		Negative	-48.2	156	2.53	9-No. 5

* Minimum $A_s = 0.0018bh = 0.0018 \times 132 \times 9 = 2.14$ in.^{2} (ACI 13.3.1)
$= 0.0018 \times 156 \times 9 = 2.53$ in.^{2}
Maximum spacing = $2h$ = 18 in. For b = 132 in., 132/18 = 7.3 spaces, say 8 bars
For b = 156 in., 156/18 = 8.6 spaces, say 9 bars

Reinforcement Details.

Slab reinforcement must conform to the requirements given in ACI 13.3. The provisions in ACI 13.3.8 must also be satisfied for slabs without beams; included are requirements for structural integrity (ACI 13.3.8.5).

Reinforcement details for this example are very similar to those shown in Figure 3-16.

3.6.4.3 Design of Column B2

As noted previously, column B2 is not part of the SFRS. The design of this member supporting the first floor level in this example building is very similar to the design for SDC C presented in Section 3.4.4.3 of the publication. It can be shown that a 28 × 28 in. column with 8-No. 9 bars ($\rho_g = 1.02\%$) is adequate for the load combinations. Reinforcement details for the transverse reinforcement are similar to those depicted in Figure 3-14.

3.6.4.4 Design of Shear Wall on Line 4

Table 3-42 contains a summary of the design axial forces, bending moments, and shear forces at the base of the wall. As noted above, this special reinforced concrete shear wall, as well as all of the other shear walls in the building, is part of the SFRS.

Table 3-42 Summary of Design Axial Forces, Bending Moments, and Shear Forces at Base of Shear Wall on Line 4 (SDC D)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (*D*)	918	0	0
Live (*L*)	111	0	0
Seismic (Q_E)	0	± 23,325	± 530
Load Combinations			
1.4*D* + 1.7*L*	1,474	0	0
1.4*D* + 0.5*L* + Q_E	1,341	23,325	530
0.7*D* + Q_E	643	-23,325	-530

Design for shear.

Reinforcement requirements. Special reinforced structural walls are to be provided with reinforcement in two orthogonal directions in the plane of the wall in accordance with ACI 21.6.2. The minimum reinforcement ratio in both directions is 0.0025, unless the design shear force is less than or equal to $A_{cv}\sqrt{f_c'}$, where A_{cv} is the gross area of concrete bounded by the web thickness and the length of wall in the direction of analysis.

In such cases, minimum reinforcement in accordance with ACI 14.3 for ordinary walls must be provided. For the wall in this example, A_{cv} = 12 × 276 = 3,312 in.^{2}, so that

$$A_{cv}\sqrt{f_c'} = 3{,}312 \times \sqrt{4{,}000}/1{,}000 = 210 \text{ kips} < V_u = 530 \text{ kips}$$

Therefore, the minimum reinforcement ratio is 0.0025 and the maximum spacing is 18 in. (ACI 21.6.2.1).

Two curtains of reinforcement are required in a wall when the in-plane factored shear force exceeds $2A_{cv}\sqrt{f_c'}$ = 2 × 210 = 420 kips. In this case, two curtains are required, since V_u = 530 kips > 420 kips. Note that V_u is less than the upper limit on shear strength, which is $\phi 8A_{cv}\sqrt{f_c'} = 0.85 \times 8 \times 210 = 1{,}428$ kips (ACI 21.6.4.4).

The minimum required reinforcement in each direction per foot of wall is equal to 0.0025 × 12 × 12 = 0.36 in.^{2} Assuming No. 4 bars in two curtains, required spacing s is

$$s = \frac{2 \times 0.20}{0.36} \times 12 = 13.3 \text{ in.} < 18 \text{ in.}$$

Try 2 curtains of No. 4 bars spaced at 12 in.

Shear strength requirements. ACI Eq. 21-7 is used to determine nominal shear strength V_n of structural walls:

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

where $\alpha_c = 2$ for ratio of wall height to length h_w/ℓ_w = 54/23 = 2.4 > 2 (ACI 21.6.4.1).

For 2 curtains of No. 4 horizontal bars spaced at 12 in. ($\rho_n = 0.40/(12 \times 12) = 0.0028$):

$$\phi V_n = 0.85 \times 3{,}312 \times [2\sqrt{4{,}000} + (0.0028 \times 60{,}000)]/1{,}000$$

$$= 829 \text{ kips} > V_u = 530 \text{ kips} \qquad \text{O.K.}$$

where $\phi = 0.85$ for walls with $h_w/\ell_w > 2$ (ACI 9.3.4(a)). Therefore, use 2 curtains of No. 4 bars @ 12 in. on center in horizontal direction.

Reinforcement ratio ρ_v for the vertical reinforcement must not be less than ρ_n when $h_w/\ell_w \leq 2.0$ (ACI 21.6.4.3). Since h_w/ℓ_w = 2.4 > 2, use minimum reinforcement ratio of 0.0025.

Use 2 curtains of No. 4 bars spaced at 12 in. on center in the vertical direction (ρ_v = 0.0028 > 0.0025).

Design for axial force and bending.

Figure 3-21 contains the interaction diagram of the wall reinforced with 2-No. 4 vertical bars @ 12 in. and 8-No. 11 bars at each end of the wall. As seen from the figure, the wall is adequate for the load combinations in Table 3-42.

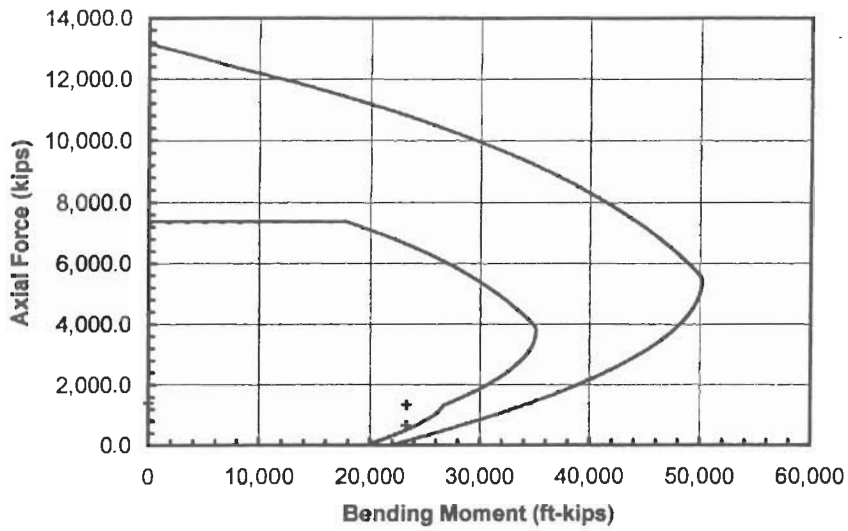

Figure 3-21 Design and Nominal Strength Interaction Diagrams for the Shear Wall Along Line 4 (SDC D)

Special boundary elements.

The need for special boundary elements at the edges of structural walls is evaluated in accordance with ACI 21.6.6.2 or 21.6.6.3. The displacement-based approach in ACI 21.6.6.2 is utilized in this example. In this method, the wall is displaced at the top an amount equal to the expected design displacement; special boundary elements are required to confine the concrete when the strain in the extreme compression fiber of the wall exceeds a critical value. This method is applicable to walls or wall piers that are essentially continuous in cross-section over the entire height and designed to have one critical section for flexure and axial loads.

Compression zones are to be reinforced with special boundary elements where (Eq. 21-8):

$$c \geq \frac{\ell_w}{600(\delta_u / h_w)}, \quad \delta_u / h_w \geq 0.007$$

where c = distance from extreme compression fiber to the neutral axis per ACI 10.2.7 calculated for the factored axial force and nominal moment strength, consistent with the design displacement δ_u, resulting in the largest neutral axis depth

ℓ_w = length of entire wall or segment of wall considered in the direction of the shear force

δ_u = design displacement

= total lateral displacement expected for the design-basis earthquake as specified by the governing code

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

The lower limit on the quantity δ_u / h_w is specified to require moderate wall deformation capacity for stiff buildings.

In this example, ℓ_w = 23 ft = 276 in., h_w = 54 ft = 648 in., δ_u is equal to δ_x from Table 3-38, which is 1.95 in. at the top of the wall, and $\delta_u / h_w = 0.0030 < 0.007$ (use 0.007). Therefore, special boundary elements are required if c is greater than or equal to $276/(600 \times 0.007) = 65.7$ in.

The distance c to be used in Eq. 21-8 is the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with the design displacement δ_u. From a strain compatibility analysis, the largest c is equal to 50.1 in. corresponding to a factored axial load of 1,341 kips and nominal moment strength of 34,123 ft-kips, which is less than 65.7 in. Therefore, special boundary elements are not required.

Where special boundary elements are not required according to ACI 21.6.6.2, the provisions of ACI 21.6.5.5 must be satisfied. These provisions require boundary transverse reinforcement for walls with moderate amounts of boundary longitudinal reinforcement to help prevent buckling of the longitudinal bars.

Boundary transverse reinforcement in accordance with ACI 21.4.4.1(c), 21.4.4.3, and 21.6.6.4(a) must be provided at the ends of walls where the longitudinal reinforcement ratio at the wall boundary is greater than $400/f_y$. In this example, the reinforcement layout is similar to that shown in the upper portion of ACI Figure R21.6.6.5, i.e., larger longitudinal bars concentrated at the ends of the wall. The longitudinal reinforcement ratio ρ at the wall boundary is (see Figure 3-22):

$$\rho = \frac{8 \times 1.56}{12[11 + (2 \times 1.96)]} = 0.0697 > \frac{400}{60,000} = 0.0067$$

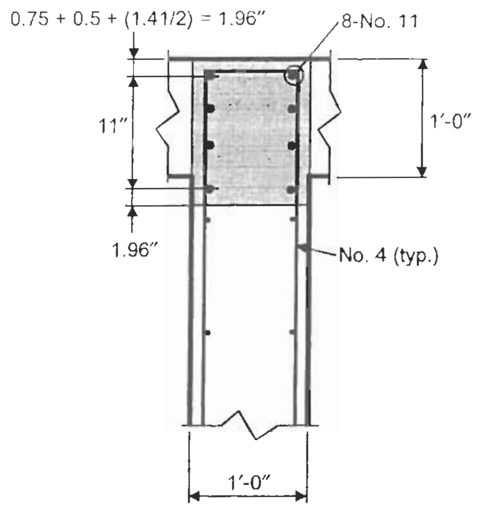

Figure 3-22 Longitudinal Reinforcement Ratio at Wall Boundary

Therefore, provide transverse reinforcement in accordance with ACI 21.4.4.1(c), 21.4.4.3, and 21.6.6.4(a) at ends of wall. The maximum spacing of the transverse reinforcement is 8 in. (ACI 21.6.6.5(a)).

The boundary elements must extend horizontally from the extreme compression fiber a distance not less than the larger of the following (ACI 21.6.6.4):

- $c - 0.1\ell_w = 50.1 - (0.1 \times 276) = 22.5$ in.
- $c/2 = 50.1/2 = 25.1$ in. (governs)

Based on the 12 in. spacing of the No. 4 vertical bars in the web, provide confinement over a length of 30 in. at each end of the wall.

Since $V_u = 530$ kips > $A_{cv}\sqrt{f_c'} = 210$ kips, horizontal reinforcement terminating at the edges of the wall must have a standard hook engaging the edge reinforcement or the edge reinforcement must be enclosed in U-stirrups that are spliced to the horizontal reinforcement; the U-stirrups must have the same size and spacing as the horizontal reinforcement (ACI 21.6.6.5(b)).

Splice lengths of reinforcement are computed similar to that shown in Section 3.5.4.4.

Reinforcement details for the shear wall along line 4 are shown in Figure 3-23.

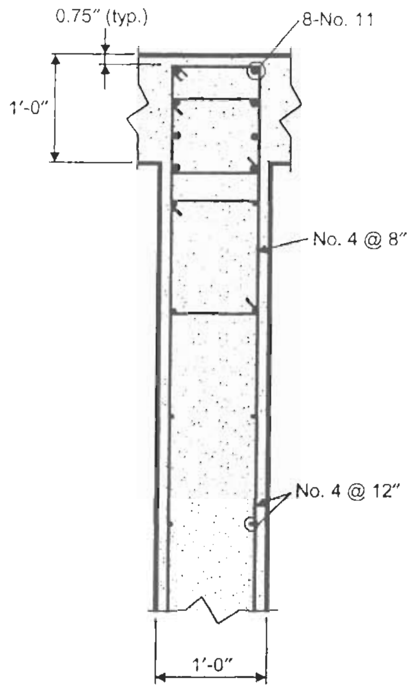

Figure 3-23 Reinforcement Details for Shear Wall Along Line 4 (SDC D)

3.7 DESIGN FOR SDC E

The 5-story residential building in Figure 3-20 is assumed to be located in Berkeley, CA.

3.7.1 Design Data

- Building Location: Berkeley, CA (zip code 94705)
- Material Properties

  Concrete: $f_c' = 4{,}000$ psi, $w_c = 150$ pcf
  Reinforcement: $f_y = 60{,}000$ psi
- Service Loads

  Live loads: roof = 20 psf
  floor = 50 psf

  Superimposed dead loads: roof = 10 psf + 200 kips for penthouse
  floor = 30 psf (20 psf permanent partitions + 10 psf ceiling, etc.)

- Seismic Design Data

  For zip code 94705: $S_S = 2.08g, S_1 = 0.92g$ [3.1]
  Site Class D (stiff soil profile; IBC Table 1615.1.1)
  For Category (Seismic Use Group) I occupancy, $I_E = 1.0$ (IBC Table 1604.5)

- Wind Design Data

  Basic wind speed = 85 mph (IBC Figure 1609)
  Exposure B (IBC 1609.4)
  For Category I occupancy, $I_W = 1.0$ (IBC Table 1604.5)

- Member Dimensions

  Slab: 9 in.
  Columns: 28 × 28 in.
  Wall thickness: 12 in. in both directions

3.7.2 Seismic Load Analysis

3.7.2.1 Seismic Design Category (SDC)

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1-second period S_{M1} are determined from IBC Eqs. 16-16 and 16-17, respectively. According to IBC 1616.6.3, for regular structures 5 stories or less having a period determined in accordance with IBC 1617.4.2 of 0.5 sec or less, S_{DS} and S_{D1} need not exceed the values calculated using values of S_S = 1.50g and S_1 = 0.60g, respectively. The 5-story example building is regular according to IBC 1616.6.3, and it was shown in Section 3.6.2.2 that its period is less than 0.5 sec. Therefore,

$$S_{MS} = F_a S_S = 1.0 \times 1.50 = 1.50g$$

$$S_{M1} = F_v S_1 = 1.5 \times 0.60 = 0.90g$$

where F_a and F_v are contained in IBC Table 1615.1.2(1) and Table 1615.1.2(2), respectively.

Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from Eqs. 16-18 and 16-19:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 1.50 = 1.00g$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.90 = 0.60g$$

Tables 1616.3(1) and 1616.3(2) are utilized to determine the SDC. From the footnote to Table 1616.3(1), Seismic Use Group I structures located on sites with mapped $S_1 \geq 0.75g$ shall be assigned to SDC E when $S_{DS} \geq 0.50g$. Similarly, from the footnote to Table 1616.3(2), the SDC is E when $S_{D1} \geq 0.20g$. Thus, the SDC is E for this building.

3.7.2.2 Seismic Forces

Due to the provisions in IBC 1616.6.3, the seismic forces for this example are the same as those shown in Table 3-37 for SDC D.

Design of the slab, columns, and shear walls are exactly the same as those presented in the previous section of this publication. Thus, calculations are not repeated here.

3.8 REFERENCES

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3.7 Fanella, D.A., Munshi, J.A., and Rabbat, B.G., (Eds.), *Notes on ACI 318-99 Building Code Requirements for Structural Concrete with Design Applications*, Portland Cement Association, Skokie, Il, 1999.

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3.9 International Code Council, *2002 Accumulative Supplement to the International Codes*, Falls Chuch, VA, 2002.

CHAPTER 4

SCHOOL BUILDING WITH MOMENT-RESISTING FRAME SYSTEM

4.1 INTRODUCTION

In a moment-resisting frame system (MRFS), an essentially complete space frame provides support for gravity loads, and moment-resisting frames provide resistance to lateral loads primarily by flexural action of the frame members. It is vital that a MRFS have a substantially complete vertical load-carrying space frame. Either the entire frame or portions of the frame can be designated as the moment-resisting frame that resists the lateral loads. Any members that are not included in the moment-resisting frame must satisfy the deformational compatibility requirements in IBC 1617.6.4.3 when applicable.

According to IBC Table 1617.6, moment-resisting frames can be ordinary moment-resisting frames (OMRF) in SDC A or B (IBC 1910.3). In such cases, the members can be designed according to ACI 318, exclusive of Chapter 21, with no special seismic detailing required. An intermediate moment-resisting frame (IMRF) is required in SDC C (IBC 1910.4). Design and detailing of frame members in an IMRF must conform to the requirements in ACI 21.2.1.3 and 21.10. In SDC D, E, and F, special moment-resisting frames must be utilized (IBC 1910.5), and they are to be designed and detailed in accordance with ACI 21.2 through 21.5.

This chapter illustrates the design and detailing of typical members in a structure with a MRFS assigned to SDC B, C, and D.

4.2 DESIGN FOR SDC B

4.2.1 Design Data

A typical plan and elevation of a 3-story school building is shown in Figure 4-1. The computation of wind and seismic forces according to the 2000 IBC is illustrated below. Typical structural members are designed and detailed for combined effects of gravity, wind, and seismic forces.

As noted above, ordinary reinforced concrete moment frames may be used for structures assigned to SDC B without any limitations according to IBC Table 1617.6. This type of system is utilized in this example.

For a building of this size, it is economical to have constant cross-sections for the members in the floor system and the columns throughout the height of the building. It is assumed for simplicity that the bases of the lowest story segments are fixed. Although the member dimensions in the following sections are within the practical range, the structure itself is a hypothetical one, and has been chosen mainly for illustrative purposes.

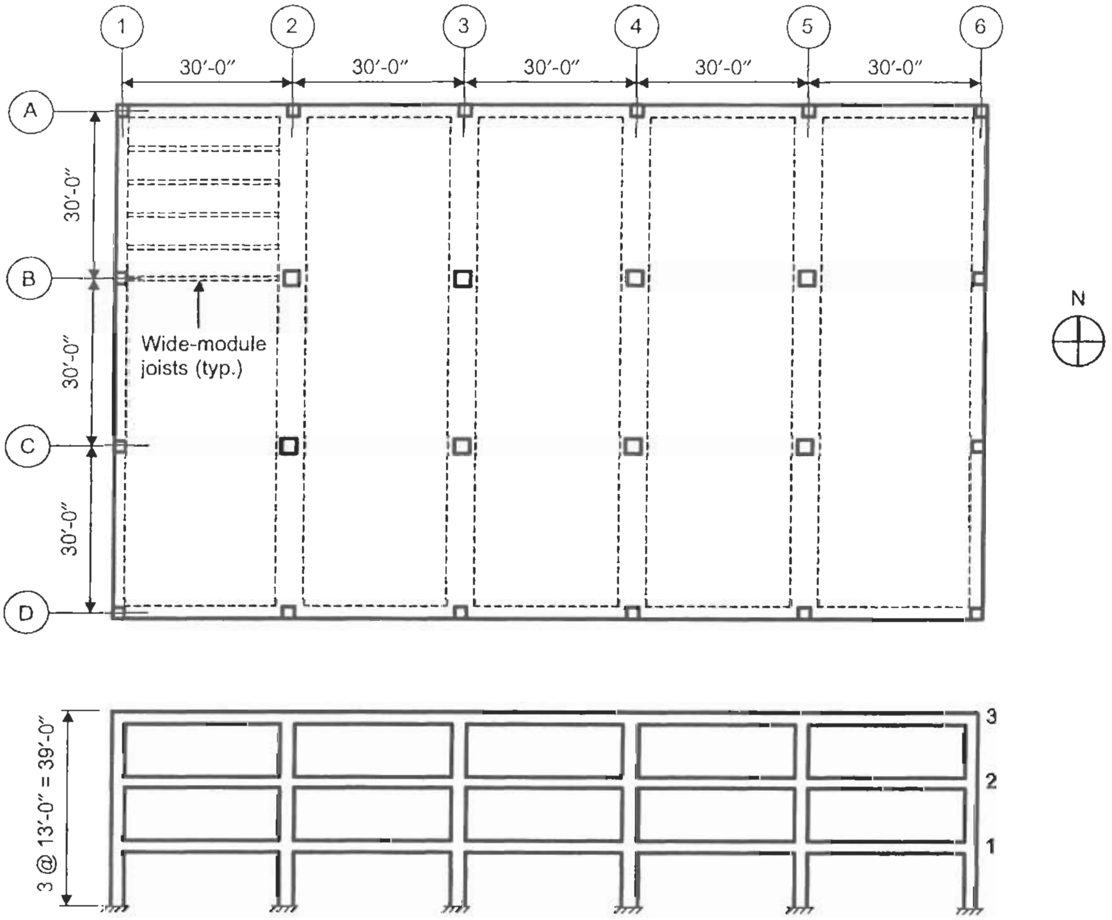


Figure 4-1 Typical Plan and Elevation of Example Building

- Building Location: Atlanta, GA (zip code 30350)
- Material Properties

  Concrete: $f'_c = 4{,}000$ psi, $w_c = 150$ pcf
  Reinforcement: $f_y = 60{,}000$ psi

- Service Loads

  Live loads: roof = 20 psf
  floor = 60 psf (average between 40 psf for floors and 80 psf for corridors above first floor level; IBC Table 1607.1). In addition, floor is to be checked for 1,000 lb concentrated live load uniformly distributed over an area of 2.5 sq ft (IBC 1607.4).

  Superimposed dead loads: roof = 10 psf + 200 kips for penthouse
  floor = 45 psf (20 psf permanent partitions + 25 psf ceiling, etc.)

- Seismic Design Data

  For zip code 30350: $S_S = 0.276g, S_1 = 0.117g$ [4.1]
  Site Class C (very dense soil and soft rock; IBC Table 1615.1.1)
  For Category (Seismic Use Group) II occupancy, $I_E = 1.25$ (IBC Table 1604.5)

- Wind Design Data

  Basic wind speed = 90 mph (IBC Figure 1609)
  Exposure B (IBC 1609.4)
  For Category II occupancy, $I_W = 1.15$ (IBC Table 1604.5)

- Member Dimensions

  Joists: 14 + 4.5 × 6 + 66 (74 psf)
  Interior beams: 28 × 18.5 in.
  Spandrel beams: 16 × 18.5 in.
  Interior columns: 20 × 20 in.
  Exterior columns: 16 × 16 in.

4.2.2 Seismic Load Analysis

4.2.2.1 Seismic Design Category (SDC)

Analysis procedures for seismic design are given in IBC 1616.6. The appropriate procedure to use depends on the Seismic Design Category (SDC), which is determined in accordance with IBC 1616.3. Structures are assigned to a SDC based on their Seismic Use Group and the design spectral response acceleration parameters S_{DS} and S_{D1}. These parameters can be computed from Eqs. 16-18 and 16-19 in IBC 1615.1.3 or can be obtained from the provisions of IBC 1615.2.5 where site-specific procedures are used as required or permitted by IBC 1615.

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1-second period S_{M1} are determined from IBC Eqs. 16-16 and 16-17, respectively:

$$S_{MS} = F_a S_S = 1.2 \times 0.276 = 0.33g$$

$$S_{M1} = F_v S_1 = 1.68 \times 0.117 = 0.20g$$

where F_a and F_v are the values of site coefficients as a function of site class and of mapped spectral response acceleration at short periods S_S and at 1-second period S_1, respectively. The values of these coefficients for Site Class C in the equations above are contained in IBC Table 1615.1.2(1) for F_a and Table 1615.1.2(2) for F_v. Straight-line interpolation was used to determine F_v.

Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from Eqs. 16-18 and 16-19:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 0.33 = 0.22g$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.20 = 0.13g$$

Tables 1616.3(1) and 1616.3(2) are utilized to determine the SDC. From Table 1616.3(1) for Seismic Use Group II and $S_{DS} = 0.22g$, the SDC is B. Similarly, the SDC is B for $S_{D1} = 0.13g$ from Table 1616.3(2). Thus, the SDC is B for this building.

4.2.2.2 Seismic Forces

According to IBC 1616.6.2, the equivalent lateral force procedure in IBC 1617.4 may be used to compute the seismic base shear V for structures assigned to SDC B. In a given direction, V is determined from Eq. 16-34:

$$V = C_s W$$

where C_s is the seismic response coefficient determined in accordance with IBC 1617.4.1.1 and W is the effective weight of the structure defined in IBC 1617.4. For the member sizes and superimposed dead loads given above, W = 5,547 kips (see Table 4-1 below).

As noted above, a moment-resisting frame system is utilized as the seismic-force-resisting system. As a minimum, this must be an ordinary reinforced concrete moment frame to satisfy the provisions of IBC 1910.3.1 for structures assigned to SDC B. For this system, the response modification coefficient $R = 3$ and the deflection amplification factor $C_d = 2.5$, which are found in IBC Table 1617.6.

Approximate period (T_a). The fundamental period of the building is determined in accordance with Eq. 16-39 in IBC 1617.4.2:

Building period coefficient $C_T = 0.03$

Building height $h_n = 39$ ft

Period $T_a = C_T (h_n)^{3/4} = 0.030 \times (39)^{3/4} = 0.47$ sec

Seismic base shear (V). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.13}{\left(\frac{3}{1.25}\right) \times 0.47} = 0.115$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{0.22}{\left(\frac{3}{1.25}\right)} = 0.092$$

Also, C_s must not be less than the following value from Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 0.22 \times 1.25 = 0.012$$

Thus, Eq. 16-35 governs, and the base shear V is:

$$V = C_s W = 0.092 \times 5{,}547 = 510 \text{ kips}$$

Vertical distribution of seismic forces. The lateral forces F_x and the story shears V_x are contained in Table 4-1. For $T = 0.47$ sec, $k = 1.0$.

4.2.2.3 Method of Analysis

A three-dimensional analysis of the building was performed in the N-S and E-W directions for the seismic forces contained in Table 4-1 using SAP2000 [4.2]. In the model, rigid diaphragms were assigned at each floor level, and rigid-end offsets were defined at the ends of the horizontal members so that results were automatically obtained at the faces of the supports. The stiffness properties of the members were input assuming cracked sections. In lieu of a more accurate analysis, the following cracked section properties were used:

- Beams: $I_{eff} = 0.5I_g$
- Columns: $I_{eff} = 0.7I_g$

where I_g is the gross moment of inertia of the section. P-delta effects were also considered in the analysis.

Table 4-1 Seismic Forces and Story Shears (SDC B)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
3	1,627	39	63,453	231	231
2	1,960	26	50,960	186	417
1	1,960	13	25,480	93	510
Σ	5,547		139,893	510	

In accordance with IBC 1617.4.4.4, the center of mass was displaced each way from its actual location a distance equal to 5 percent of the building dimension perpendicular to the applied forces to account for accidental torsion in seismic design.

4.2.2.4 Story Drift and P-delta Effects

Story drift determination. Table 4-2 contains the displacements δ_{xe} obtained from the 3-D static, elastic analyses using the design seismic forces in the N-S direction, including accidental torsional effects. The table also contains the design earthquake displacement δ_x computed by IBC Eq. 16-46:

$$\delta_x = \frac{C_d \delta_{xe}}{I_E}$$

where C_d is the deflection amplification factor in Table 1617.6, which depends on the seismic-force-resisting system. As noted above, C_d is equal to 2.5 for an ordinary reinforced concrete moment frame.

The interstory drifts Δ computed from the δ_x are also contained in the table. For this structure that does not have plan irregularity Type 1a or 1b of Table 1616.5.1, the drift at story level x is determined by subtracting the design earthquake displacement at the center of mass at the bottom of the story from the design earthquake displacement at the center of mass at the top of the story (IBC 1617.4.6.1):

$\Delta = \delta_x - \delta_{x-1}$

Table 4-2 Lateral Displacements and Interstory Drifts due to Seismic Forces in N-S Direction (SDC B)

Story	δ_{xe} (in.)	δ_x (in.)	Δ (in.)
3	1.74	3.48	0.94
2	1.27	2.54	1.42
1	0.56	1.12	1.12

The design story drifts Δ must not exceed the allowable story drift Δ_a from Table 1617.3 (IBC 1617.3). For Seismic Use Group II, $\Delta_a = 0.020h_{sx}$ for buildings 4 stories or less that have been designed to accommodate the story drifts. Thus, for the 13-ft story heights, $\Delta_a = 0.020 \times 13 \times 12 = 3.12$ in. It is evident from Table 4-2 that for all stories, the lateral drifts obtained are less than the limiting value.

P-delta effects. As noted above, P-delta effects were automatically considered in the analysis using SAP2000. The provisions of IBC 1617.4.6.2 are illustrated in Section 4.4.2.4 of this publication for SDC D.

4.2.3 Wind Load Analysis

4.2.3.1 Wind Forces

Wind forces are determined in accordance with the analytical procedure (Method 2) given in ASCE 6.5 [4.3].

The example building is regular shaped by the definition in ASCE 6.2, i.e., it has no unusual geometrical irregularity in spatial form. Also, the building does not have response characteristics making it subject to across wind loading, vortex shedding, or instability due to galloping or flutter. It is assumed that the site location is such that channeling effects or buffeting in the wake of upwind obstructions need not be considered. Thus, the analytical procedure (Method 2) of ASCE 6.5 may be used to determine the wind forces.

Design Procedure.

The design procedure outlined in Section 6.5.3 of ASCE 7 is used to determine the wind forces on the building in the N-S direction.

1. Basic wind speed, *V*, and wind directionality factor, K_d. Both quantities are determined in accordance with ASCE 6.5.4. As noted above, V is equal to 90 mph for Atlanta according to IBC Figure 1609 or ASCE Figure 6-1.

The wind directionality factor K_d is equal to 0.85 for main wind-force-resisting systems per ASCE Table 6-6 when load combinations specified in Sections 2.3 and 2.4 are used. Note that these load combinations are essentially the same as those in Sections 1605.2 and 1605.3 of the 2000 IBC. It is important to note exception 1 to IBC 1605.2.1, Basic load combinations: load combinations of ACI 9.2 shall be used for concrete structures where combinations do not include seismic forces. The load factors in the ACI 318 combinations are different than those in ASCE 7 and the IBC. The exception goes on to state that for concrete structures designed for wind in accordance with ASCE 7, wind forces are to be divided by the directionality factor. Thus, in the following computations, instead of multiplying and then subsequently dividing the external wind pressures/forces by 0.85, K_d is taken equal to 1.0.

2. Importance factor, I_W. As noted above, I_W is equal to 1.15 for Category II occupancy according to IBC Table 1604.5 and Category III occupancy according to ASCE Table 1-1 (note that IBC Category II and ASCE 7 Category III are the same).

3. Velocity pressure exposure coefficient, K_z. According to ASCE 6.5.6.4, values of K_z are to be determined from ASCE Table 6-5. In lieu of linear interpolation, K_z may be calculated at any height z above ground level from the equations given at the bottom of Table 6-5:

$$K_z = \begin{cases} 2.01\left(\dfrac{15}{z_g}\right)^{2/\alpha} & \text{for } z < 15\text{ ft} \\ 2.01\left(\dfrac{z}{z_g}\right)^{2/\alpha} & \text{for } 15\text{ ft} \le z \le z_g \end{cases}$$

where α = 3-second gust speed power law exponent from ASCE Table 6-4
= 7.0 for Exposure B

z_g = nominal height of the atmospheric boundary layer from ASCE Table 6-4
= 1,200 ft for Exposure B

Values of K_z are summarized in Table 4-3 at the various story heights for the example building.

Table 4-3 Velocity Pressure Exposure Coefficient K_z

Level	Height above ground level, z (ft)	K_z
3	39	0.755
2	26	0.673
1	13	0.575

4. Topographic factor, K_{zt}. The topographic factor is to be determined in accordance with ASCE 6.5.7, Eq. 6-1. Assuming the example building is situated on level ground and not on a hill, ridge, or escarpment, K_{zt} is equal to 1.

5. Gust effect factors, G and G_f. Effects due to wind gust depend on whether a building is rigid or flexible (ASCE 6.5.8). A rigid building has a fundamental natural frequency n_1 greater than or equal to 1 Hz, while a flexible building has a fundamental natural frequency less than 1 Hz (ASCE 6.2).

In lieu of a more exact method, the approximate fundamental period T_a = 0.47 sec determined in Section 4.2.2.2 is used. The natural frequency is computed by taking the inverse of the period: $n_1 = 1/0.47 = 2.1\text{ Hz}$.

Since n_1 > 1.0 Hz, the building is considered rigid, and G may be taken equal to 0.85 or may be calculated by Eq. 6-2 (ASCE 6.5.8.1). For simplicity, G is taken as 0.85.

6. Enclosure classification. It is assumed in this example that the building is enclosed per ASCE 6.5.9 or IBC 1609.2.

7. Internal pressure coefficient, GC_{pi}. According to ASCE 6.5.11.1, internal pressure coefficients are to be determined from Table 6-7 based on building enclosure classification. For an enclosed building, $GC_{pi} = \pm 0.18$.

8. External pressure coefficients, C_p. External pressure coefficients for main wind-force-resisting systems are given in Figure 6-3 for this example building. For wind in the N-S direction:

Windward wall: $C_p = 0.8$

Leeward wall (L/B = 91.33/151.33 = 0.60): $C_p = -0.5$

Side wall: $C_p = -0.7$

Roof (h/L = 39/91.33 = 0.43):

$C_p = -0.9$ from 0 to h = 39 ft from windward edge.

$C_p = -0.5$ from 39 ft from windward edge to $2h$ = 78 ft

$C_p = -0.3$ from 78 ft from windward edge to 91.33 ft

9. Velocity pressure, q_z. The velocity pressure at height z is determined from Eq. 6-13 in ASCE 6.5.10:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I$$

where all terms have been defined previously. Table 4-4 contains a summary of the velocity pressures for the example building.

Table 4-4 Velocity Pressure q_z (V = 90 mph)

Level	Height above ground level, z (ft)	K_z	q_z (psf)
3	39	0.755	18.0
2	26	0.673	16.1
1	13	0.575	13.7

10. Design wind pressure, *p*. Design wind pressures on the main wind-force-resisting systems of enclosed and partially enclosed buildings are determined in accordance with ASCE 6.5.12. For rigid buildings of all heights, design wind pressures are calculated from Eq. 6-15:

$$p = qGC_p - q_i(GC_{pi})$$

Tables 4-5 and 4-6 contain summaries of design pressures and forces, respectively, for wind in the N-S direction. It has been assumed that the design wind pressure is constant over the tributary height of the floor level.

Since the mean roof height of the building is less than 60 ft and does not exceed the least horizontal dimension (91.33 ft), this building is considered a low-rise building according to ASCE 6.2. Therefore, in lieu of using ASCE Eq. 6-15, the design wind pressure may be calculated from ASCE Eq. 6-16 (ASCE 6.5.12.2.2).

Table 4-5 Design Wind Pressures in N-S Direction (V = 90 mph)

Location	Level	Height above ground level, z (ft)	External Pressure				Internal Pressure		
			q (psf)	G	C_p	qGC_p (psf)	q_i (psf)	GC_{pi}	q_iGC_{pi} (psf)
Windward	3	39	18.0	0.85	0.80	12.2	18.0	± 0.18	± 3.2
	2	26	16.1	0.85	0.80	11.0	18.0	± 0.18	± 3.2
	1	13	13.7	0.85	0.80	9.3	18.0	± 0.18	± 3.2
Leeward	---	All	18.0	0.85	-0.50	-7.7	18.0	± 0.18	± 3.2
Side	---	All	18.0	0.85	-0.70	-10.7	18.0	± 0.18	± 3.2
Roof	---	39*	18.0	0.85	-0.90	-13.8	18.0	± 0.18	± 3.2
	---	39†	18.0	0.85	-0.50	-7.7	18.0	± 0.18	± 3.2
	---	39‡	18.0	0.85	-0.30	-4.6	18.0	± 0.18	± 3.2

* from windward edge to 39 ft

† from 39 ft to 78 ft

‡ from 78 ft to 91.33 ft

Table 4-6 Design Wind Forces in N-S Direction (V = 90 mph)

Level	Height above ground level, z (ft)	Tributary Height (ft)	Windward: External Design Wind Pressure, q_zGC_p (psf)	Windward: Design Wind Force, P* (kips)	Leeward: External Design Wind Pressure, q_hGC_p (psf)	Leeward: Design Wind Force, P* (kips)	Total Design Wind Force (kips)
3	39	6.5	12.2	12.0	-7.7	7.6	19.6
2	26	13.0	11.0	21.6	-7.7	15.2	36.8
1	13	13.0	9.3	18.3	-7.7	15.2	33.5
						Σ	89.9

*$P = qGC_p \times$ Tributary height ×151.33

4.2.3.2 Method of Analysis

Similar to the seismic analysis, a three-dimensional analysis of the building was performed for the wind forces contained in Table 4-6 using SAP2000. The modeling assumptions utilized for the seismic analysis were also used for the wind analysis.

Comparing the seismic forces in Table 4-1 to the wind forces in Table 4-6, it is clear that seismic forces will govern the design of the members.

4.2.4 Design for Combined Load Effects

4.2.4.1 Load Combinations

Basic load combinations for strength design are given in IBC 1605.2.1. As noted above, the first exception in this section requires that the non-seismic load combinations of ACI 9.2 be used for concrete structures. Thus, the following load combinations are utilized in the design of the structural members:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)
4. $1.2D + f_1 L + 1.0E$ (Formula 16-5)
5. $0.9D + 1.0E$ (Formula 16-6)

where D, L, W, and E are the effects due to dead, live, wind, and seismic loads, respectively, and f_1 is equal to either 1.0 for places of public assembly, for live loads in excess of 100 psf, and for parking garage live load, or is equal to 0.5 for other live loads.

The seismic load effect E for use in Formula 16-5, which is the combined effect of horizontal and vertical earthquake-induced forces, is computed from Eq. 16-28 where the effects of gravity and seismic ground motion are additive:

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of horizontal seismic forces
ρ = redundancy coefficient
= 1.0 for structures assigned to SDC A, B, or C (IBC 1617.2.1)

Similarly, E for use in Formula 16-6 is computed from Eq. 16-29 where the effects of gravity and seismic ground motion counteract:

$$E = \rho Q_E - 0.2 S_{DS} D$$

Substituting S_{DS} = 0.22g and ρ = 1.0 into the equations for E, and then substituting E along with f_1 = 0.5 into load combinations 4 and 5 above results in the following:

4a. $1.2D + 0.5L + 1.0Q_E + (0.2 \times 0.22)D = 1.24D + 0.5L + Q_E$

4b. $1.2D + 0.5L + 1.0Q_E - (0.2 \times 0.22)D = 1.16D + 0.5L + Q_E$

5a. $0.9D + 1.0Q_E + (0.2 \times 0.22)D = 0.94D + Q_E$

5b. $0.9D + 1.0Q_E - (0.2 \times 0.22)D = 0.86D + Q_E$

Values of Q_E are obtained from the structural analysis for both sidesway to the left and to the right based on the code-prescribed horizontal seismic forces acting on the building. Also, the special seismic load combinations in IBC 1605.4 are not applicable in this example.

4.2.4.2 Design of Beam B3-C3

Deflection control.

ACI 9.5 contains provisions for control of deflections for reinforced concrete members subjected to flexure. Deflection requirements can be satisfied for nonprestressed beams by providing the minimum thickness contained in ACI Table 9.5(a). It is important to note that the provisions of ACI 9.5.2.1 apply only to one-way construction not supporting or attached to partitions or other construction likely to be damaged by large deflections.

From Table 9.5(a), the minimum thickness of beams made of normal weight concrete and Grade 60 reinforcement with one end continuous is:

$\ell/18.5 = [(30 \times 12) - 20/2 - 16/2]/18.5 = 18.5$ in. = h = 18.5 in. O.K.

Flexural design.

Table 4-7 contains a summary of the governing design bending moments and shear forces for beam B3-C3 at the first floor level due to gravity and seismic forces. The live load on the beam is reduced in accordance with IBC 1607.9.2:

$$R = r(A - 150) \leq 40\% \text{ for horizontal members}$$

where r = rate of reduction = 0.08 for floors

A = area of floor supported by member = $30 \times 30 = 900 \text{ ft}^2 > 150 \text{ ft}^2$

Therefore,

$$R = 0.08(900 - 150) = 60\% > 40\% \quad \text{Use } 40\%$$

Reduced live load = $(1 - 0.40) \times 60 = 36$ psf

The uniformly distributed gravity loads on the beam are computed as follows:

$$w_D = [(74+45)\times 30/1{,}000] + \left(\frac{28\times 18.5}{144}\times 0.15\right) = 4.1 \text{ kips/ft}$$

$$w_L = 36 \times 30/1{,}000 = 1.1 \text{ kips/ft}$$

Table 4-7 Summary of Design Bending Moments and Shear Forces for Beam B3-C3 at Floor Level 1 (SDC B)

Load Case	Location	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	Support	-289	58
	Midspan	198	
Live (L)	Support	-53	16
	Midspan	36	
Seismic (Q_E)	Support	± 138	± 10
Load Combinations			
$1.4D + 1.7L$	Support	-495	108
	Midspan	338	
$1.24D + 0.5L + Q_E$	Support	-523	90
	Midspan	264	
$0.86D + Q_E$	Support	-111	40
	Midspan	170	

The required flexural reinforcement is contained in Table 4-8. The provided areas of steel are within the limits prescribed in ACI 10.3.3 for maximum reinforcement and ACI 10.5 for minimum reinforcement. The selected reinforcement satisfies ACI 7.6.1 and 3.3.2

(minimum spacing for concrete placement), ACI 7.7.1 (minimum cover for protection of reinforcement), and ACI 10.6 (maximum spacing for control of flexural cracking).

Table 4-8 Required Flexural Reinforcement for Beam B3-C3 at Floor Level 1 (SDC B)

Location	M_u (ft-kips)	A_s^* (in.^{2})	Reinforcement*	ϕM_n (ft-kips)
Support	-523	8.78	7-No. 10	528
Midspan	338	5.23	5-No. 10	400

$$^*A_{s,min} = \frac{3\sqrt{f'_c}\,b_w d}{f_y} = \frac{3\sqrt{4,000} \times 28 \times 16}{60,000} = 1.42 \text{ in.}^2$$

ACI 10.5

$$= \frac{200 b_w d}{f_y} = \frac{200 \times 28 \times 16}{60,000} = 1.49 \text{ in.}^2 \text{ (governs)}$$

$$A_{s,max} = \rho_{max} b_w d = 0.0214 \times 28 \times 16 = 9.59 \text{ in.}^2$$

ACI 10.3.3

IBC 1910.3.1 requires that at least 2 of the top and bottom flexural bars be continuous over the length of the beam.

Shear design.

Shear requirements for beams are contained in ACI 11.1, 11.3, and 11.5. For members subjected to shear and flexure, the nominal shear strength provided by concrete V_c is determined from Eq. 11-3:

$$V_c = 2\sqrt{f'_c}\,b_w d = 2\sqrt{4,000} \times 28 \times 16 / 1,000 = 56.7 \text{ kips}$$

The design shear strength of the concrete ϕV_c is equal to 0.85 × 56.7 = 48.2 kips. This is less than the maximum factored shear force V_u = 108 – [7.6 × (16/12)] = 98 kips at a distance d = 16 in. from the face of the support, where the total factored gravity load on the beam is equal to $w_u = 1.4w_D + 1.7w_L = (1.4 \times 4.1) + (1.7 \times 1.1) = 7.6$ kips/ft (see Table 4-7 and ACI 11.1.3.1).

Therefore, provide shear reinforcement in accordance with ACI 11.5.6.

The required spacing of No. 3 stirrups with 4 legs is determined from Eq. 11-15 (ACI 11.5.6.2):

$$s = \frac{A_v f_y d}{\frac{V_u}{\phi} - V_c} = \frac{(4 \times 0.11) \times 60 \times 16}{\frac{98}{0.85} - 56.7} = 7.2 \text{ in.}$$

$$< \begin{cases} d/2 = 16/2 = 8 \text{ in. O.K. (ACI 11.5.4.1)} \\ \frac{A_v f_y}{50 b_w} = 18.9 \text{ in. O.K. (ACI 11.5.5.3)} \end{cases}$$

Note that

$$V_s = \frac{A_v f_y d}{s} = \frac{(4 \times 0.11) \times 60 \times 16}{7} = 60.3 \text{ kips} < 4\sqrt{f_c'} b_w d = 113.3 \text{ kips}$$

so that the maximum spacing given above need not be reduced by one-half (ACI 11.5.4.3). The shear strength V_s is also less than the maximum value of $8\sqrt{f_c'} b_w d = 226.7$ kips (ACI 11.5.6.9).

According to ACI 11.5.5.1, shear reinforcement may be terminated where $V_u \leq \phi V_c / 2 =$ 24.1 kips. Length over which stirrups are required = (108 – 24.1)/7.6 = 11.0 ft.

Use 20-No. 3 stirrups (4 legs) spaced at 7 in. at each end of the beam with the first stirrup located at 2 in. from the face of the support.

Reinforcing bar cutoff points.

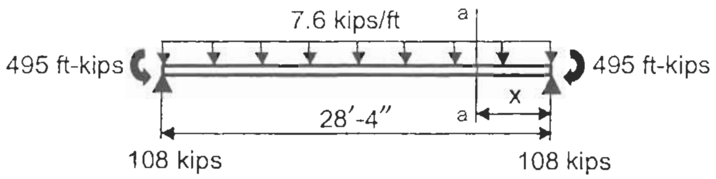
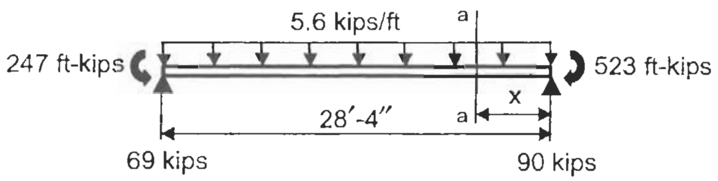
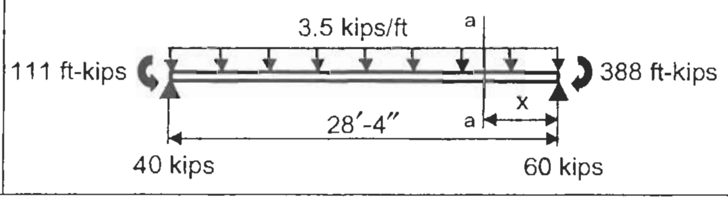
The negative reinforcement at the supports is 7-No. 10 bars. The location where 5 of the 10 bars can be terminated will be determined. The design flexural strength ϕM_n provided by 2-No. 10 bars is 174 ft-kips.

The load combinations used to determine the cutoff points are shown in Table 4-9. The distance x from the face of the support to the section where the bending moment is equal to 174 ft-kips is obtained by summing moments about section *a-a* in the figure. For example, for the second load combination:

$$\frac{5.6x^2}{2} - 90x + 523 = 174$$

Solving for x gives a distance of 4.5 ft from the face of the support. The cutoff locations for other load combinations are determined in a similar fashion. It is evident from the table that the second load combination governs.

Table 4-9 Cutoff Location of Negative Bars (SDC B)

Load Combination	Load Diagram	x (ft)
$1.4D + 1.7L$		3.4
$1.24D + 0.5L + Q_E$		4.5
$0.86D + Q_E$		4.0

The 5-No. 10 bars must extend a distance d = 16 in. (governs) or $12d_b = 12\times1.27 = 15.2$ in. beyond the distance x (ACI 12.10.3). Thus, from the face of the support, the total bar length must be at least equal to 4.5 + (16/12) = 5.8 ft.

Also, the bars must extend a full development length ℓ_d beyond the face of the support (ACI 12.10.4), which is determined from ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.3 for top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 10 bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5 + 0.375 + \dfrac{1.27}{2} = 2.5 \text{ in.} \\ \dfrac{28 - 2(1.5 + 0.375) - 1.27}{2\times6} = 1.9 \text{ in. (governs)} \end{cases}$$

K_{tr} = transverse reinforcement index

$$= \frac{A_{tr} f_{yt}}{1{,}500 sn} = \frac{4 \times 0.11 \times 60{,}000}{1{,}500 \times 7 \times 7} = 0.4$$

$$\frac{c + K_{tr}}{d_b} = \frac{1.9 + 0.4}{1.27} = 1.8 < 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.3 \times 1.0 \times 1.0 \times 1.0}{1.8} = 51.4$$

$$\ell_d = 51.4 \times 1.27 = 65.3 \text{ in.} = 5.4 \text{ ft} < 5.8 \text{ ft}$$

Thus, the total required length of the 5-No. 10 bars must be at least 5.8 ft beyond the face of the support.

Note that flexural reinforcement shall not be terminated in a tension zone unless one or more of the conditions of ACI 12.10.5 are satisfied. In this case, the point of inflection is approximately 7.6 ft from the face of the support, which is greater than 5.8 ft. Thus, the No. 10 bars cannot be terminated here unless one of the conditions of ACI 12.10.5 is satisfied. In this case, check if the factored shear force V_u at the cutoff point does not exceed $2\phi V_n / 3$ (ACI 12.10.5.1). With No. 3 stirrups at 7 in. on center that are provided in this region of the beam, ϕV_n is determined by ACI Eqs. 11-1 and 11-2:

$$\phi V_n = \phi(V_c + V_s) = 0.85 \times (56.7 + 60.3) = 99.5 \text{ kips}$$

$$\frac{2}{3} \phi V_n = 66.3 \text{ kips}$$

At 5.8 ft from the face of the support, $V_u = 90 - (5.6 \times 5.8) = 57.5$ kips, which is less than 66.3 kips. Therefore, the 5-No. 10 bars can be terminated at 5.8 ft from the face of the support.

It is assumed in this example that all positive reinforcement is continuous with splices over the columns. Note that the structural integrity provisions of ACI 7.13.2.3 require that at least one-quarter of the positive reinforcement be continuous or spliced over the support with a Class A tension splice for other than perimeter beams that do not have closed stirrups.

Figure 4-2 shows the reinforcement details for beam B3-C3.

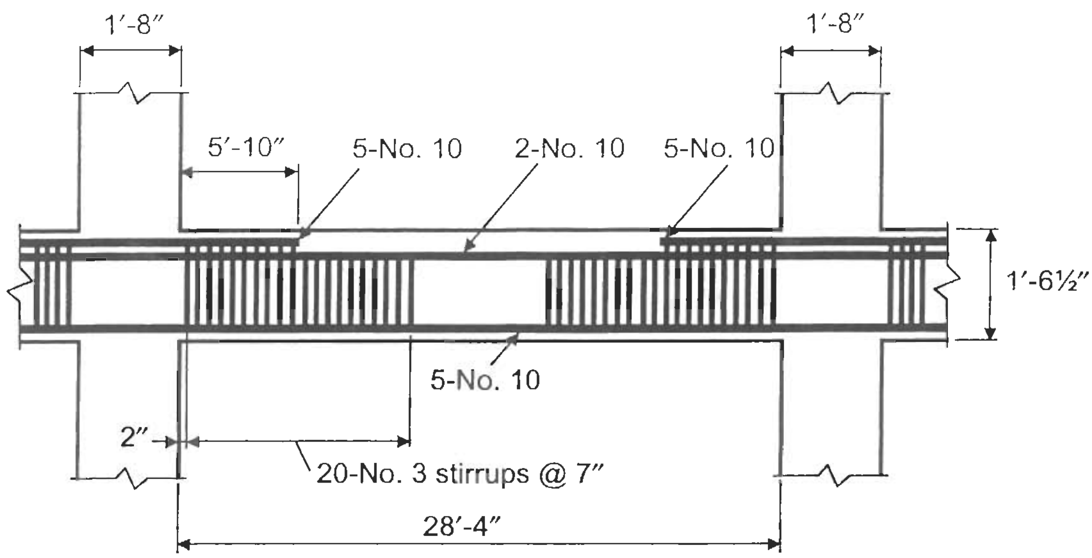


Figure 4-2 Reinforcement Details for Beam B3-C3 at Floor Level 1 (SDC B)

4.2.4.3 Design of Column C3

This section outlines the design of column C3 supporting the first floor level. Table 4-10 contains a summary of the design axial forces, bending moments, and shear forces on this column.

Table 4-10 Summary of Design Axial Forces, Bending Moments, and Shear Forces on Column C3 between Ground and 1st Floor Level (SDC B)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	355	0	0
Live (L)*	64	0	0
Seismic (Q_E)	0	± 335	± 39
Load Combinations			
$1.4D + 1.7L$	606	0	0
$1.24D + 0.5L + Q_E$	472	335	39
$0.86D + Q_E$	305	-335	-39

* Live load reduced per IBC 1607.9

Design for axial force and bending.

Based on the governing load combinations in Table 4-10, a 20 × 20 in. column with 8-No. 9 bars ($\rho_g = 2.0\%$) is adequate for column C3 supporting the first floor level.

Figure 4-3 contains the interaction diagram for this column. As noted above, slenderness effects need not be considered since P-delta effects were included in the analysis. Also, the provided reinforcement ratio is within the allowable range of 1% and 8% (ACI 10.9.1).

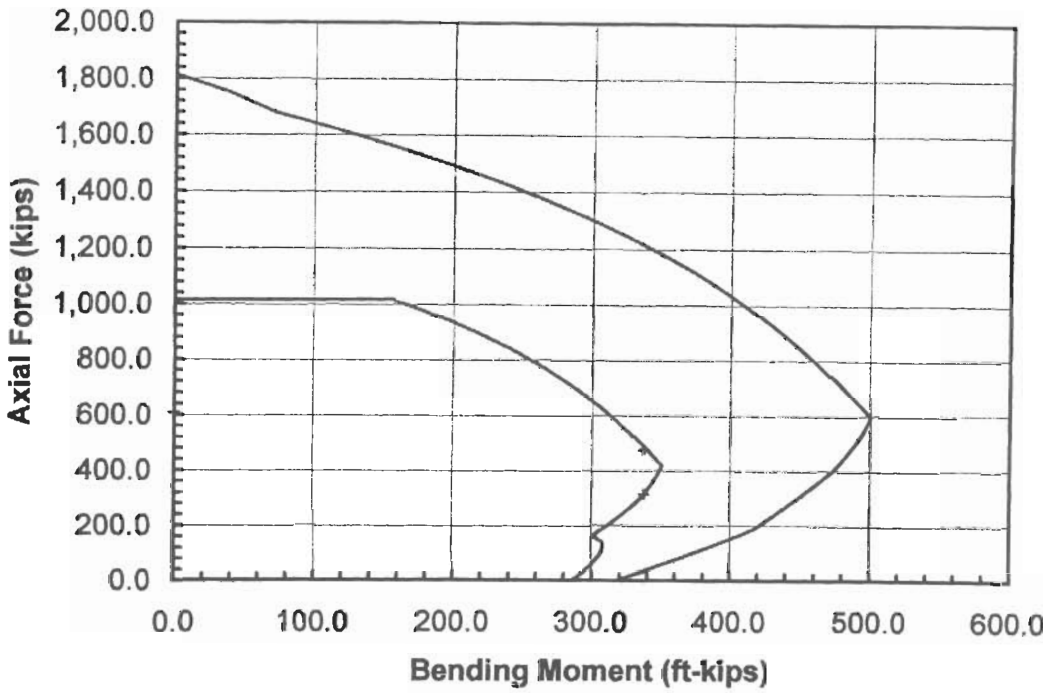


Figure 4-3 Design and Nominal Strength Interaction Diagrams for Column C3 Supporting Floor Level 1 (SDC B)

ACI 7.6.3 requires that the clear distance between longitudinal bars shall not be less than $1.5d_b = 1.5 \times 1.128 = 1.7$ in. nor 1.5 in. In this case, assuming No. 3 ties, the clear distance is equal to the following:

$$\frac{20 - 2\left(1.5 + 0.375 + \frac{1.128}{2}\right)}{2} - 1.128 = 6.4 \text{ in.} > 1.7 \text{ in. O.K.}$$

A No. 3 crosstie is required; otherwise, the clear spacing between laterally supported bars would be greater than 6 in., which is not allowed (ACI 7.10.5.3).

Design for shear.

Since the clear height to maximum plan dimension of the column = [(13 × 12) − 18.5]/20 = 6.9 > 5, design for shear need not be in accordance with ACI 21.10.3 (IBC 1910.3.1).

The shear capacity of the column is checked using ACI Eq. 11-4 for members subjected to axial compression:

$$V_c = 2\left(1 + \frac{N_u}{2{,}000A_g}\right)\sqrt{f'_c}\,b_w d$$

$$= 2\left(1 + \frac{305{,}000}{2{,}000 \times 20^2}\right)\sqrt{4{,}000} \times 20 \times 14.5/1{,}000 = 50.7 \text{ kips}$$

where N_u = 305 kips is the smallest axial force on the section (see Table 4-10) and d = 14.5 in. was obtained from a strain compatibility analysis.

Since $\phi V_c/2 = 0.85 \times 50.7/2 =$ 21.6 kips < V_u = 39 kips, provide minimum shear reinforcement in accordance with ACI 11.5.5.3. Required spacing s of No. 3 ties and crossties is determined from Eq. 11-13:

$$s = \frac{A_v f_y}{50 b_w} = \frac{(3 \times 0.11) \times 60{,}000}{50 \times 20} = 19.8 \text{ in.}$$

The transverse reinforcement requirements in ACI 7.10.5 must also be satisfied. The vertical spacing of the No. 3 ties must not exceed the least of the following:

- 16(smallest longitudinal bar diameter) = 16 × 1.128 = 18.0 in.
- 48(tie bar diameter) = 48 × 0.375 = 18.0 in.
- Least column dimension = 20 in.

Use No. 3 ties @ 18 in. with the first tie located vertically not more than 18/2 = 9 in. above the top of the slab and not more than 3 in. below the lowest horizontal reinforcement in the beams (ACI 7.10.5.4 and 7.10.5.5).

Splice length of longitudinal reinforcement.

ACI 12.17 contains special provisions for splices in columns. From the interaction diagram in Figure 4-3, it can be seen that the bar stress due to factored load combinations 2 and 3 is greater than $0.5 f_y$. Thus, Class B tension lap splices must be provided (ACI 12.17.2.3).

From ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 7 and larger bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5 + 0.375 + \dfrac{1.128}{2} = 2.4 \text{ in.} \quad \text{(governs)} \\ \\ \dfrac{20 - 2(1.5 + 0.375) - 1.128}{2 \times 2} = 3.8 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index

$$= \frac{A_{tr} f_{yt}}{1{,}500 sn} = \frac{3 \times 0.11 \times 60{,}000}{1{,}500 \times 18 \times 3} = 0.2$$

$$\frac{c + K_{tr}}{d_b} = \frac{2.4 + 0.2}{1.128} = 2.3 < 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.0 \times 1.0 \times 1.0 \times 1.0}{2.3} = 30.9$$

$$\ell_d = 30.9 \times 1.128 = 34.9 \text{ in.} = 2.9 \text{ ft}$$

Class B splice length = $1.3\ell_d$ = 3.8 ft

Use a 3 ft-10 in. splice length with the splice located just above the floor level.

Reinforcement details for column C3 are shown in Figure 4-4.

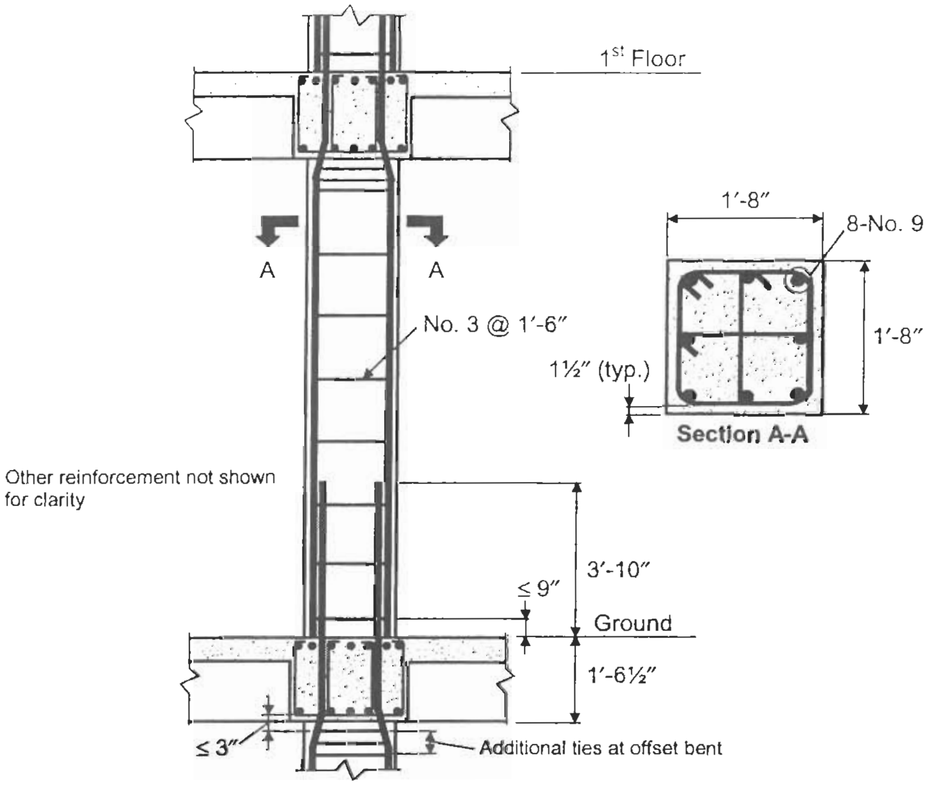


Figure 4-4 Reinforcement Details for Column C3 Supporting the 1st Floor Level (SDC B)

4.3 DESIGN FOR SDC C

To illustrate the design requirements for Seismic Design Category (SDC) C, the school building in Figure 4-1 is assumed to be located in New York, NY. Typical beams and columns are designed and detailed for combined effects of gravity, wind, and seismic forces.

4.3.1 Design Data

- Building Location: New York, NY (zip code 10013)
- Material Properties

  Concrete: $f_c' = 4{,}000$ psi, $w_c = 150$ pcf

  Reinforcement: $f_y = 60{,}000$ psi

- Service Loads

Live loads: roof = 20 psf
floor = 60 psf (average between 40 psf for floors and 80 psf for corridors above first floor level; IBC Table 1607.1). In addition, floor is to be checked for 1,000 lb concentrated live load uniformly distributed over an area of 2.5 sq ft (IBC 1607.4).

Superimposed dead loads: roof = 10 psf + 200 kips for penthouse
floor = 45 psf (20 psf permanent partitions + 25 psf ceiling, etc.)

- Seismic Design Data

For zip code 10013: $S_S = 0.424g, S_1 = 0.094g$ [4.1]

Site Class D (stiff soil profile; IBC Table 1615.1.1)

For Category (Seismic Use Group) II occupancy, $I_E = 1.25$ (IBC Table 1604.5)

- Wind Design Data

Basic wind speed = 110 mph (IBC Figure 1609)
Exposure B (IBC 1609.4)
For Category II occupancy, $I_W = 1.15$ (IBC Table 1604.5)

- Member Dimensions

Joists: 14 + 4.5 × 6 + 66 (74 psf)
Interior beams: 28 × 18.5 in.
Spandrel beams: 20 × 18.5 in.
Interior columns: 24 × 24 in.
Exterior columns: 20 × 20 in.

4.3.2 Seismic Load Analysis

4.3.2.1 Seismic Design Category (SDC)

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1-second period S_{M1} are determined from IBC Eqs. 16-16 and 16-17, respectively:

$$S_{MS} = F_a S_S = 1.46 \times 0.424 = 0.62g$$

$$S_{M1} = F_v S_1 = 2.40 \times 0.094 = 0.23g$$

where F_a and F_v are the values of site coefficients as a function of site class and of mapped spectral response acceleration at short periods S_S and at 1-second period S_1, respectively. The values of these coefficients for Site Class D in the equations above are contained in IBC Table 1615.1.2(1) for F_a and Table 1615.1.2(2) for F_v. Straight-line interpolation was used to determine F_a.

Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from Eqs. 16-18 and 16-19:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 0.62 = 0.41g$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.23 = 0.15g$$

Tables 1616.3(1) and 1616.3(2) are utilized to determine the SDC. From Table 1616.3(1) for Seismic Use Group II and S_{DS} = 0.41g, the SDC is C. Similarly, from Table 1616.3(2), the SDC is C for $S_{D1} = 0.15g$. Thus, the SDC is C for this building.

4.3.2.2 Seismic Forces

According to IBC 1616.6.2, the equivalent lateral force procedure in IBC 1617.4 may be used to compute the seismic base shear V for structures assigned to SDC C. In a given direction, V is determined from Eq. 16-34:

$$V = C_s W$$

where C_s is the seismic response coefficient determined in accordance with IBC 1617.4.1.1 and W is the effective weight of the structure defined in IBC 1617.4. For the member sizes and superimposed dead loads given above, W = 5,776 kips (see Table 4-11 below).

As noted above, a moment-resisting frame system is utilized as the seismic-force-resisting system. As a minimum, this must be an intermediate reinforced concrete moment frame to satisfy the provisions of IBC 1910.4.1 for structures assigned to SDC C. For this system, the response modification coefficient R = 5 and the deflection amplification factor $C_d = 4.5$ (IBC Table 1617.6).

Approximate period (T_a). The fundamental period of the building is determined in accordance with Eq. 16-39 in IBC 1617.4.2:

Building period coefficient $C_T = 0.03$

Building height $h_n = 39$ ft

Period $T_a = C_T (h_n)^{3/4} = 0.030 \times (39)^{3/4} = 0.47$ sec

Seismic base shear (*V*). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.15}{\left(\frac{5}{1.25}\right)\times 0.47} = 0.080$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{0.41}{\left(\frac{5}{1.25}\right)} = 0.103$$

Also, C_s must not be less than the following value from Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 0.41 \times 1.25 = 0.023$$

Thus, Eq. 16-36 governs, and the base shear *V* is:

$$V = C_s W = 0.080 \times 5{,}776 = 462 \text{ kips}$$

Vertical distribution of seismic forces. The lateral forces F_x and the story shears V_x are contained in Table 4-11. For *T* = 0.47 sec, *k* = 1.0.

Table 4-11 Seismic Forces and Story Shears (SDC C)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
3	1,684	39	65,676	209	209
2	2,046	26	53,196	169	378
1	2,046	13	26,598	84	462
Σ	5,776		145,470	462	

4.3.2.3 Method of Analysis

A three-dimensional analysis of the building was performed in the N-S and E-W directions for the seismic forces contained in Table 4-11 using SAP2000 [4.2]. In the model, rigid diaphragms were assigned at each floor level, and rigid-end offsets were defined at the ends of the horizontal members so that results were automatically obtained at the faces of the supports. The stiffness properties of the members were input assuming cracked sections. In lieu of a more accurate analysis, the following cracked section properties were used:

- Beams: $I_{eff} = 0.5 I_g$
- Columns: $I_{eff} = 0.7 I_g$

where I_g is the gross moment of inertia of the section. P-delta effects were also considered in the analysis.

In accordance with IBC 1617.4.4.4, the center of mass was displaced each way from its actual location a distance equal to 5 percent of the building dimension perpendicular to the applied forces to account for accidental torsion in seismic design.

4.3.2.4 Story Drift and P-delta Effects

Story drift determination. Table 4-12 contains the displacements δ_{xe} obtained from the 3-D static, elastic analyses using the design seismic forces in the N-S direction, including accidental torsional effects. The table also contains the design earthquake displacement δ_x computed by IBC Eq. 16-46:

$$\delta_x = \frac{C_d \delta_{xe}}{I_E}$$

where C_d is the deflection amplification factor in Table 1617.6, which depends on the seismic-force-resisting system. As noted above, C_d is equal to 4.5 for an intermediate reinforced concrete moment frame.

Table 4-12 Lateral Displacements and Interstory Drifts due to Seismic Forces in N-S Direction (SDC C)

Story	δ_{xe} (in.)	δ_x (in.)	Δ (in.)
3	1.12	4.03	1.19
2	0.79	2.84	1.69
1	0.32	1.15	1.15

The interstory drifts Δ computed from the δ_x are also contained in the table. For this structure that does not have plan irregularity Type 1a or 1b of Table 1616.5.1, the drift at story level x is determined by subtracting the design earthquake displacement at the center of mass at the bottom of the story from the design earthquake displacement at the center of mass at the top of the story (IBC 1617.4.6.1):

$$\Delta = \delta_x - \delta_{x-1}$$

The design story drifts Δ must not exceed the allowable story drift Δ_a from Table 1617.3 (IBC 1617.3). For Seismic Use Group II, $\Delta_a = 0.020 h_{sx}$ for buildings 4 stories or less that have been designed to accommodate the story drifts. Thus, for the 13-ft story heights, $\Delta_a = 0.020 \times 13 \times 12 = 3.12$ in. It is evident from Table 4-12 that for all stories, the lateral drifts obtained from the prescribed lateral forces in the N-S direction are less than the limiting value.

P-delta effects. As noted above, P-delta effects were automatically considered in the analysis using SAP2000. The provisions of IBC 1617.4.6.2 are illustrated in Section 4.4.2.4 of this publication for SDC D.

4.3.3 Wind Load Analysis

4.3.3.1 Wind Forces

Wind forces are determined in accordance with the analytical procedure (Method 2) given in ASCE 6.5 [4.3].

Details on how to compute the wind forces are given in Section 4.2.3 of this publication. In this example, the wind velocity is 110 mph. A summary of the design wind forces in the N-S direction at all floor levels is contained in Table 4-13. Once again it is important to note that the wind directionality factor K_d has been taken equal to 1.0 (see Exception 1 in IBC 1605.2.1).

Table 4-13 Design Wind Forces in N-S Direction (V = 110 mph)

Level	Height above ground level, *z* (ft)	Total Design Wind Force (kips)
3	39	29.4
2	26	55.1
1	13	50.2
	Σ	134.7

4.3.3.2 Method of Analysis

Similar to the seismic analysis, a three-dimensional analysis of the building was performed for the wind forces contained in Table 4-13 using SAP2000. The modeling assumptions utilized for the seismic analysis were also used for the wind analysis.

Comparing the seismic forces in Table 4-11 to the wind forces in Table 4-13, it is clear that seismic forces will govern the design of the members.

4.3.4 Design for Combined Load Effects

4.3.4.1 Load Combinations

Basic load combinations for strength design are given in IBC 1605.2.1. As noted above, the first exception in this section requires that the non-seismic load combinations of ACI 9.2 be used for concrete structures. Thus, the following load combinations are utilized in the design of the structural members:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)
4. $1.2D + f_1 L + 1.0E$ (Formula 16-5)
5. $0.9D + 1.0E$ (Formula 16-6)

where D, L, W, and E are the effects due to dead, live, wind, and seismic loads, respectively, and f_1 is equal to either 1.0 for places of public assembly, for live loads in excess of 100 psf, and for parking garage live load, or is equal to 0.5 for other live loads.

The seismic load effect E for use in Formula 16-5, which is the combined effect of horizontal and vertical earthquake-induced forces, is computed from Eq. 16-28 where the effects of gravity and seismic ground motion are additive:

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of horizontal seismic forces
ρ = redundancy coefficient
= 1.0 for structures assigned to SDC A, B, or C (IBC 1617.2.1)

Similarly, E for use in Formula 16-6 is computed from Eq. 16-29 where the effects of gravity and seismic ground motion counteract:

$$E = \rho Q_E - 0.2 S_{DS} D$$

Substituting $S_{DS} = 0.41g$ and $\rho = 1.0$ into the equations for E, and then substituting E along with $f_1 = 0.5$ into load combinations 4 and 5 above results in the following:

4a. $1.2D + 0.5L + 1.0Q_E + (0.2 \times 0.41)D = 1.28D + 0.5L + Q_E$

4b. $1.2D + 0.5L + 1.0Q_E - (0.2 \times 0.41)D = 1.12D + 0.5L + Q_E$

5a. $0.9D + 1.0Q_E + (0.2 \times 0.41)D = 0.98D + Q_E$

5b. $0.9D + 1.0Q_E - (0.2 \times 0.41)D = 0.82D + Q_E$

Values of Q_E are obtained from the structural analysis for both sidesway to the left and to the right based on the code-prescribed horizontal seismic forces acting on the building. Also, the special seismic load combinations in IBC 1605.4 are not applicable in this example.

4.3.4.2 Design of Beam B3-C3

Deflection control.

As was shown in Section 4.2.4.2 of this publication, the 18.5-in. depth is adequate for deflection control (see ACI Table 9.5(a)).

Flexural design.

Table 4-14 contains a summary of the governing design bending moments and shear forces for beam B3-C3 at the first floor level due to gravity and seismic forces. The live load on the beam has been reduced in accordance with IBC 1607.9.2.

Table 4-14 Summary of Design Bending Moments and Shear Forces for Beam B3-C3 at Floor Level 1 (SDC C)

Load Case	Location	Bending Moment (ft-kips)	Shear Force (kips)
Dead (*D*)	Support	-289	57
	Midspan	198	
Live (*L*)	Support	-53	15
	Midspan	36	
Seismic (Q_E)	Support	± 113	± 8
Load Combinations			
$1.4D + 1.7L$	Support	-495	106
	Midspan	338	
$1.28D + 0.5L + Q_E$	Support	-509	89
	Midspan	271	
$0.82D + Q_E$	Support	-124	39
	Midspan	162	

Requirements for intermediate moment frames are given in ACI 21.10. The factored axial force on the member, which is negligible, is less than $A_g f_c' / 10$; thus, the provisions of ACI 21.10.4 for beams must be satisfied. All other applicable provisions in Chapters 1 through 18 of ACI 318-99 are to be satisfied as well.

The required flexural reinforcement is contained in Table 4-15. The provided areas of steel are within the limits prescribed in ACI 10.3.3 for maximum reinforcement and ACI 10.5 for minimum reinforcement. The selected reinforcement satisfies ACI 7.6.1 and 3.3.2 (minimum spacing for concrete placement), ACI 7.7.1 (minimum cover for protection of reinforcement), and ACI 10.6 (maximum spacing for control of flexural cracking).

Table 4-15 Required Flexural Reinforcement for Beam B3-C3 at Floor Level 1 (SDC C)

Location	M_u (ft-kips)	A_s^* (in.^{2})	Reinforcement*	ϕM_n (ft-kips)
Support	-509	8.49	7-No. 10	528
Midspan	338	5.23	5-No. 10	400

$$^*A_{s,min} = \frac{3\sqrt{f'_c}\,b_w d}{f_y} = \frac{3\sqrt{4{,}000}\times 28\times 16}{60{,}000} = 1.42 \text{ in.}^2$$

ACI 10.5

$$= \frac{200 b_w d}{f_y} = \frac{200\times 28\times 16}{60{,}000} = 1.49 \text{ in.}^2 \text{ (governs)}$$

$$A_{s,max} = \rho_{max} b_w d = 0.0214\times 28\times 16 = 9.59 \text{ in.}^2$$

ACI 10.3.3

ACI 21.10.4.1 requires that the positive moment strength at the face of the joint be greater than or equal to 33% of the negative moment strength at that location. This is satisfied, since 400 ft-kips > 528/3 = 176 ft-kips. Also, the negative or positive moment strength at any section along the length of the member must be greater than or equal to 20% of the maximum moment strength provided at the face of either joint. In this case, 20% of the maximum moment strength is equal to 528/5 = 106 ft-kips. Providing 2-No. 10 bars (ϕM_n = 174 ft-kips) satisfies this provision.

Shear design.

Shear requirements for beams, columns, and two-way slabs in intermediate moment frames are contained in ACI 21.10.3. Design shear strength shall not be less than either (a) the sum of the shear associated with development of nominal moment strength at each end of the clear span and shear due to factored gravity loads or (b) the maximum shear obtained from load combinations that include *E*, where *E* is taken to be twice that prescribed by the governing code. In this example, the first of the two options is utilized.

The largest shear force associated with seismic effects is obtained from the second of the three load combinations in Table 4-14. Figure 4-5 shows the beam and shear forces due to gravity loads plus nominal moment strengths for sidesway to the right. Due to the symmetric distribution of longitudinal reinforcement in the beam, sidesway to the left gives the same maximum shear force. The factored uniform load on the beam is determined as follows:

$$w_D = [(74+45)\times 30/1{,}000] + \left(\frac{28\times 18.5}{144}\times 0.15\right) = 4.1 \text{ kips/ft}$$

$$w_L = 36\times 30/1{,}000 = 1.1 \text{ kips/ft}$$

$w_u = 1.28 w_D + 0.5 w_L = 5.8$ kips/ft

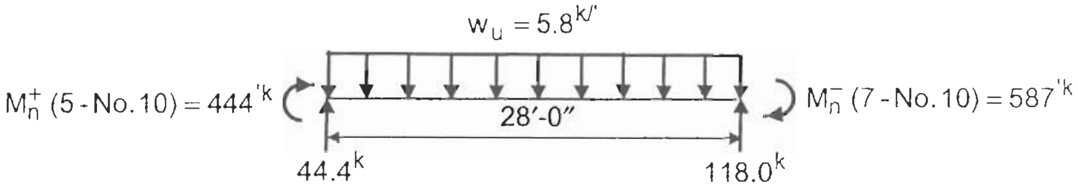


Figure 4-5 Design Shear Forces for Beam B3-C3 (SDC C)

The maximum combined shear force as shown in Figure 4-5 (118.0 kips) is larger than the maximum shear force obtained from the structural analysis (89 kips; see Table 4-14).

The nominal shear strength provided by concrete V_c is computed from ACI Eq. 11-3:

$$V_c = 2\sqrt{f_c'}b_w d = 2\sqrt{4{,}000} \times 28 \times 16 / 1{,}000 = 56.7 \text{ kips}$$

Since V_u = 118.0 kips is greater than $\phi V_c = 0.85 \times 56.7 = 48.2$ **kips**, provide shear reinforcement in accordance with ACI 11.5.6. Assuming No. 3 stirrups with 4 legs, the required spacing s is determined from Eq. (11-15):

$$s = \frac{A_v f_y d}{\frac{V_u}{\phi} - V_c} = \frac{(4 \times 0.11) \times 60 \times 16}{\frac{118.0}{0.85} - 56.7} = 5.1 \text{ in.}$$

According to ACI 21.10.4.2, the maximum spacing of stirrups over the length $2h$ = $2 \times 18.5 = 37$ in. from the face of the support at each end of the member is the smaller of the following:

- $d/4 = 16/4 = 4.0$ in. (governs)
- 8(diameter of smallest longitudinal bar) = 8 × 1.27 = 10.2 in.
- 24(diameter of stirrup bar) = 24 × 0.375 = 9.0 in.
- 12 in.

Use 10-No. 3 stirrups (4 legs) at each end of the beam spaced at 4 in. on center with the first stirrup located 2 in. from the face of the support. For the remainder of the beam, the maximum stirrup spacing is $d/2$ = 8 in. (ACI 21.10.4.3). Use No. 3 stirrups @ 8 in. for the remainder of the beam.

Reinforcing bar cutoff points.

The negative reinforcement at the supports is 7-No. 10 bars. The location where 5 of the 10 bars can be terminated will be determined.

The second load combination is used to determine the cutoff point of the 5-No. 10 bars, since this combination produces the longest bar lengths. The design flexural strength ϕM_n provided by 2-No. 10 bars is 174 ft-kips. Therefore, the 5-No. 10 bars can be terminated after the required moment strength M_u has been reduced to 174 ft-kips.

The distance x from the support to the location where the moment is equal to 174 ft-kips can readily be determined by summing moments about section a-a in Figure 4-6:

$$\frac{5.8x^2}{2} - 89x + 509 = 174$$

Solving for x gives a distance of 4.4 ft from the face of the support.

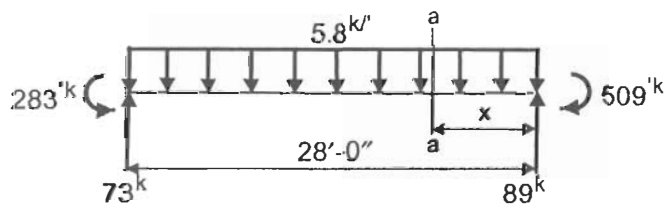


Figure 4-6 Cutoff Location of Negative Bars (SDC C)

The 5-No. 10 bars must extend a distance d = 16 in. (governs) or $12d_b = 12 \times 1.27 = 15.2$ in. beyond the distance x (ACI 12.10.3). Thus, from the face of the support, the total bar length must be at least equal to 4.4 + (16/12) = 5.7 ft.

Also, the bars must extend a full development length ℓ_d beyond the face of the support (ACI 12.10.4), which is determined from ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.3 for top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 10 bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5 + 0.375 + \dfrac{1.27}{2} = 2.5 \text{ in.} \\ \dfrac{28 - 2(1.5 + 0.375) - 1.27}{2 \times 6} = 1.9 \text{ in. (governs)} \end{cases}$$

K_{tr} = transverse reinforcement index

$$= \frac{A_{tr} f_{yt}}{1{,}500 sn} = \frac{4 \times 0.11 \times 60{,}000}{1{,}500 \times 8 \times 7} = 0.3$$

$$\frac{c + K_{tr}}{d_b} = \frac{1.9 + 0.3}{1.27} = 1.7 < 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.3 \times 1.0 \times 1.0 \times 1.0}{1.7} = 54.4$$

$$\ell_d = 54.4 \times 1.27 = 69.1 \text{ in.} = 5.8 \text{ ft}$$

Thus, the total required length of the 5-No. 10 bars must be at least 5.8 ft beyond the face of the support.

Note that flexural reinforcement shall not be terminated in a tension zone unless one or more of the conditions of ACI 12.10.5 are satisfied. In this case, the point of inflection is approximately 7.6 ft from the face of the support, which is greater than 5.8 ft. Thus, the No. 10 bars cannot be terminated here unless one of the conditions of ACI 12.10.5 is satisfied. In this case, check if the factored shear force V_u at the cutoff point does not exceed $2\phi V_n / 3$ (ACI 12.10.5.1). With No. 3 stirrups at 8 in. on center that are provided in this region of the beam, ϕV_n is determined by ACI Eqs. 11-1 and 11-2:

$$\phi V_n = \phi(V_c + V_s) = 0.85 \times (56.7 + 52.8) = 93.1 \text{ kips}$$

$$\frac{2}{3} \phi V_n = 62.1 \text{ kips}$$

At 5.8 ft from the face of the support, $V_u = 88 - (5.8 \times 5.8) = 54.4$ kips, which is less than 62.1 kips. Therefore, the 5-No. 10 bars can be terminated at 5.8 ft from the face of the support.

It is assumed in this example that all positive reinforcement is continuous with splices over the columns. Note that the structural integrity provisions of ACI 7.13.2.3 require that at least one-quarter of the positive reinforcement be continuous or spliced over the support with a Class A tension splice for other than perimeter beams that do not have closed stirrups.

Figure 4-7 shows the reinforcement details for beam B3-C3.

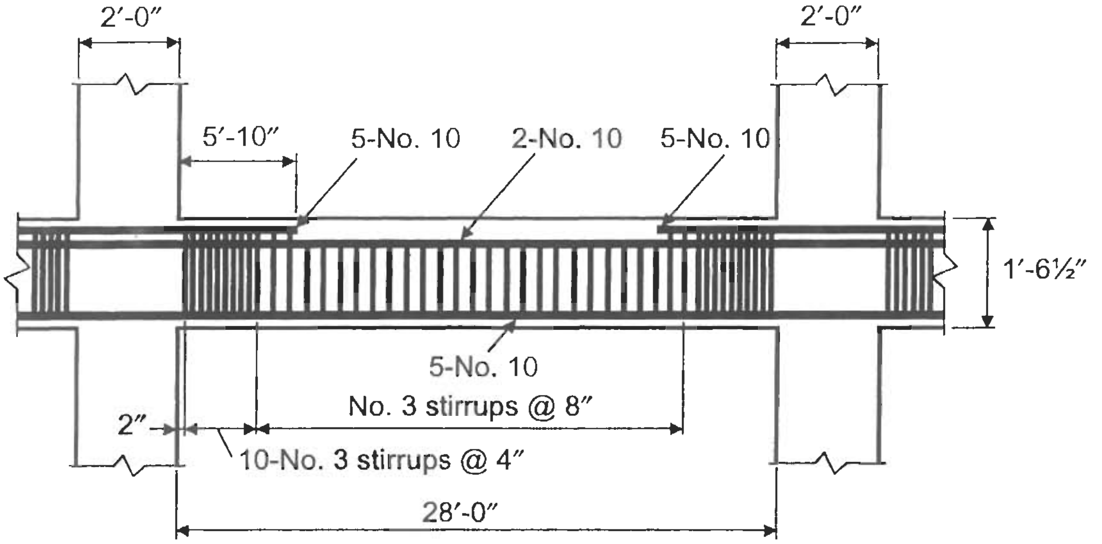


Figure 4-7 Reinforcement Details for Beam B3-C3 at Floor Level 1 (SDC C)

4.3.4.3 Design of Column C3

This section outlines the design of column C3 supporting the first floor level. Table 4-16 contains a summary of the design axial forces, bending moments, and shear forces on this column for gravity and seismic loads. Since the factored compressive axial loads exceed $A_g f'_c / 10 = 24^2 \times 4/10 =$ 230 kips, the provisions of ACI 21.10.5 must be satisfied, unless spiral reinforcement conforming to Eq. 10-6 is used. In this example, transverse reinforcement consists of ties and crossties.

Design for axial force and bending.

Based on the governing load combinations in Table 4-16, a 24 × 24 in. column with 8-No. 9 bars ($\rho_g = 1.39\%$) is adequate for column C3 supporting the first floor level. Figure 4-8 contains the interaction diagram for this column. As noted above, slenderness effects need not be considered since P-delta effects were included in the analysis. Also, the provided reinforcement ratio is within the allowable range of 1% and 8% (ACI 10.9.1).

Table 4-16 Summary of Design Axial Forces, Bending Moments, and Shear Forces on Column C3 between Ground and 1st Floor Level (SDC C)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	362	0	0
Live (L)*	64	0	0
Seismic (Q_E)	0	$\pm$ 264	$\pm$ 28
Load Combinations			
$1.4D + 1.7L$	616	0	0
$1.28D + 0.5L + Q_E$	495	264	28
$0.82D + Q_E$	297	-264	-28

* Live load reduced per IBC 1607.9

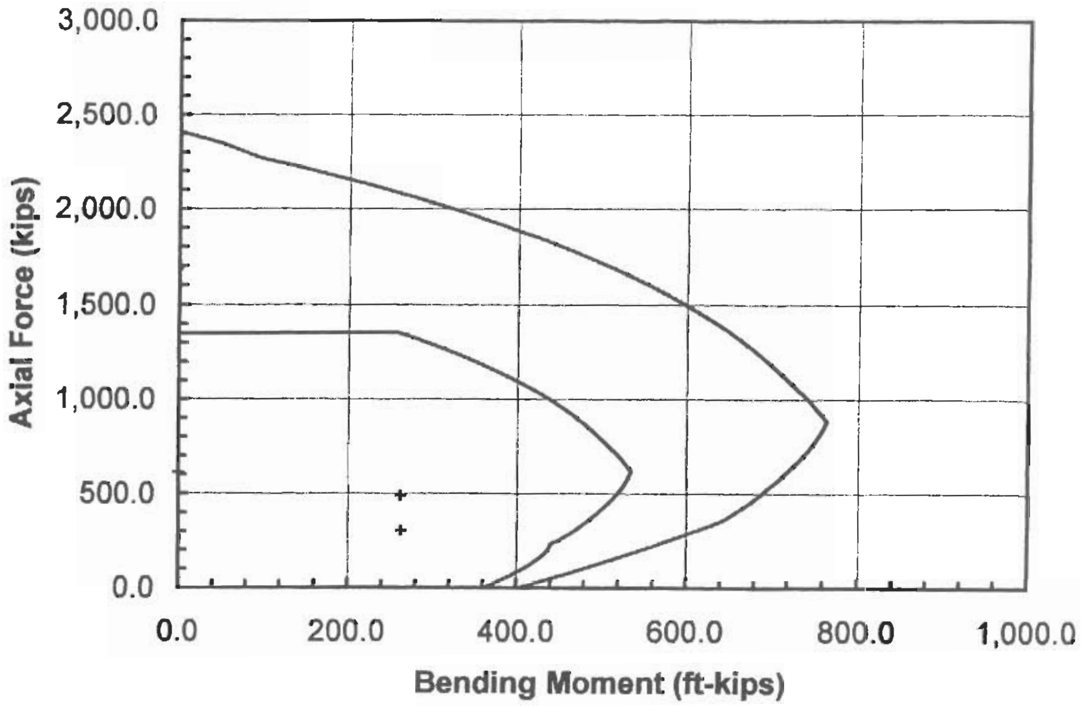


Figure 4-8 Design and Nominal Strength Interaction Diagrams for Column C3 Supporting Floor Level 1 (SDC C)

ACI 7.6.3 requires that the clear distance between longitudinal bars shall not be less than $1.5d_b = 1.5 \times 1.128 = 1.7$ in. nor 1.5 in. In this case, assuming No. 3 ties, the clear distance is equal to the following:

$$\frac{24-2\left(1.5+0.375+\frac{1.128}{2}\right)}{2}-1.128=8.4\text{ in.}>1.7\text{ in. O.K.}$$

A No. 3 crosstie is required; otherwise, the clear spacing between laterally supported bars would be greater than 6 in., which is not allowed (ACI 7.10.5.3).

Design for shear.

Similar to beams, columns in intermediate moment frames must satisfy the shear requirements in ACI 21.10.3. The second of the two options in that section is utilized here to determine the design shear strength.

From Table 4-16, the maximum shear force V_u obtained from design load combinations that include E is 28 kips. According to ACI 21.10.3(b), the design shear force must be two times this value, which is 56 kips.

The shear capacity of the column will be checked using ACI Eq. 11-4 for members subjected to axial compression:

$$V_c = 2\left(1+\frac{N_u}{2{,}000A_g}\right)\sqrt{f'_c}b_w d$$

$$= 2\left(1+\frac{297{,}000}{2{,}000\times 24^2}\right)\sqrt{4{,}000}\times 24\times 17.7/1{,}000 = 67.6\text{ kips}$$

where N_u = 297 kips is the smallest axial force on the section (see Table 4-16) and d = 17.7 in. was obtained from a strain compatibility analysis.

Since $\phi V_c/2 = 0.85\times 67.6/2 = 28.7$ kips < V_u = 56 kips, minimum shear reinforcement would generally have to satisfy ACI 11.5.5.3. However, for intermediate moment frames, the requirements in ACI 21.10.5.1 take precedence. These requirements are intended primarily to confine the concrete within the core and provide lateral support for the longitudinal reinforcement. For No. 3 ties, the vertical spacing s_o must not exceed the least of the following over a length ℓ_o measured from the joint face:

- 8(smallest longitudinal bar diameter) = 8 × 1.128 = 9.0 in. (governs)
- 24(tie bar diameter) = 24 × 0.375 = 9.0 in. (governs)
- Least column dimension/2 = 24/2 = 12 in.
- 12 in.

where ℓ_o is the largest of the following:

- Clear span/6 = [(13 × 12) − 18.5]/6 = 22.9 in.
- Maximum cross-sectional dimension of member = 24 in. (governs)
- 18 in.

Use 5-No. 3 ties and crossties @ 9 in. with the first tie located at 4 in. (< $s_o/2 = 4.5$ in; ACI 21.10.5.2) from the joint face above the ground. The 9 in. spacing is used over the lap splice length of 42 in. (> ℓ_o = 24 in.), which is determined below. Below the first floor level, 4-No. 3 ties and crossties are used. For the remainder of the column, tie spacing shall not exceed $2s_o$ = 18 in. (ACI 21.10.5.4).

ACI 21.10.5.3 requires that joint reinforcement in intermediate moment frames conform to ACI 11.11.2. Since this beam-column joint is part of the primary seismic load-resisting system, lateral reinforcement in the joint must not be less than that computed by Eq. (11-13). For No. 3 ties with one crosstie, the required spacing is:

$$s = \frac{A_v f_y}{50 b_w} = \frac{(3 \times 0.11) \times 60{,}000}{50 \times 24} = 16.5 \text{ in.}$$

For detailing simplicity, continue the 9 in. spacing at the column ends through the joint.

Splice length of longitudinal reinforcement.

ACI 12.17 contains special provisions for splices in columns. From the interaction diagram in Figure 4-8, it can be seen that the bar stress due to factored load combinations 2 and 3 is greater than $0.5 f_y$. Thus, Class B tension lap splices must be provided (ACI 12.17.2.3).

From ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 7 and larger bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5 + 0.375 + \dfrac{1.128}{2} = 2.4 \text{ in.} \quad \text{(governs)} \\ \\ \dfrac{24 - 2(1.5 + 0.375) - 1.128}{2 \times 2} = 4.8 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index

$$= \frac{A_{tr} f_{yt}}{1{,}500sn} = \frac{3 \times 0.11 \times 60{,}000}{1{,}500 \times 9 \times 3} = 0.5$$

$$\frac{c + K_{tr}}{d_b} = \frac{2.4 + 0.5}{1.128} = 2.6 > 2.5, \text{ use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.0 \times 1.0 \times 1.0 \times 1.0}{2.5} = 28.5$$

$$\ell_d = 28.5 \times 1.128 = 32.2 \text{ in.} = 2.7 \text{ ft}$$

$$\text{Class B splice length} = 1.3\ell_d = 3.5 \text{ ft}$$

Use a 3 ft-6 in. splice length with the splice located just above the floor level. Note that in Section 3.4.4.3 of this publication, the lap splices for the column in SDC C were located within the center half of the member length, which is outside of potential plastic hinge regions. Although it may be prudent to locate the lap splices at this location, it is not required. Reinforcement details for column C3 with the lap splices located just above the floor level are shown in Figure 4-9.

4.4 DESIGN FOR SDC D -- SOUTHEASTERN U.S.

To illustrate the design requirements for Seismic Design Category (SDC) D, the school building in Figure 4-1 is assumed to be located in Atlanta, GA, on a site with a soft soil profile. Typical beams and columns are designed and detailed for combined effects of gravity, wind, and seismic forces.

4.4.1 Design Data

- Building Location: Atlanta, GA (zip code 30350)

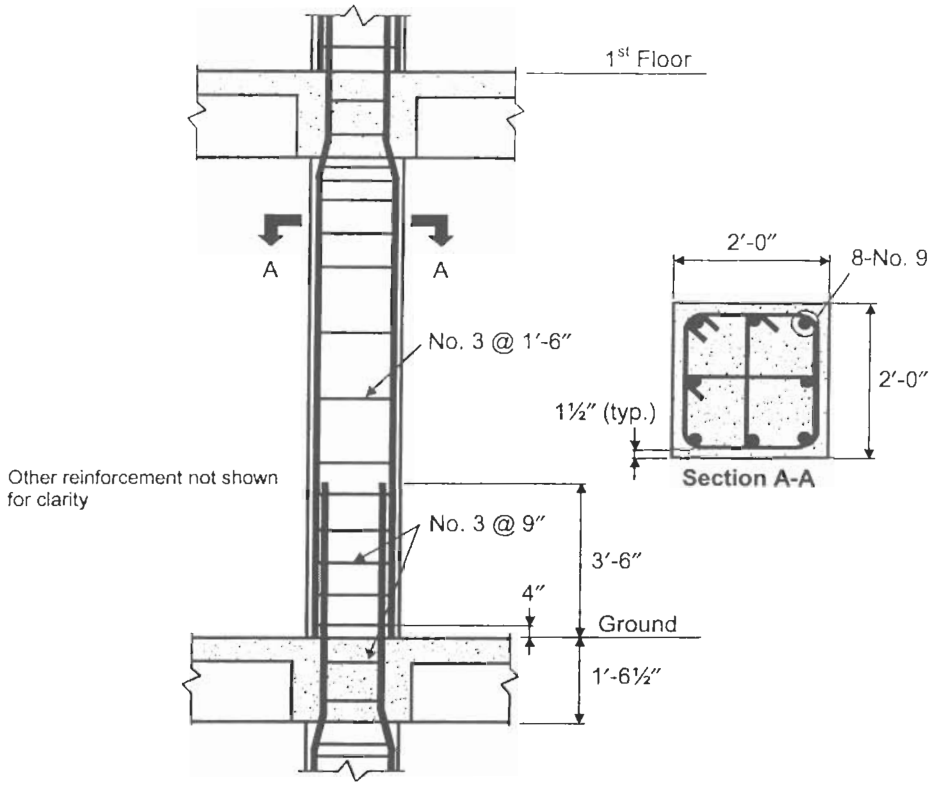


Figure 4-9 Reinforcement Details for Column C3 Supporting the 1st Floor Level (SDC C)

- Material Properties

Concrete: $f_c' = 4{,}000$ psi, $w_c = 150$ pcf
Reinforcement: $f_y = 60{,}000$ psi

- Service Loads

Live loads: roof = 20 psf
floor = 60 psf (average between 40 psf for floors and 80 psf for corridors above first floor level; IBC Table 1607.1). In addition, floor is to be checked for 1,000 lb concentrated live load uniformly distributed over an area of 2.5 sq ft (IBC 1607.4).

Superimposed dead loads: roof = 10 psf + 200 kips for penthouse
floor = 45 psf (20 psf permanent partitions + 25 psf ceiling, etc.)

- Seismic Design Data

  For zip code 30350: $S_S = 0.276g$, $S_1 = 0.117g$ [4.1]

  Site Class E (soft soil profile; IBC Table 1615.1.1)

  For Category (Seismic Use Group) II occupancy, $I_E = 1.25$ (IBC Table 1604.5)

- Wind Design Data

  Basic wind speed = 90 mph (IBC Figure 1609)
  Exposure B (IBC 1609.4)
  For Category II occupancy, $I_W = 1.15$ (IBC Table 1604.5)

- Member Dimensions

  Joists: 14 + 4.5 × 6 + 66 (74 psf)
  Interior beams: 28 × 18.5 in.
  Spandrel beams: 24 × 18.5 in.
  Interior columns: 34 × 34 in.
  Exterior columns: 30 × 30 in.

4.4.2 Seismic Load Analysis

4.4.2.1 Seismic Design Category (SDC)

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1-second period S_{M1} are determined from IBC Eqs. 16-16 and 16-17, respectively:

$$S_{MS} = F_a S_S = 2.42 \times 0.276 = 0.67g$$

$$S_{M1} = F_v S_1 = 3.45 \times 0.117 = 0.40g$$

where F_a and F_v are the values of site coefficients as a function of site class and of mapped spectral response acceleration at short periods S_S and at 1-second period S_1, respectively. The values of these coefficients for Site Class E in the equations above are contained in IBC Table 1615.1.2(1) for F_a and Table 1615.1.2(2) for F_v. Straight-line interpolation was used to determine F_a and F_v.

Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from Eqs. 16-18 and 16-19:

$$S_{DS} = \frac{2}{3}S_{MS} = \frac{2}{3}\times 0.67 = 0.44g$$

$$S_{D1} = \frac{2}{3}S_{M1} = \frac{2}{3}\times 0.40 = 0.27g$$

Tables 1616.3(1) and 1616.3(2) are utilized to determine the SDC. From Table 1616.3(1) for Seismic Use Group II and S_{DS} = 0.44g, the SDC is C. Similarly, from Table 1616.3(2), the SDC is D for S_{D1} = 0.27g. The most severe SDC is assigned to the structure in accordance with IBC 1616.3. Thus, the SDC is D for this building.

4.4.2.2 Seismic Forces

Since the building does not have plan irregularity Type 1a, 1b, or 4 of Table 1616.5.1 or vertical irregularity Type 1a, 1b, 4, or 5 of Table 1616.5.2, it can be considered regular (IBC 1616.6.3). For this regular building that is less than 240 ft in height, Table 1616.6.3 allows the equivalent lateral force procedure in IBC 1617.4 to be used to compute the seismic base shear V (see Eq. 16-34):

$$V = C_s W$$

where C_s is the seismic response coefficient determined in accordance with IBC 1617.4.1.1 and W is the effective weight of the structure defined in IBC 1617.4. For the member sizes and superimposed dead loads given above, W = 6,293 kips (see Table 4-17 below).

As noted above, a moment-resisting frame system is utilized as the seismic-force-resisting system. To satisfy the provisions of IBC 1910.5.1 for structures assigned to SDC D, a special reinforced concrete moment frame must be used. For this system, the response modification coefficient R = 8 and the deflection amplification factor C_d = 5.5, which are found in IBC Table 1617.6.

Approximate period (T_a). The fundamental period of the building is determined in accordance with Eq. 16-39 in IBC 1617.4.2:

Building period coefficient $C_T = 0.03$

Building height $h_n = 39$ ft

Period $T_a = C_T(h_n)^{3/4} = 0.030\times(39)^{3/4} = 0.47$ sec

Seismic base shear (V). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.27}{\left(\frac{8}{1.25}\right)\times 0.47} = 0.090$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{0.44}{\left(\frac{8}{1.25}\right)} = 0.069$$

Also, C_s must not be less than the following value from Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 0.44 \times 1.25 = 0.024$$

Thus, Eq. 16-35 governs, and the base shear V is:

$$V = C_s W = 0.069 \times 6{,}293 = 434 \text{ kips}$$

Vertical distribution of seismic forces. The lateral forces F_x and the story shears V_x are contained in Table 4-17. For $T = 0.47$ sec, $k = 1.0$.

Table 4-17 Seismic Forces and Story Shears (SDC D)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
3	1,795	39	70,005	193	193
2	2,249	26	58,474	161	354
1	2,249	13	29,237	80	434
Σ	6,293		157,716	434	

4.4.2.3 Method of Analysis

A three-dimensional analysis of the building was performed in the N-S and E-W directions for the seismic forces contained in Table 4-17 using SAP2000 [4.2]. In the model, rigid diaphragms were assigned at each floor level, and rigid-end offsets were defined at the ends of the horizontal members so that results were automatically obtained at the faces of the supports. The stiffness properties of the members were input assuming cracked sections. In lieu of a more accurate analysis, the following cracked section properties were used:

- Beams: $I_{eff} = 0.5 I_g$

- Columns: $I_{eff} = 0.7I_g$

where I_g is the gross moment of inertia of the section. P-delta effects were also considered in the analysis.

In accordance with IBC 1617.4.4.4, the center of mass was displaced each way from its actual location a distance equal to 5 percent of the building dimension perpendicular to the applied forces to account for accidental torsion in seismic design.

For all structures assigned to SDC D and higher, IBC 1620.3.5 requires that orthogonal effects of the seismic forces be considered for design and detailing of the components of the seismic-force-resisting system. The orthogonal combination procedure in IBC 1620.2.2 is permitted to be used to satisfy the requirements of IBC 1620.3.5. In the 2002 supplement to the 2000 IBC [4.4], the orthogonal combination procedure is required only for columns or walls that form part of two or more intersecting seismic-force-resisting systems and are subjected to axial loads due to seismic forces greater than or equal to 20% of the axial load design strength.

4.4.2.4 Story Drift and P-delta Effects

Story drift determination. Table 4-18 contains the displacements δ_{xe} obtained from the 3-D static, elastic analyses using the design seismic forces in the N-S direction, including accidental torsional effects. The table also contains the design earthquake displacement δ_x computed by IBC Eq. 16-46:

$$\delta_x = \frac{C_d \delta_{xe}}{I_E}$$

where C_d is the deflection amplification factor in Table 1617.6, which depends on the seismic-force-resisting system. As noted above, C_d is equal to 5.5 for a special reinforced concrete moment frame.

Table 4-18 Lateral Displacements and Interstory Drifts due to Seismic Forces in N-S Direction (SDC D)

Story	δ_{xe} (in.)	δ_x (in.)	Δ (in.)
3	0.58	2.55	0.97
2	0.36	1.58	1.01
1	0.13	0.57	0.57

The interstory drifts Δ computed from the δ_x are also contained in the table. For this structure that does not have plan irregularity Type 1a or 1b of Table 1616.5.1, the drift at story level x is determined by subtracting the design earthquake displacement at the center of mass at the bottom of the story from the design earthquake displacement at the center of mass at the top of the story (IBC 1617.4.6.1):

$$\Delta = \delta_x - \delta_{x-1}$$

The design story drifts Δ must not exceed the allowable story drift Δ_a from Table 1617.3 (IBC 1617.3). For Seismic Use Group II, $\Delta_a = 0.020 h_{sx}$ for buildings 4 stories or less that have been designed to accommodate the story drifts. Thus, for the 13-ft story heights, $\Delta_a = 0.020 \times 13 \times 12 = 3.12$ in. It is evident from Table 4-18 that for all stories, the lateral drifts obtained from the prescribed lateral forces in the N-S direction are less than the limiting value.

P-delta effects. As noted above, P-delta effects were automatically considered in the analysis using SAP2000. However, for illustration purposes, the following procedure can be used to determine whether P-delta effects need to be considered or not in accordance with IBC 1617.4.6.2.

P-delta effects need not be considered when the stability coefficient θ determined by Eq. 16-47 is less than or equal to 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d}$$

where P_x = total unfactored vertical design load at and above level x
Δ = design story drift occurring simultaneously with V_x
V_x = seismic shear force acting between level x and $x - 1$
h_{sx} = story height below level x
C_d = deflection amplification factor

The stability coefficient θ must not exceed θ_{max} determined from Eq. 16-48:

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25$$

where β is the ratio of shear demand to shear capacity between level x and $x - 1$, which may be taken equal to 1.0 when it is not calculated.

Table 4-19 contains the calculations for the N-S direction. It is clear that P-delta effects need not be considered at any of the floor levels. Note that θ_{max} is equal to 0.091 using $\beta = 1.0$.

4.4.3 Wind Load Analysis

4.4.3.1 Wind Forces

Wind forces are determined in accordance with the analytical procedure (Method 2) given in ASCE 6.5 [4.3].

Table 4-19 P-delta Effects (SDC D)

Level	h_{sx} (ft)	P_x (kips)	V_x (kips)	Δ (in.)	θ
3	13	2,077	193	0.97	0.0122
2	13	4,665	354	1.01	0.0155
1	13	7,252	434	0.57	0.0111

In this example, the wind velocity is 90 mph. A summary of the design wind forces in the N-S direction at all floor levels is contained in Table 4-20. Once again it is important to note that the wind directionality factor K_d has been taken equal to 1.0 (see Exception 1 in IBC 1605.2.1).

Table 4-20 Design Wind Forces in N-S Direction (V = 90 mph)

Level	Height above ground level, z (ft)	Total Design Wind Force (kips)
3	39	19.8
2	26	37.1
1	13	33.8
	Σ	90.7

4.4.3.2 Method of Analysis

Similar to the seismic analysis, a three-dimensional analysis of the building was performed for the wind forces contained in Table 4-20 using SAP2000. The modeling assumptions utilized for the seismic analysis were also used for the wind analysis.

Comparing the seismic forces in Table 4-17 to the wind forces in Table 4-20, it is clear that seismic forces will govern the design of the members.

4.4.4 Design for Combined Load Effects

4.4.4.1 Load Combinations

Basic load combinations for strength design are given in IBC 1605.2.1. As noted above, the first exception in this section requires that the non-seismic load combinations of

ACI 9.2 be used for concrete structures. Thus, the following load combinations are utilized in the design of the structural members:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)
4. $1.2D + f_1 L + 1.0E$ (Formula 16-5)
5. $0.9D + 1.0E$ (Formula 16-6)

where D, L, W, and E are the effects due to dead, live, wind, and seismic loads, respectively, and f_1 is equal to either 1.0 for places of public assembly, for live loads in excess of 100 psf, and for parking garage live load, or is equal to 0.5 for other live loads.

The seismic load effect E for use in Formula 16-5, which is the combined effect of horizontal and vertical earthquake-induced forces, is computed from Eq. 16-28 where the effects of gravity and seismic ground motion are additive:

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of horizontal seismic forces
ρ = redundancy coefficient determined in accordance with IBC 1617.2.2 for SDC D, E, or F

Similarly, E for use in Formula 16-6 is computed from Eq. 16-29 where the effects of gravity and seismic ground motion counteract:

$$E = \rho Q_E - 0.2 S_{DS} D$$

According to IBC 1617.2.2, the redundancy coefficient ρ, which shall not be less than 1.0 and need not exceed 1.5, is the largest of the values of ρ_i calculated at each story i from Equation 16-32:

$$\rho_i = 2 - \frac{20}{r_{\max_i}\sqrt{A_i}}$$

For moment frames:

$r_{\max_i}$ = maximum sum of shears in any 2 adjacent columns/total story shear

In the column shear summation, use 70% of shear in columns common to 2 adjacent bays, with moment-resisting connections on opposite sides.

Computations for the redundancy coefficient in the N-S direction for the moment-resisting frame along column line 3 (most heavily loaded frame when center of mass is displaced west of actual location) are summarized in Table 4-21. Subscripts on column shear forces refer to column lines. To illustrate the computations, $r_{\max_i}$ is calculated for the first story as follows:

$$r_{\max_1} = \frac{\text{maximum sum of shears in 2 adjacent columns}}{\text{total story shear}} = \frac{36}{434} = 0.083$$

From Table 4-21, maximum $r_{\max_i} = 0.088$; thus, ρ is:

$$\rho = 2 - \frac{20}{0.088\sqrt{152.5 \times 92.5}} = 0.09 < 1.0$$

Therefore, $\rho = 1.0$. For SDC D, IBC 1617.2.2 requires that ρ be less than or equal to 1.25 for special moment frames, which is satisfied in this case.

Table 4-21 Redundancy Coefficient Calculations in N-S Direction (SDC D)*

Story	Story Shear	Column Shear Forces				$V_A + 0.7V_B$	$0.7(V_B + V_C)$	$0.7V_C + V_D$	$r_{\max_i}$
		V_A	V_B	V_C	V_D				
3	193	6	12	12	6	14	17	14	0.088
2	354	12	21	21	12	27	29	27	0.082
1	434	15	26	26	15	33	36	33	0.083

*Shear forces are in kips.

Once ρ has been computed, load combinations 4 and 5 can be rewritten as follows. Substituting $S_{DS} = 0.45g$ and $\rho = 1.0$ into the equations for E, and then substituting E along with $f_1 = 0.5$ into load combinations 4 and 5 above results in the following:

4a. $1.2D + 0.5L + 1.0Q_E + (0.2 \times 0.45)D = 1.29D + 0.5L + Q_E$

4b. $1.2D + 0.5L + 1.0Q_E - (0.2 \times 0.45)D = 1.11D + 0.5L + Q_E$

5a. $0.9D + 1.0Q_E + (0.2 \times 0.45)D = 0.99D + Q_E$

5b. $0.9D + 1.0Q_E - (0.2 \times 0.45)D = 0.81D + Q_E$

Values of Q_E are obtained from the structural analysis for both sidesway to the left and to the right based on the code-prescribed horizontal seismic forces acting on the building. Also, the special seismic load combinations in IBC 1605.4 are not applicable in this example.

4.4.4.2 Design of Beam B3-C3

Deflection control.

As was shown in Section 4.2.4.2 of this publication, the 18.5-in. depth is adequate for deflection control (see ACI Table 9.5(a)).

Flexural design.

Table 4-22 contains a summary of the governing design bending moments and shear forces for beam B3-C3 at the first floor level due to gravity and seismic forces. The live load on the beam has been reduced in accordance with IBC 1607.9.2. Also, according to the 2002 supplement to the IBC [4.4], orthogonal effects need not be considered for beams.

Requirements for special moment frames are given in ACI 21.2 through 21.5. The factored axial load on the member, which is negligible, is less than $A_g f_c' / 10$; thus, the provisions of ACI 21.3 for flexural members of special moment frames must be satisfied. All other applicable provisions in Chapters 1 through 18 are to be satisfied as well.

Table 4-22 Summary of Design Bending Moments and Shear Forces for Beam B3-C3 at Floor Level 1 (SDC D)

Load Case	Location	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	Support	-275	56
	Midspan	189	
Live (L)	Support	-51	15
	Midspan	34	
Seismic (Q_E)	Support	± 58	± 4
Load Combinations			
$1.4D + 1.7L$	Support	-472	104
	Midspan	322	
$1.29D + 0.5L + Q_E$	Support	-438	84
	Midspan	261	
$0.81D + Q_E$	Support	-165	41
	Midspan	153	

Check limitations on section dimensions per ACI 21.3.1:

- Factored axial compressive force on member is negligible. O.K.
- $\frac{\ell_n}{d} = \frac{(30 \times 12) - 34}{16} = 20.4 > 4$ O.K.

- $\frac{\text{width}}{\text{depth}} = \frac{28}{18.5} = 1.5 > 0.3$ O.K.
- width = 28 in. > 10 in. O.K.
  < width of supporting column + (1.5 × depth of beam)
  < 34 + (1.5 × 18.5) = 61.8 in. O.K.

The required flexural reinforcement is contained in Table 4-23. The provided areas of steel are within the limits prescribed in ACI 21.3.2.1 for maximum and minimum reinforcement. The selected reinforcement satisfies ACI 7.6.1 and 3.3.2 (minimum spacing for concrete placement), ACI 7.7.1 (minimum cover for protection of reinforcement), and ACI 10.6 (maximum spacing for control of flexural cracking).

ACI 21.3.2.2 requires that the positive moment strength at the face of the joint be greater than or equal to 50% of the negative moment strength at that location. This is satisfied, since 325 ft-kips > 485/2 = 243 ft-kips. Also, the negative or positive moment strength at any section along the length of the member must be greater than or equal to 25% of the maximum moment strength provided at the face of either joint. In this case, 25% of the maximum moment strength is equal to 485/4 = 121 ft-kips. Providing 2-No. 9 bars (ϕM_n = 138 ft-kips) satisfies this provision. This also automatically satisfies the requirement that at least 2 bars be continuous at both the top and bottom of the section (ACI 21.3.2.1).

Table 4-23 Required Flexural Reinforcement for Beam B3-C3 at Floor Level 1 (SDC D)

Location	M_u (ft-kips)	A_s^* (in.^{2})	Reinforcement*	ϕM_n (ft-kips)
Support	-472	7.73	8-No. 9	485
Midspan	322	4.96	5-No. 9	325

$$^*A_{s,min} = \frac{3\sqrt{f'_c}\,b_w d}{f_y} = \frac{3\sqrt{4{,}000} \times 28 \times 16}{60{,}000} = 1.42 \text{ in.}^2$$

ACI 21.3.2.1

$$= \frac{200 b_w d}{f_y} = \frac{200 \times 28 \times 16}{60{,}000} = 1.49 \text{ in.}^2 \text{ (governs)}$$

$$A_{s,max} = \rho_{max} b_w d = 0.025 \times 28 \times 16 = 11.20 \text{ in.}^2$$ ACI 21.3.2.1

When reinforcing bars extend through a joint, the column dimension parallel to the beam reinforcement must be at least 20 times the diameter of the largest longitudinal bar for normal weight concrete (ACI 21.5.1.4). In this case, the minimum required column dimension = 20 × 1.128 = 22.6 in., which is less than the 34-in. column width that is provided.

Shear design.

Shear requirements for beams in special moment frames are contained in ACI 21.3.4. The method of determining design shear forces in beams in special moment frames takes into consideration the likelihood of yielding (i.e., plastic hinges forming) at regions near the supports. In general, the shear forces are determined assuming simultaneous hinging at the beam supports under lateral loading. To properly confine the concrete and to maintain lateral support of the longitudinal bars in regions where yielding is expected, the transverse reinforcement requirements of ACI 21.3.3 must also be satisfied.

According to ACI 21.3.4.1, shear forces are computed from statics assuming that moments of opposite sign corresponding to the probable flexural strength M_{pr} act at the joint faces and that the member is loaded with tributary factored gravity load along its span. Sidesway to the right and to the left must be considered when calculating the maximum design shear forces.

The probable flexural strength M_{pr} for a section is determined using the stress in the tensile reinforcement equal to $1.25 f_y$ and a strength reduction factor ϕ equal to 1.0 (ACI 21.0). The following equation can be used to compute M_{pr}:

$$M_{pr} = A_s(1.25 f_y)\left(d - \frac{a}{2}\right)$$

where $a = \dfrac{A_s(1.25 f_y)}{0.85 f'_c b}$

For example, for sidesway to the right, the joint on column line C is subjected to the negative moment M_{pr} that is determined as follows:

For 8-No. 9 top bars, $A_s = 8 \times 1.00 = 8.00$ in.^{2}

$$a = \frac{8.00 \times 1.25 \times 60}{0.85 \times 4 \times 28} = 6.3 \text{ in.}$$

$$M_{pr} = 8.00 \times 1.25 \times 60 \times \left(16 - \frac{6.3}{2}\right) = 7{,}709 \text{ in.-kips} = 642 \text{ ft-kips}$$

Similarly, for the joint on column line B, the positive moment M_{pr} based on 5-No. 9 bars is equal to 439 ft-kips.

The largest shear force associated with seismic effects is obtained from the second of the three load combinations in Table 4-22. Figure 4-10 shows the beam and shear forces due to gravity loads plus probable flexural strengths for sidesway to the right. Due to the symmetric distribution of longitudinal reinforcement in the beam, sidesway to the left gives the same maximum shear force. The factored uniform load on the beam is determined as follows:

$$w_D = [(74+45)\times 30/1{,}000] + \left(\frac{28\times 18.5}{144}\times 0.15\right) = 4.1 \text{ kips/ft}$$

$$w_L = 36\times 30/1{,}000 = 1.1 \text{ kips/ft}$$

$$w_u = 1.29 w_D + 0.5 w_L = 5.8 \text{ kips/ft}$$

The maximum combined shear force as shown in Figure 4-10 (118.6 kips) is larger than the maximum shear force obtained from the structural analysis (84 kips; see Table 4-22).

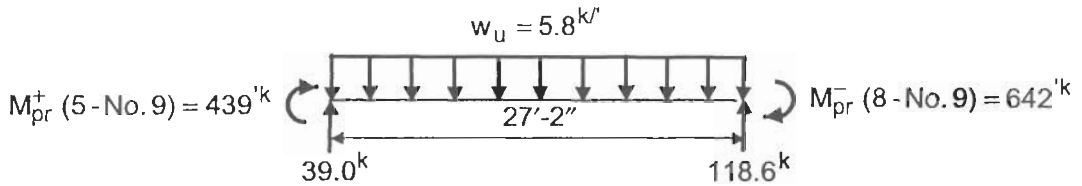


Figure 4-10 Design Shear Forces for Beam B3-C3 (SDC D)

In general, shear strength is provided by both concrete (V_c) and reinforcing steel (V_s). However, according to ACI 21.3.4.2, V_c is to be taken as zero when the earthquake-induced shear force calculated in accordance with ACI 21.3.4.1 is greater than or equal to 50% of the total shear force and the factored axial compressive force including earthquake effects is less than $A_g f_c'/20$. In this example, the beam carries negligible axial forces and the maximum earthquake-induced shear force = (642 + 439)/27.17 = 39.8 kips < 118.6/2 = 59.3 kips. Thus, the nominal shear strength provided by concrete V_c may be computed from ACI Eq. 11-3:

$$V_c = 2\sqrt{f_c'}\, b_w d = 2\sqrt{4{,}000}\times 28\times 16/1{,}000 = 56.7 \text{ kips}$$

The maximum shear force V_s is (ACI 11.1):

$$V_s = \frac{V_u}{\phi} - V_c = \frac{118.6}{0.85} - 56.7 = 82.8 \text{ kips}$$

where the strength reduction factor was taken as 0.85 in accordance with ACI 9.3.4.

Shear strength contributed by shear reinforcement shall not exceed $(V_s)_{max}$ (ACI 11.5.6.9):

$$(V_s)_{max} = 8\sqrt{f'_c}b_w d = 8\sqrt{4{,}000} \times 28 \times 16/1{,}000 = 226.7 \text{ kips} > 82.8 \text{ kips} \qquad \text{O.K.}$$

Also, V_s is less than $4\sqrt{f'_c}b_w d$ = 113.3 kips.

Required spacing s of No. 3 closed stirrups (hoops) for a factored shear force of 82.8 kips is determined from Eq. (11-15):

$$s = \frac{A_v f_y d}{V_s} = \frac{(5 \times 0.11) \times 60 \times 16}{82.8} = 6.4 \text{ in.}$$

Note that 5 legs are required for support of the longitudinal bars (ACI 21.3.3.3).

Maximum allowable hoop spacing within a distance of $2h = 2 \times 18.5 = 37$ in. from the face of the support at each end of the member is the smaller of the following (ACI 21.3.3.2):

- $d/4 = 16/4 = 4.0$ in. (governs)
- 8(diameter of smallest longitudinal bar) = 8 × 1.128 = 9.0 in.
- 24(diameter of hoop bar) = 24 × 0.375 = 9.0 in.
- 12 in.

Use 10-No. 3 hoops at each end of the beam spaced at 4 in. on center with the first hoop located 2 in. from the face of the support (ACI 21.3.3.2).

Where hoops are no longer required, stirrups with seismic hooks at both ends may be used (ACI 21.3.3.4). At a distance of 38 in. from the face of the support:

$$V_u = 118.6 - [5.8 \times (38/12)] = 100.2 \text{ kips}$$

Therefore, the required stirrup spacing for No. 3 stirrups (4 legs) is:

$$s = \frac{A_v f_y d}{V_s} = \frac{(4 \times 0.11) \times 60 \times 16}{(100.2/0.85) - 56.7} = 6.9 \text{ in.} \quad \text{(governs)}$$

$$= \frac{A_v f_y}{50 b_w} = \frac{(4 \times 0.11) \times 60{,}000}{50 \times 28} = 18.9 \text{ in.}$$

Confinement of the longitudinal bars in accordance with ACI 21.3.3.3 must be provided only where hoops are required; that is why only 4 legs are used in this portion of the beam where stirrups with seismic hooks are required.

The maximum allowable spacing of the stirrups is $d/2$ = 8 in. (ACI 21.3.3.4), which is greater than 6.9 in. A 6 in. spacing, starting at 38 in. from the face of the support is sufficient for the remaining portion of the beam.

Reinforcing bar cutoff points.

The negative reinforcement at the supports is 8-No. 9 bars. The location where 6 of the 8 bars can be terminated will be determined.

The third load combination is used to determine the cutoff point of the 6-No. 9 bars (0.81 times the dead load in combination with the probable flexural strengths M_{pr} at the ends of the member), since this combination produces the longest bar lengths. The design flexural strength ϕM_n provided by 2-No. 9 bars is 138 ft-kips. Therefore, the 6-No. 9 bars can be terminated after the required moment strength M_u has been reduced to 138 ft-kips.

The distance x from the support to the location where the moment is equal to 138 ft-kips can readily be determined by summing moments about section a-a in Figure 4-11:

$$\frac{3.3x^2}{2} - 84.6x + 642 = 138$$

Solving for x gives a distance of 6.9 ft from the face of the support.

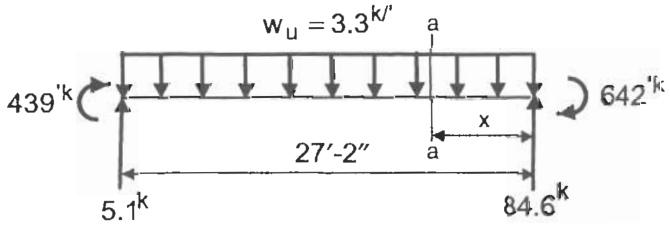


Figure 4-11 Cutoff Location of Negative Bars (SDC D)

The 6-No. 9 bars must extend a distance d = 16 in. (governs) or $12d_b = 12 \times 1.128 = 13.5$ in. beyond the distance x (ACI 12.10.3). Thus, from the face of the support, the total bar length must be at least equal to 6.9 + (16/12) = 8.2 ft.

Also, the bars must extend a full development length ℓ_d beyond the face of the support (ACI 12.10.4), which is determined from ACI Eq. (12-1):

$$\frac{\ell_d}{d_b} = \frac{3}{40}\frac{f_y}{\sqrt{f_c'}}\frac{\alpha\beta\gamma\lambda}{\left(\frac{c+K_{tr}}{d_b}\right)}$$

where α = reinforcement location factor = 1.3 for top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 9 bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5 + 0.375 + \dfrac{1.128}{2} = 2.4 \text{ in.} \\ \\ \dfrac{28 - 2(1.5 + 0.375) - 1.128}{2 \times 7} = 1.7 \text{ in. (governs)} \end{cases}$$

K_{tr} = transverse reinforcement index = 0 (conservative)

$$\frac{c+K_{tr}}{d_b} = \frac{1.7+0}{1.128} = 1.5 < 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.3 \times 1.0 \times 1.0 \times 1.0}{1.5} = 61.7$$

$$\ell_d = 61.7 \times 1.128 = 69.6 \text{ in.} = 5.8 \text{ ft} < 8.2 \text{ ft}$$

Thus, the total required length of the 6-No. 9 bars must be at least 8.2 ft beyond the face of the support.

Note that flexural reinforcement shall not be terminated in a tension zone unless one or more of the conditions of ACI 12.10.5 are satisfied. In this case, the point of inflection is approximately 9.2 ft from the face of the support, which is greater than 8.2 ft. Thus, the No. 9 bars cannot be terminated here unless one of the conditions of ACI 12.10.5 is satisfied. In this case, check if the factored shear force V_u at the cutoff point does not exceed $2\phi V_n/3$ (ACI 12.10.5.1). With No. 3 stirrups at 6 in. on center that are provided in this region of the beam, ϕV_n is determined by ACI Eqs. 11-1 and 11-2:

$$\phi V_n = \phi(V_c + V_s) = 0.85 \times (56.7 + 70.4) = 108.0 \text{ kips}$$

$$\frac{2}{3}\phi V_n = 72.0 \text{ kips}$$

At 8.2 ft from the face of the support, $V_u = 84.6 - (3.3 \times 8.2) = 57.5$ kips, which is less than 72.0 kips. Therefore, the 6-No. 9 bars can be terminated at 8.2 ft from the face of the support.

Flexural reinforcement splices.

According to ACI 21.3.2.3, lap splices of flexural reinforcement must not be placed within a joint, within a distance $2h$ from the face of the joint (plastic hinge region), or at locations where analysis indicates flexural yielding due to inelastic lateral displacements of the frame. Lap splices must be confined by hoops or spiral reinforcement along the entire lap length, and the maximum spacing of the transverse reinforcement is $d/4$ or 4 in. In lieu of lap splices, mechanical and welded splices conforming to ACI 21.2.6 and 21.2.7, respectively, may be used (ACI 21.3.2.4).

Lap splices are determined for the No. 9 bottom bars. Since all of the bars are to be spliced within the required length, a Class B splice must be used (ACI 12.15.1, 12.15.2):

Class B splice length = $1.3\,\ell_d$

The development length ℓ_d is determined from ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 9 bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5 + 0.375 + \dfrac{1.128}{2} = 2.4 \text{ in.} \quad \text{(governs)} \\[2ex] \dfrac{28 - 2(1.5 + 0.375) - 1.128}{2 \times 4} = 2.9 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index

$$= \frac{A_{tr} f_{yt}}{1{,}500sn} = \frac{4 \times 0.11 \times 60{,}000}{1{,}500 \times 4 \times 5} = 0.9$$

$$\frac{c + K_{tr}}{d_b} = \frac{2.4 + 0.9}{1.128} = 2.9 > 2.5, \quad \text{use } 2.5$$

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.0 \times 1.0 \times 1.0 \times 1.0}{2.5} = 28.5$$

$$\ell_d = 28.5 \times 1.128 = 32.2 \text{ in.} = 2.7 \text{ ft}$$

Class B splice length = $1.3\,\ell_d = 1.3 \times 2.7 = 3.5$ ft

Use a 3 ft-6 in. splice length.

Figure 4-12 shows the reinforcement details for beam B3-C3.

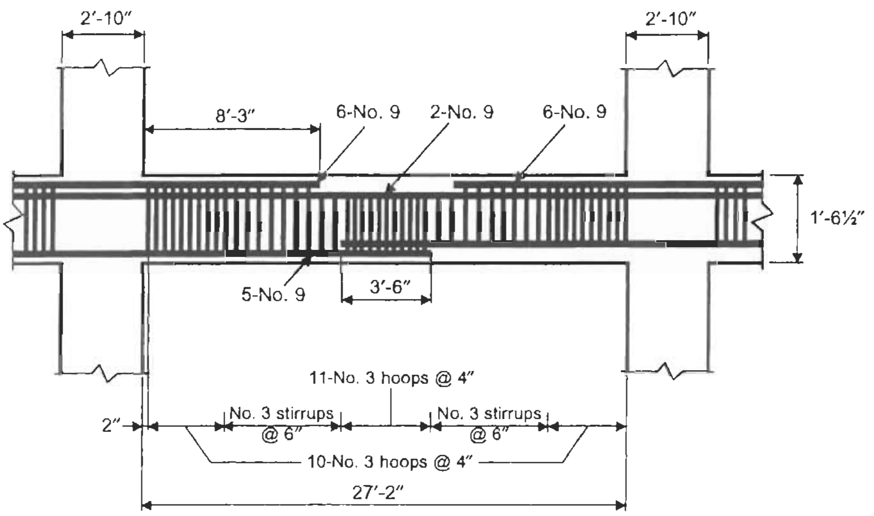


Figure 4-12 Reinforcement Details for Beam B3-C3 at Floor Level 1 (SDC D)

4.4.4.3 Design of Column C3

This section outlines the design of column C3 supporting the first floor level. Table 4-24 contains a summary of the design axial forces, bending moments, and shear forces on this column for gravity and seismic loads.

Table 4-24 Summary of Design Axial Forces, Bending Moments, and Shear Forces on Column C3 between Ground and 1st Floor Level (SDC D)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	385	0	0
Live (L)*	64	0	0
Seismic (Q_E)	0	$\pm$ 366	$\pm$ 26
Load Combinations			
$1.4D + 1.7L$	648	0	0
$1.29D + 0.5L + Q_E$	529	366	26
$0.81D + Q_E$	312	-366	-26

* Live load reduced per IBC 1607.9

According to the supplement to the IBC [4.4], orthogonal effects on this member need not be considered, since the axial forces due to seismic forces in both directions (which are about equal to zero) are less than 20% of the axial load design strength.

Since the factored axial compressive force exceeds $A_g f_c' / 10 = 34^2 \times 4/10 =$ 462 kips, the provisions of ACI 21.4 are applicable. Thus, the following two criteria must be satisfied (ACI 21.4.1):

- Shortest cross-sectional dimension = 34 in. > 12 in. O.K.
- Ratio of shortest cross-sectional dimension to perpendicular dimension = 1.0 > 0.4 O.K.

Design for axial force and bending.

Based on the governing load combinations in Table 4-24, a 34 × 34 in. column with 12-No. 9 bars ($\rho_g = 1.04\%$) is adequate for column C3 supporting the first floor level. Figure 4-13 contains the interaction diagram for this column. As noted above, slenderness effects need not be considered since P-delta effects were included in the analysis. Also, the provided reinforcement ratio is within the allowable range of 1% and 6% (ACI 21.4.3.1).

Relative Flexural Strength of Columns and Girders.

ACI 21.4.2 requires that the sum of flexural strengths of columns at a joint must be greater than or equal to 6/5 times the sum of flexural strengths of girders framing into that joint. The intent is to provide columns with sufficient strength so that they will not yield prior to the beams. Yielding at both ends of a column prior to the beams could result in total collapse of the structure. Only seismic load combinations need to be considered when checking the relative strengths of columns and girders.

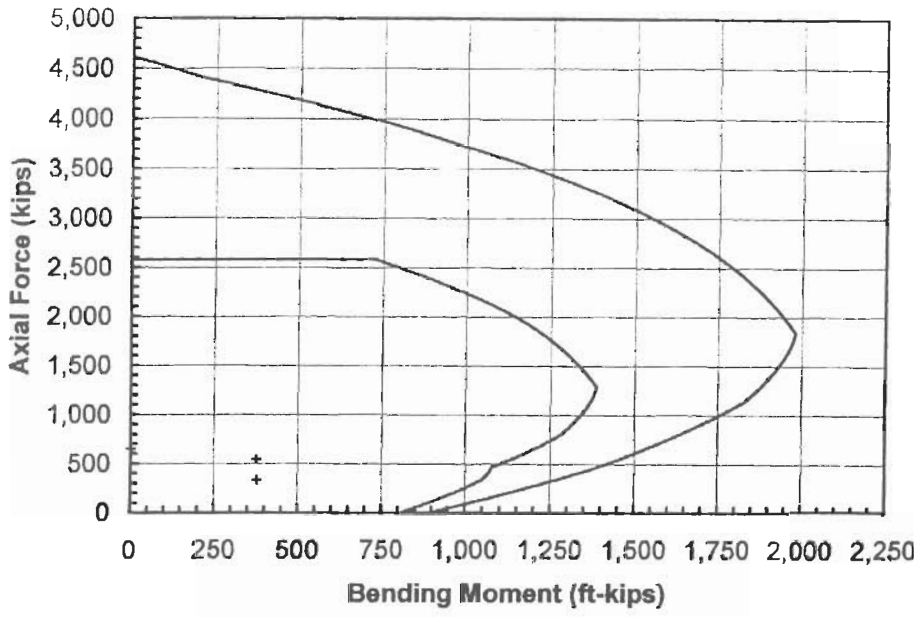


Figure 4-13 Design and Nominal Strength Interaction Diagrams for Column C3 Supporting Floor Level 1 (SDC D)

Based on the reinforcement in beam B3-C3 (see Table 4-23), the negative nominal moment strength $M_n^- = 485/0.9 = 539$ ft-kips and the positive nominal moment strength $M_n^+ = 325/0.9 = 361$ ft-kips. Therefore, $\sum M_g = 539 + 361 = 900$ ft-kips (see Figure 4-14 for sidesway to the left; due to symmetric distribution of flexural reinforcement, sidesway to the right yields the same results for the negative and positive moment strengths).

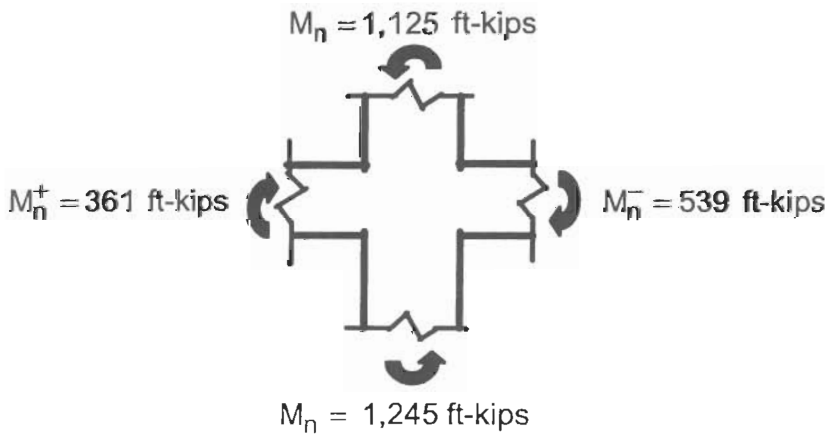


Figure 4-14 Relative Flexural Strength of Columns and Girders (SDC D)

Column flexural strength is determined for the factored axial force resulting in the lowest flexural strength, consistent with the direction of lateral forces considered. For the upper end of the lower column framing into the joint (i.e., the column supporting floor level 1), the minimum M_n = 1,245 ft-kips, which corresponds to P_u = 312 kips (see Figure 4-13). Similarly, for the lower end of the upper column framing into the joint (i.e., the column supporting floor level 2), M_n = 1,125 ft-kips, which corresponds to P_u = 199 kips. Therefore, $\sum M_c$ = 2,370 ft-kips.

Check Eq. 21-1:

$$\sum M_c = 2{,}370 \text{ ft-kips} > \frac{6}{5}\sum M_g = \frac{6}{5} \times 900 = 1{,}080 \text{ ft-kips} \quad \text{O.K.}$$

Design for shear.

Shear requirements for columns in special moment frames are contained in ACI 21.4.5. Similar to beams, the method of determining design shear forces in columns takes into consideration the likelihood of yielding (i.e., plastic hinges forming) in regions near the ends of the column. To properly confine the concrete and to maintain lateral support of the longitudinal bars in regions where yielding is expected, the transverse reinforcement requirements of ACI 21.4.4 must also be satisfied.

Confinement reinforcement. Special transverse reinforcement for confinement is required over a distance ℓ_o from each joint face at both column ends where ℓ_o is equal to the largest of (ACI 21.4.4.4):

- Depth of member = 34 in. (governs)
- Clear span/6 = [(13 × 12) – 18.5]/6 = 22.9 in.
- 18 in.

Transverse reinforcement within the distance ℓ_o shall not be spaced greater than the smallest of (ACI 21.4.4.2):

- Minimum member dimension/4 = 34/4 = 8.5 in.
- 6(diameter of longitudinal reinforcement) = 6 × 1.128 = 6.8 in.
- $s_x = 4 + \left(\dfrac{14 - h_x}{3}\right) = 4 + \left(\dfrac{14 - 11.3}{3}\right) = 4.9$ in. (governs)

where h_x = maximum horizontal spacing of hoop or crosstie legs on all faces of the 34×34 in. column (ACI 21.4.4.3)

$$= \frac{34-2(1.5+0.5)-1.128}{3}+1.128+0.5 = 11.3 \text{ in.} < 14 \text{ in.} \quad \text{O.K.}$$

assuming No. 4 rectangular hoops with crossties around every longitudinal bar. Therefore, try 4 in. spacing.

Minimum required cross-sectional area of rectangular hoop reinforcement A_{sh} is the larger value obtained from Eqs. 21-3 and 21-4:

$$A_{sh} = \frac{0.3sh_c f'_c}{f_{yh}}\left[\left(\frac{A_g}{A_{ch}}\right)-1\right] = \frac{0.3\times4\times30.5\times4}{60}\left[\left(\frac{34^2}{961}\right)-1\right] = 0.50 \text{ in.}^2$$

$$= \frac{0.09sh_c f'_c}{f_{yh}} = \frac{0.09\times4\times30.5\times4}{60} = 0.73 \text{ in.}^2 \quad \text{(governs)}$$

where h_c = cross-sectional dimension of column core measured center-to-center of confinement reinforcement

$= 34 - 2[1.5 + (0.5/2)] = 30.5$ in.

A_{ch} = cross-sectional area of member measured out-to-out of transverse reinforcement

$= [34 - (2 \times 1.5)]^2 = 961$ in.^{2}

Using No. 4 hoops with 2 crossties provides $A_{sh} = 4 \times 0.2 = 0.8$ in.^{2}, which is greater than the minimum required area from Eq. 21-4. Use 4 in. spacing for the transverse reinforcement at the column ends.

Transverse reinforcement for shear. According to ACI 21.4.5.1, shear forces are computed from statics assuming that moments of opposite sign act at the joint faces corresponding to the probable flexural strengths M_{pr} associated with the range of factored axial loads on the column. For columns in the first story, which is applicable in this case, it is possible to develop the probable flexural strength of the column at its base. At the top of a first floor column, probable flexural strengths of the beams framing into the joint will usually control. Thus, for a first story column, shear forces are computed based on the probable flexural strength of the column at the base and the probable flexural strengths of the beams at the top. Also, the design shear force must not be taken less than that determined from the structural analysis. Sidesway to the right and to the left must be considered when calculating the maximum design shear forces.

Figure 4-15 contains the design strength interaction diagram for column C3 with f_y = 75 ksi and ϕ = 1.0. At the base of the column, the largest M_{pr} is equal to 1,599 ft-kips, which corresponds to an axial load equal to 529 kips (see Table 4-24).

At the top of the column, the positive probable flexural strength of the beam framing into the joint at the face of the column is 439 ft-kips (see Figure 4-10). The negative probable flexural strength of the beam on the other side of the column is 642 ft-kips. Thus, the moment transferred to the top of the column is (439 + 642)/2 = 541 ft-kips, which is less than 1,599 ft-kips.

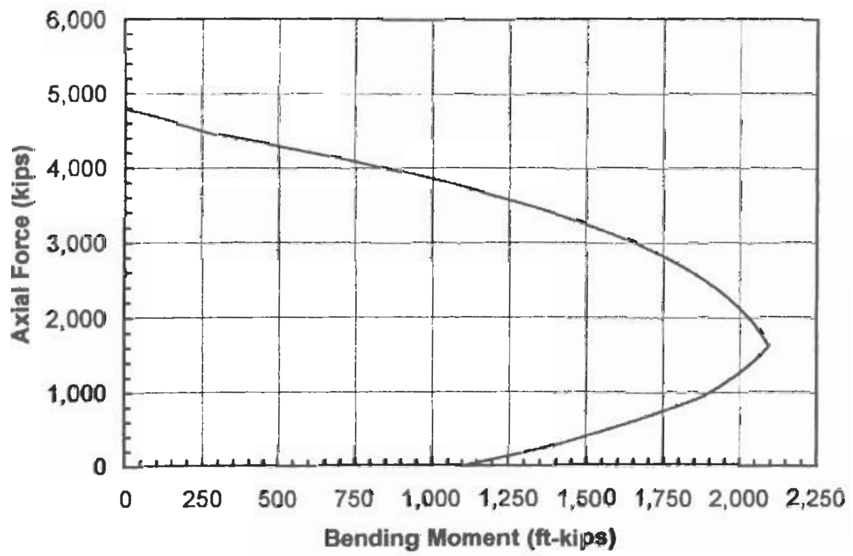


Figure 4-15 Design Strength Interaction Diagram for Column C3 with f_y = 75 ksi and ϕ = 1.0 (SDC D)

The maximum shear force V_e is equal to:

$$V_e = \frac{1{,}599 + 541}{13 - (18.5/12)} = 187 \text{ kips}$$

This design shear force is larger than the maximum shear force obtained from analysis, which is 26 kips (see Table 4-24).

Since the factored axial forces including earthquake effects are greater than $A_g f_c' / 20$ = 231 kips, the shear strength of the concrete may be used (ACI 21.4.5.2). The shear capacity of the column is checked using ACI Eq. 11-4 for members subjected to axial compression:

$$V_c = 2\left(1 + \frac{N_u}{2{,}000A_g}\right)\sqrt{f'_c}\,b_w d$$

$$= 2\left(1 + \frac{312{,}000}{2{,}000 \times 34^2}\right)\sqrt{4{,}000} \times 34 \times 24.2/1{,}000 = 118.1 \text{ kips}$$

$$\phi V_c = 0.85 \times 118.1 = 100.4 \text{ kips} < V_u = 187 \text{ kips}$$

where N_u = 312 kips is the smallest axial force on the section (see Table 4-24) and d = 24.2 in. was obtained from a strain compatibility analysis.

Determine required spacing of transverse reinforcement from Eq. 11-15:

$$s = \frac{A_v f_y d}{\dfrac{V_u}{\phi} - V_c} = \frac{(4 \times 0.2) \times 60 \times 24.2}{\dfrac{187}{0.85} - 118.1} = 11.4 \text{ in.}$$

Thus, the No. 4 hoops at 4 in. on center required for confinement over the distance ℓ_o at the column ends is also adequate for shear.

According to ACI 21.4.4.6, the remainder of the column must contain hoop reinforcement with center-to center spacing not to exceed 6 times the diameter of the longitudinal column bars = 6 × 1.128 = 6.8 in. or 6 in. (governs). For detailing simplicity, use a 4 in. spacing over the entire length of the column.

Splice length of longitudinal reinforcement.

Lap splices in columns of special moment frames are permitted only within the center half of the member and must be designed as tension lap splices (ACI 21.4.3.2). Also, they must be confined over the entire lap length with transverse reinforcement conforming to ACI 21.4.4.2 and 21.4.4.3. In lieu of lap splices, mechanical splices conforming to ACI 21.2.6 and welded splices conforming to ACI 21.2.7.1 may be utilized.

Since all of the bars are to be spliced at the same location, a Class B splice is required (ACI 12.15.1, 12.15.2).

From ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 7 and larger bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5 + 0.5 + \dfrac{1.128}{2} = 2.6 \text{ in.} \quad \text{(governs)} \\ \\ \dfrac{34 - 2(1.5 + 0.5) - 1.128}{3 \times 2} = 4.8 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index

$$= \frac{A_{tr} f_{yt}}{1{,}500 sn} = \frac{4 \times 0.20 \times 60{,}000}{1{,}500 \times 4 \times 4} = 2.0$$

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

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.0 \times 1.0 \times 1.0 \times 1.0}{2.5} = 28.5$$

$$\ell_d = 28.5 \times 1.128 = 32.2 \text{ in.} = 2.7 \text{ ft}$$

Class B splice length $= 1.3 \ell_d = 3.5$ ft

Use a 3 ft-6 in. splice length.

Reinforcement details for column C3 are shown in Figure 4-16.

4.4.4.4 Design of Beam-Column Joint

This section outlines the design of interior and exterior beam-column connections along column line 3. Similar calculations can be performed for joints along other column lines.

The overall integrity of special moment frames is dependent on the behavior of beam-column joints. Degradation of joints can result in large lateral deformations that can cause excessive damage or even failure.

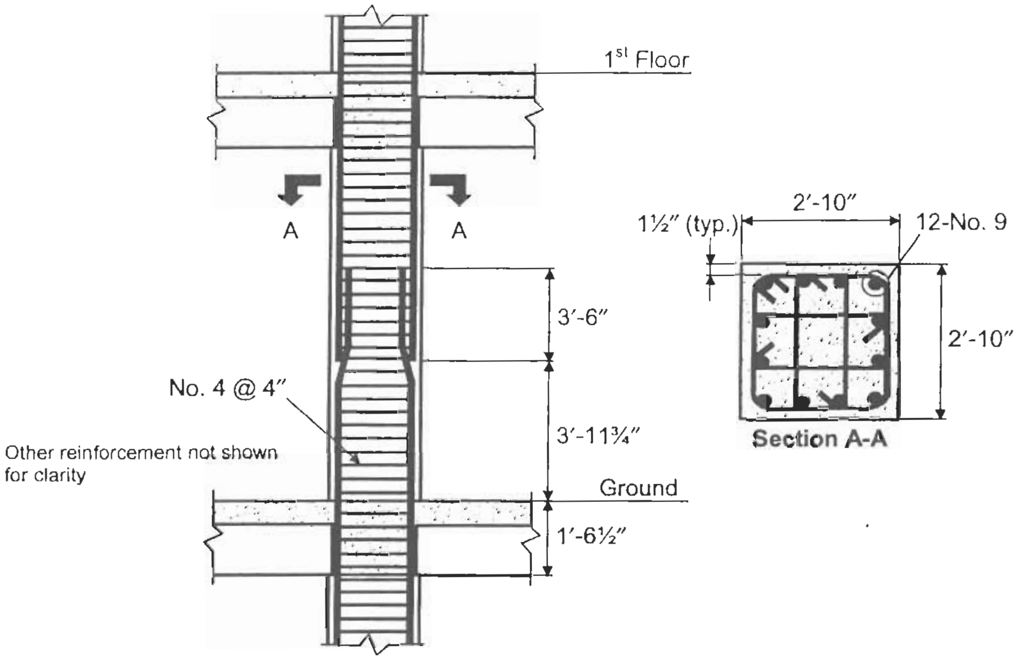


Figure 4-16 Reinforcement Details for Column C3 Supporting the 1st Floor Level (SDC D)

Interior Joint.

Transverse reinforcement for confinement. Transverse reinforcement in a beam-column joint is required to adequately confine the concrete to ensure its ductile behavior and to allow it to maintain its load-carrying capacity even after possible spalling of the outer shell. ACI 21.5.2 requires that transverse hoop reinforcement in accordance with ACI 21.4.4 be provided within the joint, unless the joint is confined on all four sides by beams that have widths equal to at least 75% the column width. When joints are adequately confined on four sides, transverse reinforcement within the joint may be reduced to one-half of that required by ACI 21.4.4 and the hoop spacing is permitted to be a maximum of 6 in.

At an interior joint in the example building, beams frame into two sides of the column (joists do not provide confinement in perpendicular direction). According to ACI 21.5.3, these beams provide confinement to the joint, since the beam width (28 in.) is greater than 75 percent of the column face (25.5 in.). Therefore, transverse reinforcement must be provided in accordance with ACI 21.4.4 as noted above. Use the transverse reinforcement required at the column ends (No. 4 @ 4 in.) through the joint.

Shear strength of joint. Figure 4-17 shows the interior beam-column joint at the first floor level. The shear strength is checked in the N-S direction in accordance with ACI 21.5.3. The shear force at section *x-x* is obtained by subtracting the column shear force from the sum of the tensile force in the top beam reinforcement and the compressive force at the top of the beam on the opposite face of the column. Since development of inelastic rotations at the joint face is associated with strains in the beam flexural reinforcement significantly greater than the yield strain, joint shear forces generated by beam reinforcement are calculated based on a stress in the reinforcement equal to $1.25 f_y$ (ACI 21.5.1.1):

$$T_1 \text{ (8 - No.9)} = A_s(1.25 f_y) = (8\times 1.00)\times(1.25\times 60) = 600 \text{ kips}$$

$$T_2 \text{ (5 - No.9)} = (5\times 1.00)\times(1.25\times 60) = 375 \text{ kips}$$

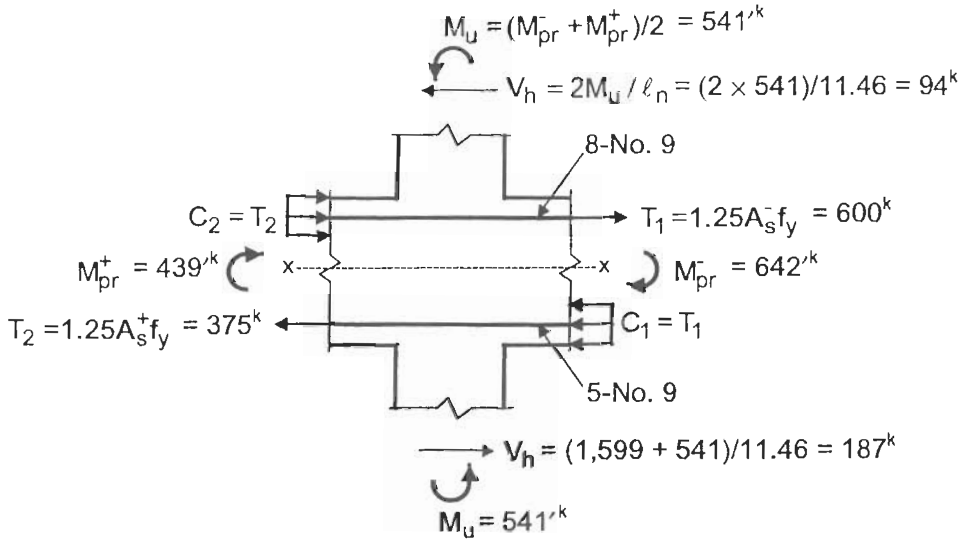


Figure 4-17 Shear Analysis of Interior Beam-Column Joint in N-S Direction (SDC D)

Column horizontal shear force V_h can be obtained by assuming that adjoining floors are deformed so that plastic hinges form at the ends of the beams. For the beams in this example, $M^-_{pr} = 642$ ft-kips and $M^+_{pr} = 439$ ft-kips (see Figure 4-10).

Since the lengths of the columns above and below the joint are equal, moments M_u in the columns above and below the joint are (642 + 439)/2 = 541 ft-kips. Shear force V_h in the column at the top of the joint is:

$$V_h = \frac{2M_u}{\ell_n} = \frac{2\times 541}{13-(18.5/12)} = 94 \text{ kips}$$

Shear force V_h in the column at the bottom of the joint is (see Section 4.4.4.3 of this publication for calculation of moments in column):

$$V_h = \frac{M_u}{\ell_n} = \frac{1{,}599+541}{13-(18.5/12)} = 187 \text{ kips}$$

The maximum net shear force at section x-x is $T_1 + C_2 - V_h = 600 + 375 - 94 = 881$ kips

For a joint confined on two opposite faces, nominal shear strength ϕV_c is determined from ACI 21.5.3.1:

$$\phi V_c = \phi 15\sqrt{f_c'}A_j = 0.85 \times 15\sqrt{4{,}000} \times 1{,}156/1{,}000 = 932 \text{ kips} > 881 \text{ kips} \quad \text{O.K.}$$

where A_j = effective cross-sectional area within a joint in a plane parallel to the plane of reinforcement generating shear in the joint (see ACI Figure R21.5.3). The joint depth is the overall depth of the column in the direction of analysis, which is 34 in. The effective width of the joint is the smaller of (1) beam width + joint depth = 28 + 34 = 62 in. or (2) beam width plus twice the smaller perpendicular distance from the edge of the beam to the edge of the column = 28 + (2 × 3) = 34 in. (governs). Thus, A_j = 34 × 34 = 1,156 in.^{2}

Joint shear strength is a function of concrete strength and effective cross-sectional area A_j only. Tests results show that shear strength of a joint is not altered significantly with changes in transverse reinforcement, provided a minimum amount of such reinforcement is present. Therefore, it is essential that at least minimum transverse reinforcement as specified in ACI 21.5.2 be provided through the joint regardless of the magnitude of calculated shear force in the joint. In cases when the net shear force exceeds the design strength prescribed in ACI 21.5.3.1, only the concrete strength or the effective cross-sectional area can be increased to increase shear capacity.

Reinforcement details for an interior joint are shown in Figure 4-18.

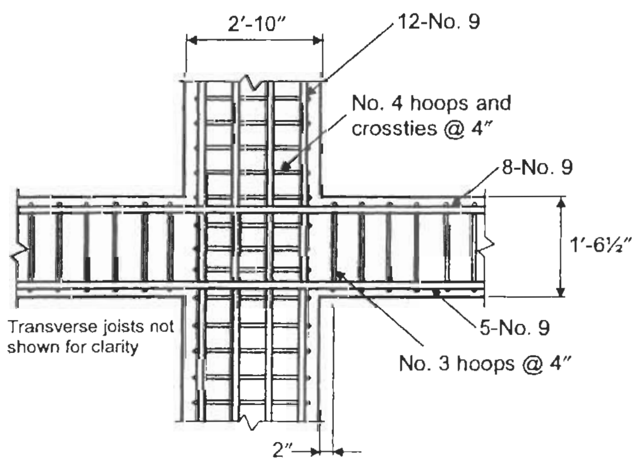


Figure 4-18 Reinforcement Details at Interior Beam-Column Joint (SDC D)

Exterior Joint.

It is assumed for purposes of this example that the beam framing into the edge column along column line 3 has the same flexural reinforcement as the interior beams along this column line.

Transverse reinforcement for confinement. Since an exterior joint is confined on less than four sides, transverse hoop reinforcement in accordance with ACI 21.4.4 must be provided within the joint (ACI 21.5.2.1). For detailing simplicity, use the transverse reinforcement required at the column ends through the joint.

Shear strength of joint. Figure 4-19 shows the exterior joint at the first floor level. In this case, the shear force at section *x-x* is determined by subtracting the column shear force from the tensile force in the top beam reinforcement.

Since the lengths of the columns above and below the joint are equal, moments M_u in the columns above and below the joint are 642/2 = 321 ft-kips. Shear force V_h in the column at the top of the joint is:

$$V_h = \frac{2M_u}{\ell_n} = \frac{2\times 321}{13-(18.5/12)} = 56 \text{ kips}$$

The net shear force at section *x-x* is $T_1 - V_h = 600 - 56 = 544$ kips.

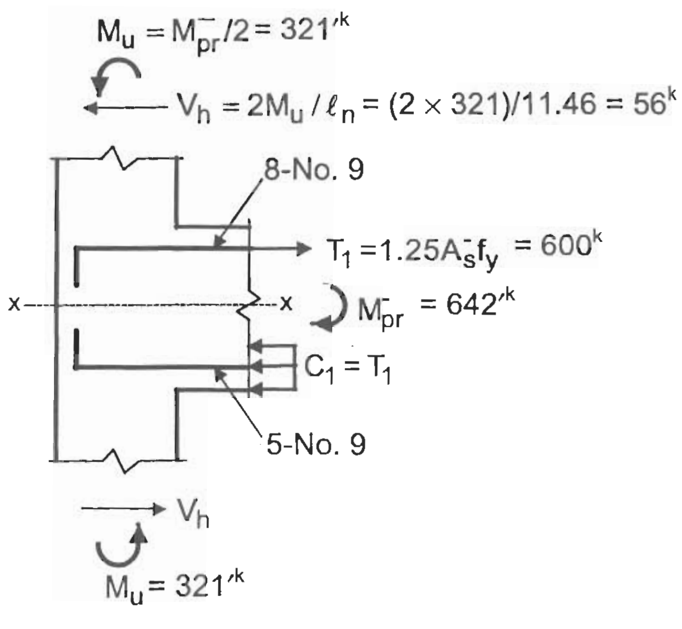


Figure 4-19 Shear Analysis of Exterior Beam-Column Joint in N-S Direction (SDC D)

For a joint confined on one face, the nominal shear strength ϕV_c is determined from ACI 21.5.3.1:

$$\phi V_c = \phi 12\sqrt{f_c'}A_j = 0.85 \times 12\sqrt{4{,}000} \times 30^2 / 1{,}000 = 581 \text{ kips} > 544 \text{ kips} \quad \text{O.K.}$$

Beam flexural reinforcement terminated in a column must extend to the far face of the confined column core and must be anchored in tension and compression according to ACI 21.5.4 and Chapter 12, respectively (ACI 21.5.1.3). The development length ℓ_{dh} for a bar with a standard 90-degree hook in normal weight concrete is the largest of (ACI 21.5.4.1):

- 8(diameter of longitudinal bar) = 8 × 1.128 = 9.0 in.
- 6 in.
- $\dfrac{f_y d_b}{65\sqrt{f_c'}} = \dfrac{60{,}000 \times 1.128}{65\sqrt{4{,}000}} = 16.5$ in. (governs)

Required development lengths for both the flexural reinforcement can be accommodated within the 30-in. deep column.

Reinforcement details for the exterior joint are shown in Figure 4-20.

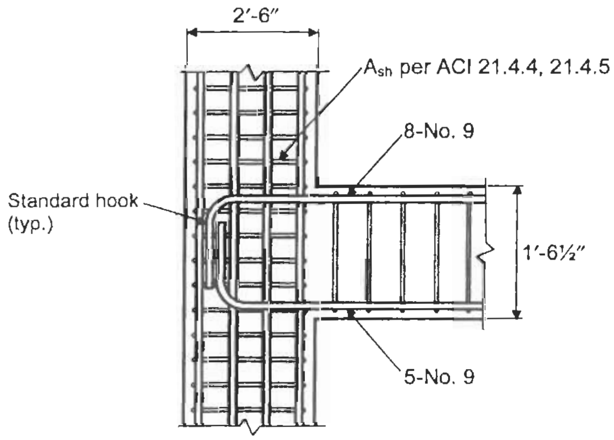


Figure 4-20 Reinforcement Details at Exterior Beam-Column Joint (SDC D)

4.5 DESIGN FOR SDC D – CALIFORNIA

The 3-story school building in Figure 4-1 is assumed to be located in San Francisco. Typical structural members are designed and detailed for combined effects of gravity, wind, and seismic forces.

4.5.1 Design Data

- Building Location: San Francisco, CA (zip code 94105)

- Material Properties

  Concrete: $f'_c = 4{,}000$ psi, $w_c = 150$ pcf
  Reinforcement: $f_y = 60{,}000$ psi

- Service Loads

  Live loads: roof = 20 psf
  floor = 60 psf (average between 40 psf for floors and 80 psf for corridors above first floor level; IBC Table 1607.1). In addition, floor is to be checked for 1,000 lb concentrated live load uniformly distributed over an area of 2.5 sq ft (IBC 1607.4).

  Superimposed dead loads: roof = 10 psf + 200 kips for penthouse
  floor = 45 psf (20 psf permanent partitions + 25 psf ceiling, etc.)

- Seismic Design Data

  For zip code 94105: $S_S = 1.50g, S_1 = 0.61g$ [4.1]
  Site Class D (stiff soil profile; IBC Table 1615.1.1)
  For Category (Seismic Use Group) II occupancy, $I_E = 1.25$ (IBC Table 1604.5)

- Wind Design Data

  Basic wind speed = 85 mph (IBC Figure 1609)
  Exposure B (IBC 1609.4)
  For Category II occupancy, $I_W = 1.15$ (IBC Table 1604.5)

- Member Dimensions

  Joists: 14 + 4.5 × 6 + 66 (74 psf)
  Interior beams: 30 × 18.5 in.
  Spandrel beams: 26 × 18.5 in.
  Interior columns: 36 × 36 in.
  Exterior columns: 32 × 32 in.

4.5.2 Seismic Load Analysis

4.5.2.1 Seismic Design Category (SDC)

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1-second period S_{M1} are determined from IBC Eqs. 16-16 and 16-17, respectively. According to IBC 1616.6.3, for regular structures 5 stories or less having a period determined in accordance with IBC 1617.4.2 of 0.5 sec or less, S_{DS} and S_{D1} need not exceed the values calculated using values of S_S = 1.50g and S_1 = 0.60g, respectively. The 3-story example building is regular according to IBC 1616.6.3, and it is shown below that its period is less than 0.5 sec. Therefore,

$$S_{MS} = F_a S_S = 1.00 \times 1.50 = 1.50g$$

$$S_{M1} = F_v S_1 = 1.50 \times 0.60 = 0.90g$$

The values of F_a and F_v for Site Class D in the equations above are contained in IBC Table 1615.1.2(1) and Table 1615.1.2(2), respectively.

Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from Eqs. 16-18 and 16-19:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 1.50 = 1.00g$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.90 = 0.60g$$

Tables 1616.3(1) and 1616.3(2) are utilized to determine the SDC. From Table 1616.3(1) for Seismic Use Group II and S_{DS} = 1.00g, the SDC is D. Similarly, from Table 1616.3(2), the SDC is D for S_{D1} = 0.60g. Thus, the SDC is D for this building.

4.5.2.2 Seismic Forces

Since the building does not have plan irregularity Type 1a, 1b, or 4 of Table 1616.5.1 or vertical irregularity Type 1a, 1b, 4, or 5 of Table 1616.5.2, it can be considered regular (IBC 1616.6.3). For this regular building that is less than 240 ft in height, Table 1616.6.3 allows the equivalent lateral force procedure in IBC 1617.4 to be used to compute the seismic base shear V (see Eq. 16-34):

$$V = C_s W$$

where C_s is the seismic response coefficient determined in accordance with IBC 1617.4.1.1 and W is the effective weight of the structure defined in IBC 1617.4. For the member sizes and superimposed dead loads given above, W = 6,484 kips (see Table 4-25 below).

To satisfy the provisions of IBC 1910.5.1 for structures assigned to SDC D, a special reinforced concrete moment frame must be used. For this system, the response modification coefficient R = 8 and the deflection amplification factor C_d = 5.5 (IBC Table 1617.6).

Approximate period (T_a). The fundamental period of the building is determined in accordance with Eq. 16-39 in IBC 1617.4.2:

Building period coefficient $C_T = 0.03$
Building height $h_n = 39$ ft
Period $T_a = C_T (h_n)^{3/4} = 0.030 \times (39)^{3/4} = 0.47$ sec

Seismic base shear (V). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.60}{\left(\frac{8}{1.25}\right)\times 0.47} = 0.200$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{1.00}{\left(\frac{8}{1.25}\right)} = 0.156$$

Also, C_s must not be less than the following value from Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 1.00 \times 1.25 = 0.055$$

For buildings assigned to SDC E or F and for those buildings for which $S_1 \geq 0.6g$, C_s shall not be taken less than that computed from Eq. 16-38. Since $S_1 = 0.60g$, Eq. 16-38 is applicable, even though the SDC is D:

$$C_s = \frac{0.5 S_1}{R / I_E} = \frac{0.5 \times 0.60}{8 / 1.25} = 0.047$$

In this case, the lower limit is 0.055 from Eq. 16-37.

Thus, Eq. 16-35 governs, and the base shear V is:

$$V = C_s W = 0.156 \times 6{,}484 = 1{,}012 \text{ kips}$$

Vertical distribution of seismic forces. The lateral forces F_x and the story shears V_x are contained in Table 4-25. For $T = 0.47$ sec, $k = 1.0$.

Table 4-25 Seismic Forces and Story Shears (SDC D)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
3	1,844	39	71,916	448	448
2	2,320	26	60,320	376	824
1	2,320	13	30,160	188	1,012
Σ	6,484		162,396	1,012	

4.5.2.3 Method of Analysis

A three-dimensional analysis of the building was performed in the N-S and E-W directions for the seismic forces contained in Table 4-25 using SAP2000 [4.2]. In the model, rigid diaphragms were assigned at each floor level, and rigid-end offsets were defined at the ends of the horizontal members so that results were automatically obtained at the faces of the supports. The stiffness properties of the members were input assuming cracked sections. In lieu of a more accurate analysis, the following cracked section properties were used:

- Beams: $I_{eff} = 0.5 I_g$
- Columns: $I_{eff} = 0.7 I_g$

where I_g is the gross moment of inertia of the section. P-delta effects were also considered in the analysis.

In accordance with IBC 1617.4.4.4, the center of mass was displaced each way from its actual location a distance equal to 5 percent of the building dimension perpendicular to the applied forces to account for accidental torsion in seismic design.

For all structures assigned to SDC D and higher, IBC 1620.3.5 requires that orthogonal effects of the seismic forces be considered for design and detailing of the components of the seismic-force-resisting system. The orthogonal combination procedure in IBC 1620.2.2 is permitted to be used to satisfy the requirements of IBC 1620.3.5. In the 2002 supplement to the 2000 IBC [4.4], the orthogonal combination procedure is required only for columns or walls that form part of two or more intersecting seismic-force-resisting systems and are subjected to axial loads due to seismic forces greater than or equal to 20% of the axial load design strength.

4.5.2.4 Story Drift and P-delta Effects

Story drift determination. Table 4-26 contains the displacements δ_{xe} obtained from the 3-D static, elastic analyses using the design seismic forces in the N-S direction, including accidental torsional effects. The table also contains the design earthquake displacement δ_x computed by IBC Eq. 16-46:

$$\delta_x = \frac{C_d \delta_{xe}}{I_E}$$

where C_d is the deflection amplification factor in Table 1617.6, which depends on the seismic-force-resisting system. As noted above, C_d is equal to 5.5 for a special reinforced concrete moment frame.

Table 4-26 Lateral Displacements and Interstory Drifts due to Seismic Forces in N-S Direction (SDC D)

Story	δ_{xe} (in.)	δ_x (in.)	Δ (in.)
3	1.15	5.06	1.94
2	0.71	3.12	2.02
1	0.25	1.10	1.10

The interstory drifts Δ computed from the δ_x are also contained in the table. For this structure that does not have plan irregularity Type 1a or 1b of Table 1616.5.1, the drift at story level x is determined by subtracting the design earthquake displacement at the center of mass at the bottom of the story from the design earthquake displacement at the center of mass at the top of the story (IBC 1617.4.6.1):

$$\Delta = \delta_x - \delta_{x-1}$$

The design story drifts Δ must not exceed the allowable story drift Δ_a from Table 1617.3 (IBC 1617.3). For Seismic Use Group II, $\Delta_a = 0.020 h_{sx}$ for buildings 4 stories or less that have been designed to accommodate the story drifts. Thus, for the 13-ft story heights, $\Delta_a = 0.020 \times 13 \times 12 = 3.12$ in. It is evident from Table 4-26 that for all stories, the lateral drifts obtained from the prescribed lateral forces in the N-S direction are less than the limiting value.

P-delta effects. As noted above, P-delta effects were automatically considered in the analysis using SAP2000. Section 4.4.2.4 of this publication illustrates the procedure to determine whether P-delta effects need to be considered or not in accordance with IBC 1617.4.6.2. Similar calculations for this example show that P-delta effects need not be considered.

4.5.3 Wind Load Analysis

4.5.3.1 Wind Forces

Wind forces are determined in accordance with the analytical procedure (Method 2) given in ASCE 6.5 [4.3].

In this example, the wind velocity is 85 mph. A summary of the design wind forces in the N-S direction at all floor levels is contained in Table 4-27. Once again it is important to note that the wind directionality factor K_d has been taken equal to 1.0 (see Exception 1 in IBC 1605.2.1).

Table 4-27 Design Wind Forces in N-S Direction (V = 85 mph)

Level	Height above ground level, *z* (ft)	Total Design Wind Force (kips)
3	39	17.7
2	26	33.1
1	13	30.2
	Σ	81.0

4.5.3.2 Method of Analysis

Similar to the seismic analysis, a three-dimensional analysis of the building was performed for the wind forces contained in Table 4-27 using SAP2000. The modeling assumptions utilized for the seismic analysis were also used for the wind analysis.

Comparing the seismic forces in Table 4-25 to the wind forces in Table 4-27, it is clear that seismic forces will govern the design of the members.

4.5.4 Design for Combined Load Effects

4.5.4.1 Load Combinations

The following load combinations are applicable:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)
4. $1.2D + f_1 L + 1.0E$ (Formula 16-5)

5. $0.9D + 1.0E$ (Formula 16-6)

where D, L, W, and E are the effects due to dead, live, wind, and seismic loads, respectively, and f_1 is equal to either 1.0 for places of public assembly, for live loads in excess of 100 psf, and for parking garage live load, or is equal to 0.5 for other live loads.

The seismic load effect E for use in Formula 16-5, which is the combined effect of horizontal and vertical earthquake-induced forces, is computed from Eq. 16-28 where the effects of gravity and seismic ground motion are additive:

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of horizontal seismic forces
ρ = redundancy coefficient determined in accordance with IBC 1617.2.2 for SDC D, E, or F

Similarly, E for use in Formula 16-6 is computed from Eq. 16-29 where the effects of gravity and seismic ground motion counteract:

$$E = \rho Q_E - 0.2 S_{DS} D$$

According to IBC 1617.2.2, the redundancy coefficient ρ, which shall not be less than 1.0 and need not exceed 1.5, is the largest of the values of ρ_i calculated at each story i from Equation 16-32:

$$\rho_i = 2 - \frac{20}{r_{\max_i} \sqrt{A_i}}$$

For moment frames:

$r_{\max_i}$ = maximum sum of shears in any 2 adjacent columns/total story shear

In the column shear summation, use 70% of shear in columns common to 2 adjacent bays, with moment-resisting connections on opposite sides.

Computations for the redundancy coefficient in the N-S direction for the moment-resisting frame along column line 3 (most heavily loaded frame when center of mass is displaced west of actual location) are summarized in Table 4-28. Subscripts on column shear forces refer to column lines. To illustrate the computations, $r_{\max_i}$ is calculated for the first story as follows:

$$r_{\max_1} = \frac{\text{maximum sum of shears in 2 adjacent columns}}{\text{total story shear}} = \frac{83}{1{,}012} = 0.082$$

Table 4-28 Redundancy Coefficient Calculations in N-S Direction (SDC D)*

Story	Story Shear	Column Shear Forces				$V_A + 0.7V_B$	$0.7(V_B + V_C)$	$0.7V_C + V_D$	r_{max_i}
		V_A	V_B	V_C	V_D				
3	448	13	28	28	13	33	39	33	0.087
2	824	27	49	49	27	61	69	61	0.084
1	1,012	35	59	59	35	76	83	76	0.082

*Shear forces are in kips.

From Table 4-28, maximum $r_{max_i} = 0.087$; thus, ρ is:

$$\rho = 2 - \frac{20}{0.087\sqrt{152.67 \times 92.67}} = 0.07 < 1.0$$

Therefore, $\rho = 1.0$. For SDC D, IBC 1617.2.2 requires that ρ be less than or equal to 1.25 for special moment frames, which is satisfied in this case.

Once ρ has been computed, load combinations 4 and 5 can be rewritten as follows. Substituting $S_{DS} = 1.00g$ and $\rho = 1.0$ into the equations for E, and then substituting E along with $f_1 = 0.5$ into load combinations 4 and 5 above results in the following:

4a. $1.2D + 0.5L + 1.0Q_E + (0.2 \times 1.00)D = 1.4D + 0.5L + Q_E$

4b. $1.2D + 0.5L + 1.0Q_E - (0.2 \times 1.00)D = 1.0D + 0.5L + Q_E$

5a. $0.9D + 1.0Q_E + (0.2 \times 1.00)D = 1.1D + Q_E$

5b. $0.9D + 1.0Q_E - (0.2 \times 1.00)D = 0.7D + Q_E$

Values of Q_E are obtained from the structural analysis for both sidesway to the left and to the right based on the code-prescribed horizontal seismic forces acting on the building. Also, the special seismic load combinations in IBC 1605.4 are not applicable in this example.

4.5.4.2 Design of Beam B3-C3

Deflection control.

As was shown in Section 4.2.4.2 of this publication, the 18.5-in. depth is adequate for deflection control (see ACI Table 9.5(a)).

Flexural design.

Table 4-29 contains a summary of the governing design bending moments and shear forces for beam B3-C3 at the first floor level due to gravity and seismic forces. The live load on the beam has been reduced in accordance with IBC 1607.9.2. Also, according to the 2002 supplement to the IBC [4.4], orthogonal effects need not be considered for beams.

Requirements for special moment frames are given in ACI 21.2 through 21.5. The factored axial load on the member, which is negligible, is less than $A_g f_c'/10$; thus, the provisions of ACI 21.3 for flexural members of special moment frames must be satisfied. All other applicable provisions in Chapters 1 through 18 are to be satisfied as well.

Check limitations on section dimensions per ACI 21.3.1:

- Factored axial compressive force on member is negligible. O.K.
- $\frac{\ell_n}{d} = \frac{(30 \times 12) - 36}{16} = 20.3 > 4$ O.K.
- $\frac{\text{width}}{\text{depth}} = \frac{30}{18.5} = 1.6 > 0.3$ O.K.
- width = 30 in. > 10 in. O.K.
  < width of supporting column + (1.5 × depth of beam)
  < 36 + (1.5 × 18.5) = 63.8 in. O.K.

Table 4-29 Summary of Design Bending Moments and Shear Forces for Beam B3-C3 at Floor Level 1 (SDC D)

Load Case	Location	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	Support	-278	57
	Midspan	191	
Live (L)	Support	-50	15
	Midspan	33	
Seismic (Q_E)	Support	± 123	± 9
Load Combinations			
$1.4D + 1.7L$	Support	-474	105
	Midspan	324	
$1.4D + 0.5L + Q_E$	Support	-537	96
	Midspan	284	
$0.7D + Q_E$	Support	-72	31
	Midspan	134	

The required flexural reinforcement is contained in Table 4-30. The provided areas of steel are within the limits prescribed in ACI 21.3.2.1 for maximum and minimum reinforcement. The selected reinforcement satisfies ACI 7.6.1 and 3.3.2 (minimum spacing for concrete placement), ACI 7.7.1 (minimum cover for protection of reinforcement), and ACI 10.6 (maximum spacing for control of flexural cracking).

Table 4-30 Required Flexural Reinforcement for Beam B3-C3 at Floor Level 1 (SDC D)

Location	M_u (ft-kips)	A_s^* (in.^{2})	Reinforcement*	ϕM_n (ft-kips)
Support	-537	8.93	9-No. 9	541
Midspan	324	4.94	5-No. 9	327

$$^*A_{s,min} = \frac{3\sqrt{f'_c}\,b_w d}{f_y} = \frac{3\sqrt{4{,}000} \times 30 \times 16}{60{,}000} = 1.52 \text{ in.}^2$$

ACI 21.3.2.1

$$= \frac{200 b_w d}{f_y} = \frac{200 \times 30 \times 16}{60{,}000} = 1.60 \text{ in.}^2 \text{ (governs)}$$

$$A_{s,max} = \rho_{max} b_w d = 0.025 \times 30 \times 16 = 12.00 \text{ in.}^2$$

ACI 21.3.2.1

ACI 21.3.2.2 requires that the positive moment strength at the face of the joint be greater than or equal to 50% of the negative moment strength at that location. This is satisfied, since 327 ft-kips > 541/2 = 271 ft-kips. Also, the negative or positive moment strength at any section along the length of the member must be greater than or equal to 25% of the maximum moment strength provided at the face of either joint. In this case, 25% of the maximum moment strength is equal to 541/4 = 135 ft-kips. Providing 2-No. 9 bars (ϕM_n = 138 ft-kips) satisfies this provision. This also automatically satisfies the requirement that at least 2 bars be continuous at both the top and bottom of the section (ACI 21.3.2.1).

When reinforcing bars extend through a joint, the column dimension parallel to the beam reinforcement must be at least 20 times the diameter of the largest longitudinal bar for normal weight concrete (ACI 21.5.1.4). In this case, the minimum required column dimension = 20 × 1.128 = 22.6 in., which is less than the 36-in. column width that is provided.

Shear design.

Shear requirements for beams in special moment frames are contained in ACI 21.3.4. The largest shear force associated with seismic effects is obtained from the second of the three load combinations in Table 4-29. Figure 4-21 shows the beam and shear forces due to gravity loads plus probable flexural strengths for sidesway to the right. Due to the symmetric distribution of longitudinal reinforcement in the beam, sidesway to the left gives the same maximum shear force. The factored uniform load on the beam is determined as follows:

$$w_D = [(74+45)\times 30/1{,}000] + \left(\frac{30\times 18.5}{144}\times 0.15\right) = 4.2 \text{ kips/ft}$$

$$w_L = 36 \times 30/1{,}000 = 1.1 \text{ kips/ft}$$

$w_u = 1.4w_D + 0.5w_L = 6.4$ kips/ft

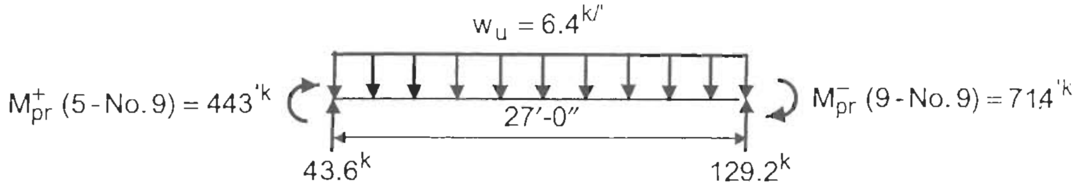


Figure 4-21 Design Shear Forces for Beam B3-C3 (SDC D)

The maximum combined shear force as shown in Figure 4-21 (129.2 kips) is larger than the maximum shear force obtained from the structural analysis (96 kips; see Table 4-29).

In general, shear strength is provided by both concrete (V_c) and reinforcing steel (V_s). However, according to ACI 21.3.4.2, V_c is to be taken as zero when the earthquake-induced shear force calculated in accordance with ACI 21.3.4.1 is greater than or equal to 50% of the total shear force and the factored axial compressive force including earthquake effects is less than $A_g f'_c / 20$. In this example, the beam carries negligible axial forces and the maximum earthquake-induced shear force = (714 + 443)/27 = 42.9 kips < 129.2/2 = 64.6 kips. Thus, the nominal shear strength provided by concrete V_c may be computed from ACI Eq. 11-3:

$$V_c = 2\sqrt{f'_c} b_w d = 2\sqrt{4,000} \times 30 \times 16 / 1,000 = 60.7 \text{ kips}$$

The maximum shear force V_s is (ACI 11.1):

$$V_s = \frac{V_u}{\phi} - V_c = \frac{129.2}{0.85} - 60.7 = 91.3 \text{ kips}$$

where the strength reduction factor was taken as 0.85 in accordance with ACI 9.3.4.

Shear strength contributed by shear reinforcement shall not exceed $(V_s)_{max}$ (ACI 11.5.6.9):

$$(V_s)_{max} = 8\sqrt{f'_c} b_w d = 8\sqrt{4,000} \times 30 \times 16 / 1,000 = 242.9 \text{ kips} > 91.3 \text{ kips} \quad \text{O.K.}$$

Also, V_s is less than $4\sqrt{f'_c} b_w d$ = 121.4 kips.

Required spacing s of No. 3 closed stirrups (hoops) for a factored shear force of 91.3 kips is determined from Eq. (11-15):

$$s = \frac{A_v f_y d}{V_s} = \frac{(5 \times 0.11) \times 60 \times 16}{91.3} = 5.8 \text{ in.}$$

Note that 5 legs are required for support of the longitudinal bars (ACI 21.3.3.3).

Maximum allowable hoop spacing within a distance of $2h = 2 \times 18.5 = 37$ in. from the face of the support at each end of the member is the smaller of the following (ACI 21.3.3.2):

- $d/4 = 16/4 = 4.0$ in. (governs)
- 8(diameter of smallest longitudinal bar) = $8 \times 1.128 = 9.0$ in.
- 24(diameter of hoop bar) = $24 \times 0.375 = 9.0$ in.
- 12 in.

Use 10-No. 3 hoops at each end of the beam spaced at 4 in. on center with the first hoop located 2 in. from the face of the support (ACI 21.3.3.2).

Where hoops are no longer required, stirrups with seismic hooks at both ends may be used (ACI 21.3.3.4). At a distance of 38 in. from the face of the support:

$$V_u = 129.2 - [6.4 \times (38/12)] = 108.9 \text{ kips}$$

Therefore, the required stirrup spacing for No. 3 stirrups (4 legs) is:

$$s = \frac{A_v f_y d}{V_s} = \frac{(4 \times 0.11) \times 60 \times 16}{(108.9/0.85) - 60.7} = 6.3 \text{ in.} \quad \text{(governs)}$$

$$= \frac{A_v f_y}{50 b_w} = \frac{(4 \times 0.11) \times 60{,}000}{50 \times 30} = 17.6 \text{ in.}$$

The maximum allowable spacing of the stirrups is $d/2 = 8$ in. (ACI 21.3.3.4), which is greater than 6.3 in. A 6 in. spacing, starting at 38 in. from the face of the support is sufficient for the remaining portion of the beam.

Reinforcing bar cutoff points.

The negative reinforcement at the supports is 9-No. 9 bars. The location where 7 of the 9 bars can be terminated will be determined.

The third load combination is used to determine the cutoff point of the 7-No. 9 bars (0.7 times the dead load in combination with the probable flexural strengths M_{pr} at the ends of the member), since this combination produces the longest bar lengths. The design flexural strength ϕM_n provided by 2-No. 9 bars is 138 ft-kips. Therefore, the 7-No. 9 bars can be terminated after the required moment strength M_u has been reduced to 138 ft-kips.

The distance x from the support to the location where the moment is equal to 138 ft-kips can readily be determined by summing moments about section a-a in Figure 4-22:

$$\frac{2.9x^2}{2} - 82x + 714 = 138$$

Solving for x gives a distance of 8.2 ft from the face of the support.

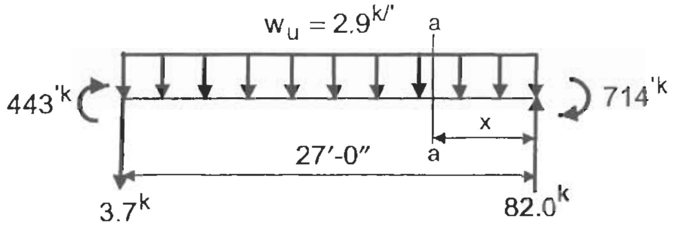


Figure 4-22 Cutoff Location of Negative Bars (SDC D)

The 7-No. 9 bars must extend a distance d = 16 in. (governs) or $12d_b = 12 \times 1.128 = 13.5$ in. beyond the distance x (ACI 12.10.3). Thus, from the face of the support, the total bar length must be at least equal to 8.2 + (16/12) = 9.5 ft.

Also, the bars must extend a full development length ℓ_d beyond the face of the support (ACI 12.10.4), which is determined from ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.3 for top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 9 bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5 + 0.375 + \dfrac{1.128}{2} = 2.4 \text{ in.} \\ \\ \dfrac{30 - 2(1.5 + 0.375) - 1.128}{2 \times 8} = 1.6 \text{ in. (governs)} \end{cases}$$

K_{tr} = transverse reinforcement index = 0 (conservative)

$$\frac{c + K_{tr}}{d_b} = \frac{1.6 + 0}{1.128} = 1.4 < 2.5$$

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.3 \times 1.0 \times 1.0 \times 1.0}{1.4} = 66.1$$

$$\ell_d = 66.1 \times 1.128 = 74.6 \text{ in.} = 6.2 \text{ ft} < 9.5 \text{ ft}$$

Thus, the total required length of the 7-No. 9 bars must be at least 9.5 ft beyond the face of the support.

Note that flexural reinforcement shall not be terminated in a tension zone unless one or more of the conditions of ACI 12.10.5 are satisfied. In this case, the point of inflection is approximately 10.8 ft from the face of the support, which is greater than 9.5 ft. Thus, the No. 9 bars cannot be terminated here unless one of the conditions of ACI 12.10.5 is satisfied. In this case, check if the factored shear force V_u at the cutoff point does not exceed $2\phi V_n/3$ (ACI 12.10.5.1). With No. 3 stirrups at 6 in. on center that are provided in this region of the beam, ϕV_n is determined by ACI Eqs. 11-1 and 11-2:

$$\phi V_n = \phi(V_c + V_s) = 0.85 \times (60.7 + 70.4) = 111.4 \text{ kips}$$

$$\frac{2}{3}\phi V_n = 74.3 \text{ kips}$$

At 9.5 ft from the face of the support, $V_u = 82 - (2.9 \times 9.5) = 54.5$ kips, which is less than 74.3 kips. Therefore, the 7-No. 9 bars can be terminated at 9.5 ft from the face of the support.

Flexural reinforcement splices.

According to ACI 21.3.2.3, lap splices of flexural reinforcement must not be placed within a joint, within a distance $2h$ from the face of the joint (plastic hinge region), or at locations where analysis indicates flexural yielding due to inelastic lateral displacements of the frame. Lap splices must be confined by hoops or spiral reinforcement along the entire lap length, and the maximum spacing of the transverse reinforcement is $d/4$ or 4 in. In lieu of lap splices, mechanical and welded splices conforming to ACI 21.2.6 and 21.2.7, respectively, may be used (ACI 21.3.2.4).

Lap splices are determined for the No. 9 bottom bars in the same way that is illustrated in Section 4.4.4.2 of this publication. Use a 3 ft-6 in. lap splice length.

Figure 4-23 shows the reinforcement details for beam B3-C3.

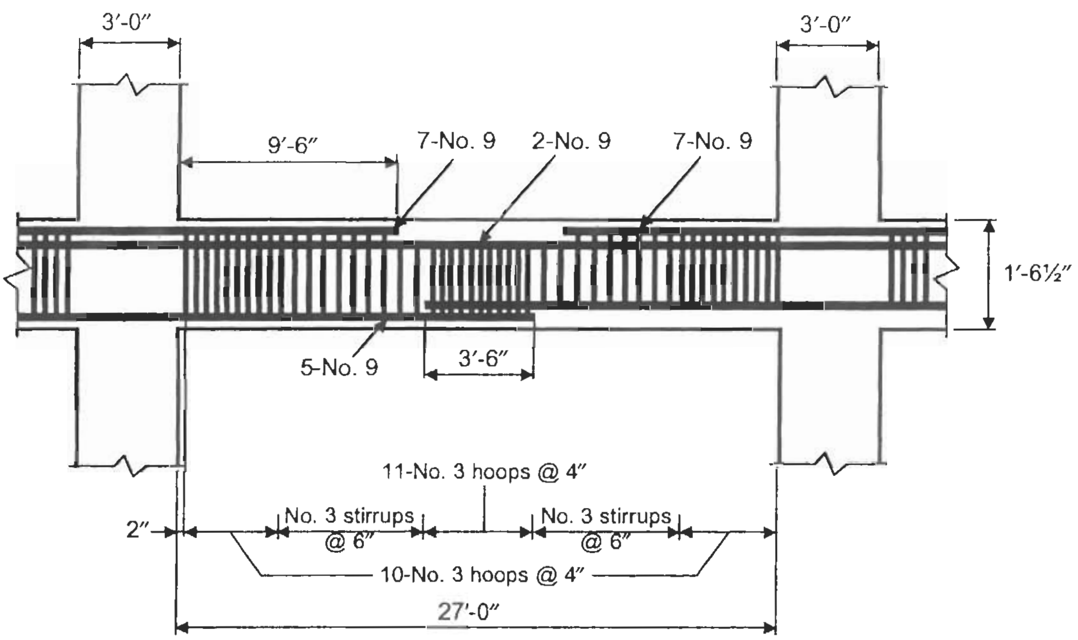


Figure 4-23 Reinforcement Details for Beam B3-C3 at Floor Level 1 (SDC D)

4.5.4.3 Design of Column C3

This section outlines the design of column C3 supporting the first floor level. Table 4-31 contains a summary of the design axial forces, bending moments, and shear forces on this column for gravity and seismic loads.

According to the supplement to the IBC [4.4], orthogonal effects on this member need not be considered, since the axial forces due to seismic forces in both directions (which are about equal to zero) are less than 20% of the axial load design strength.

Table 4-31 Summary of Design Axial Forces, Bending Moments, and Shear Forces on Column C3 between Ground and 1st Floor Level (SDC D)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (*D*)	392	0	0
Live (*L*)*	64	0	0
Seismic (Q_E)	0	± 876	± 59
Load Combinations			
1.4*D* + 1.7*L*	658	0	0
1.4*D* + 0.5*L* + Q_E	581	876	59
0.7*D* + Q_E	274	-876	-59

* Live load reduced per IBC 1607.9

Since the factored axial compressive force exceeds $A_g f'_c/10 = 36^2 \times 4/10 =$ 518 kips, the provisions of ACI 21.4 are applicable. Thus, the following two criteria must be satisfied (ACI 21.4.1):

- Shortest cross-sectional dimension = 36 in. > 12 in. O.K.
- Ratio of shortest cross-sectional dimension to perpendicular dimension = 1.0 > 0.4 O.K.

Design for axial force and bending.

Based on the governing load combinations in Table 4-31, a 36 × 36 in. column with 12-No. 10 bars ($\rho_g = 1.18\%$) is adequate for column C3 supporting the first floor level. Figure 4-24 contains the interaction diagram for this column. As noted above, slenderness effects need not be considered since P-delta effects were included in the analysis. Also, the provided reinforcement ratio is within the allowable range of 1% and 6% (ACI 21.4.3.1).

Relative Flexural Strength of Columns and Girders.

ACI 21.4.2 requires that the sum of flexural strengths of columns at a joint must be greater than or equal to 6/5 times the sum of flexural strengths of girders framing into that joint. The intent is to provide columns with sufficient strength so that they will not yield prior to the beams. Yielding at both ends of a column prior to the beams could result in total collapse of the structure. Only seismic load combinations need to be considered when checking the relative strengths of columns and girders.

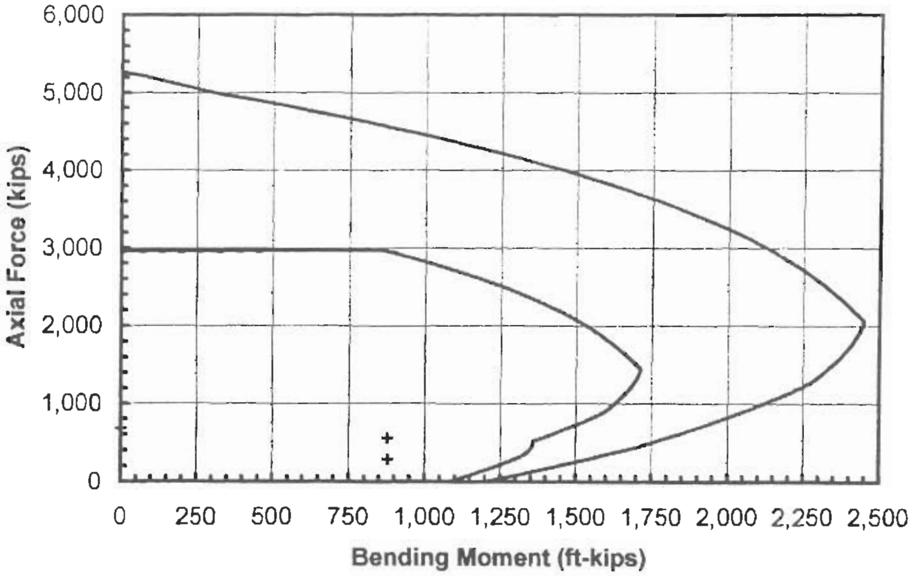


Figure 4-24 Design and Nominal Strength Interaction Diagrams for Column C3 Supporting Floor Level 1 (SDC D)

Based on the reinforcement in beam B3-C3 (see Table 4-30), $M_n^- = 541/0.9 = 601$ ft-kips and $M_n^+ = 327/0.9 = 363$ ft-kips. Therefore, $\sum M_g = 601 + 363 = 964$ ft-kips (see Figure 4-25 for sidesway to the left; due to symmetric distribution of flexural reinforcement, sidesway to the right yields the same results for the negative and positive moment strengths).

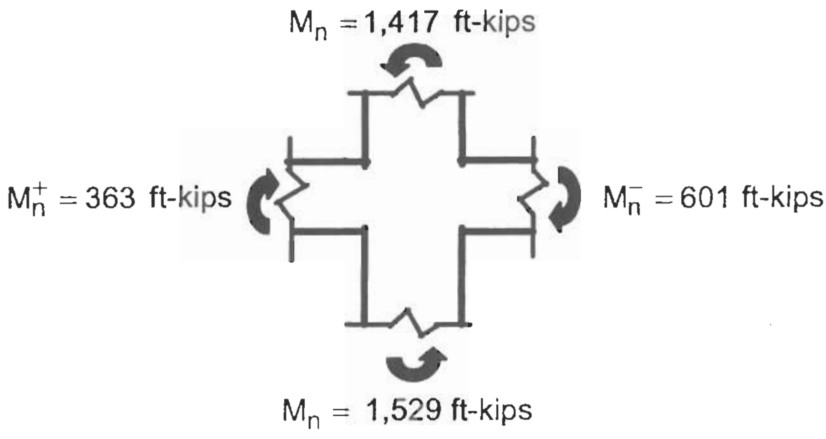


Figure 4-25 Relative Flexural Strength of Columns and Girders (SDC D)

Column flexural strength is determined for the factored axial force resulting in the lowest flexural strength, consistent with the direction of lateral forces considered. For the upper end of the lower column framing into the joint (i.e., the column supporting floor level 1), the minimum M_n = 1,529 ft-kips, which corresponds to P_u = 274 kips (see Figure 4-24). Similarly, for the lower end of the upper column framing into the joint (i.e., the column supporting floor level 2), M_n = 1,417 ft-kips, which corresponds to P_u = 176 kips. Therefore, $\sum M_c = 2{,}946$ ft-kips.

Check Eq. 21-1:

$$\sum M_c = 2{,}946 \text{ ft-kips} > \frac{6}{5}\sum M_g = \frac{6}{5}\times 964 = 1{,}157 \text{ ft-kips} \quad \text{O.K.}$$

Design for shear.

Shear requirements for columns in special moment frames are contained in ACI 21.4.5. Similar to beams, the method of determining design shear forces in columns takes into consideration the likelihood of yielding (i.e., plastic hinges forming) in regions near the ends of the column. To properly confine the concrete and to maintain lateral support of the longitudinal bars in regions where yielding is expected, the transverse reinforcement requirements of ACI 21.4.4 must also be satisfied.

Confinement reinforcement. Special transverse reinforcement for confinement is required over a distance ℓ_o from each joint face at both column ends where ℓ_o is equal to the largest of (ACI 21.4.4.4):

- Depth of member = 36 in. (governs)
- Clear span/6 = [(13 × 12) – 18.5]/6 = 22.9 in.
- 18 in.

Transverse reinforcement within the distance ℓ_o shall not be spaced greater than the smallest of (ACI 21.4.4.2):

- Minimum member dimension/4 = 36/4 = 9.0 in.
- 6(diameter of longitudinal reinforcement) = 6 × 1.27 = 7.6 in.
- $s_x = 4 + \left(\frac{14 - h_x}{3}\right) = 4 + \left(\frac{14 - 12}{3}\right) = 4.7$ in. (governs)

where h_x = maximum horizontal spacing of hoop or crosstie legs on all faces of the 36 × 36 in. column (ACI 21.4.4.3)

$$= \frac{36 - 2(1.5 + 0.5) - 1.27}{3} + 1.27 + 0.5 = 12.0 \text{ in.} < 14 \text{ in.} \quad \text{O.K.}$$

assuming No. 4 rectangular hoops with crossties around every longitudinal bar. Therefore, try 4 in. spacing.

Minimum required cross-sectional area of rectangular hoop reinforcement A_{sh} is the larger value obtained from Eqs. 21-3 and 21-4:

$$A_{sh} = \frac{0.3sh_c f_c'}{f_{yh}}\left[\left(\frac{A_g}{A_{ch}}\right)-1\right] = \frac{0.3\times4\times32.5\times4}{60}\left[\left(\frac{36^2}{1{,}089}\right)-1\right] = 0.49 \text{ in.}^2$$

$$= \frac{0.09sh_c f_c'}{f_{yh}} = \frac{0.09\times4\times32.5\times4}{60} = 0.78 \text{ in.}^2 \quad \text{(governs)}$$

where h_c = cross-sectional dimension of column core measured center-to-center of confinement reinforcement

= $36 - 2[1.5 + (0.5/2)] = 32.5$ in.

A_{ch} = cross-sectional area of member measured out-to-out of transverse reinforcement

= $[36 - (2 \times 1.5)]^2 = 1{,}089$ in.^{2}

Using No. 4 hoops with 2 crossties provides $A_{sh} = 4 \times 0.2 = 0.8$ in.^{2}, which is greater than the minimum required area from Eq. 21-4. Use 4 in. spacing for the transverse reinforcement at the column ends.

Transverse reinforcement for shear. According to ACI 21.4.5.1, shear forces are computed from statics assuming that moments of opposite sign act at the joint faces corresponding to the probable flexural strengths M_{pr} associated with the range of factored axial loads on the column. For a first story column, shear forces are computed based on the probable flexural strength of the column at the base and the probable flexural strengths of the beams at the top. Also, the design shear force must not be taken less than that determined from the structural analysis. Sidesway to the right and to the left must be considered when calculating the maximum design shear forces.

Figure 4-26 contains the design strength interaction diagram for column C3 with f_y = 75 ksi and $\phi = 1.0$. At the base of the column, the largest M_{pr} is equal to 2,036 ft-kips, which corresponds to an axial load equal to 581 kips (see Table 4-31).

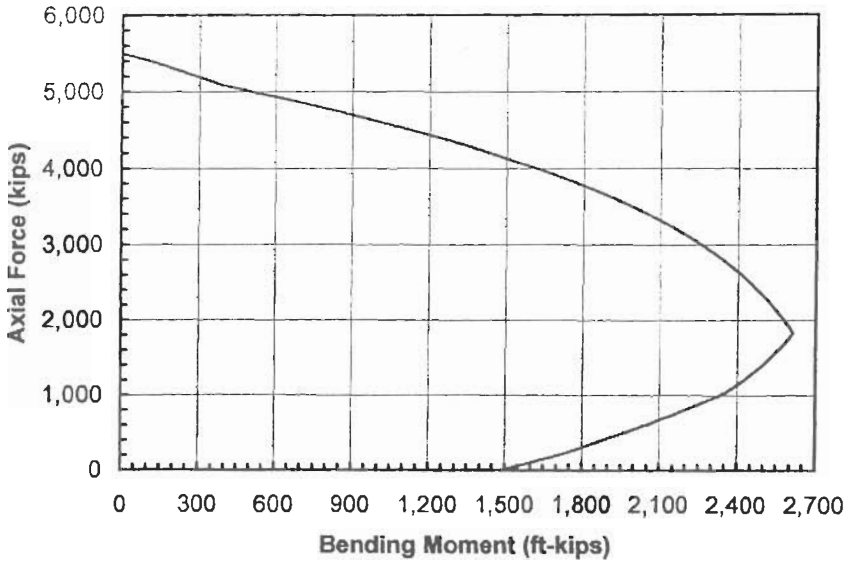


Figure 4-26 Design Strength Interaction Diagram for Column C3 with f_y = 75 ksi and ϕ = 1.0 (SDC D)

At the top of the column, the positive probable flexural strength of the beam framing into the joint at the face of the column is 443 ft-kips (see Figure 4-21). The negative probable flexural strength of the beam on the other side of the column is 714 ft-kips. Thus, the moment transferred to the top of the column is (443 + 714)/2 = 579 ft-kips, which is less than 2,036 ft-kips.

The maximum shear force V_e is equal to:

$$V_e = \frac{2{,}036 + 579}{13 - (18.5/12)} = 228 \text{ kips}$$

This design shear force is larger than the maximum shear force obtained from analysis, which is 59 kips (see Table 4-31).

Since the factored axial forces including earthquake effects are greater than $A_g f'_c / 20$ = 259 kips, the shear strength of the concrete may be used (ACI 21.4.5.2). The shear capacity of the column is checked using ACI Eq. 11-4 for members subjected to axial compression:

$$V_c = 2\left(1 + \frac{N_u}{2{,}000 A_g}\right)\sqrt{f'_c}\, b_w d$$

$$= 2\left(1 + \frac{274{,}000}{2{,}000 \times 36^2}\right)\sqrt{4{,}000} \times 36 \times 25.7 / 1{,}000 = 129.4 \text{ kips}$$

$$\phi V_c = 0.85 \times 129.4 = 110.0 \text{ kips} < V_u = 228 \text{ kips}$$

where N_u = 274 kips is the smallest axial force on the section (see Table 4-31) and d = 25.7 in. was obtained from a strain compatibility analysis.

Determine required spacing of transverse reinforcement from Eq. 11-15:

$$s = \frac{A_v f_y d}{\dfrac{V_u}{\phi} - V_c} = \frac{(4 \times 0.2) \times 60 \times 25.7}{\dfrac{228}{0.85} - 129.4} = 8.9 \text{ in.}$$

Thus, the No. 4 hoops at 4 in. on center required for confinement over the distance ℓ_o at the column ends is also adequate for shear.

According to ACI 21.4.4.6, the remainder of the column must contain hoop reinforcement with center-to center spacing not to exceed 6 times the diameter of the longitudinal column bars = 6 × 1.27 = 7.6 in. or 6 in. (governs). For detailing simplicity, use a 4 in. spacing over the entire length of the column.

Splice length of longitudinal reinforcement.

Lap splices in columns of special moment frames are permitted only within the center half of the member and must be designed as tension lap splices (ACI 21.4.3.2). Also, they must be confined over the entire lap length with transverse reinforcement conforming to ACI 21.4.4.2 and 21.4.4.3. In lieu of lap splices, mechanical splices conforming to ACI 21.2.6 and welded splices conforming to ACI 21.2.7.1 may be utilized.

Since all of the bars are to be spliced at the same location, a Class B splice is required (ACI 12.15.1, 12.15.2).

From ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 7 and larger bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 1.5 + 0.5 + \dfrac{1.27}{2} = 2.6 \text{ in.} \quad \text{(governs)} \\ \\ \dfrac{36 - 2(1.5 + 0.5) - 1.27}{3 \times 2} = 5.1 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index

$$= \frac{A_{tr} f_{yt}}{1{,}500 sn} = \frac{4 \times 0.20 \times 60{,}000}{1{,}500 \times 4 \times 4} = 2.0$$

$$\frac{c + K_{tr}}{d_b} = \frac{2.6 + 2.0}{1.27} = 3.6 > 2.5, \text{ use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.0 \times 1.0 \times 1.0 \times 1.0}{2.5} = 28.5$$

$$\ell_d = 28.5 \times 1.27 = 36.2 \text{ in.} = 3.0 \text{ ft}$$

Class B splice length = $1.3\ell_d$ = 3.9 ft

Use a 4 ft-0 in. splice length.

Reinforcement details for column C3 are shown in Figure 4-27.

4.5.4.4 Design of Beam-Column Joint

Interior Joint.

Transverse reinforcement for confinement. ACI 21.5.2 requires that transverse hoop reinforcement in accordance with ACI 21.4.4 be provided within the joint, unless the joint is confined on all four sides by beams that have widths equal to at least 75% the column width. When joints are adequately confined on four sides, transverse reinforcement within the joint may be reduced to one-half of that required by ACI 21.4.4 and the hoop spacing is permitted to be a maximum of 6 in.

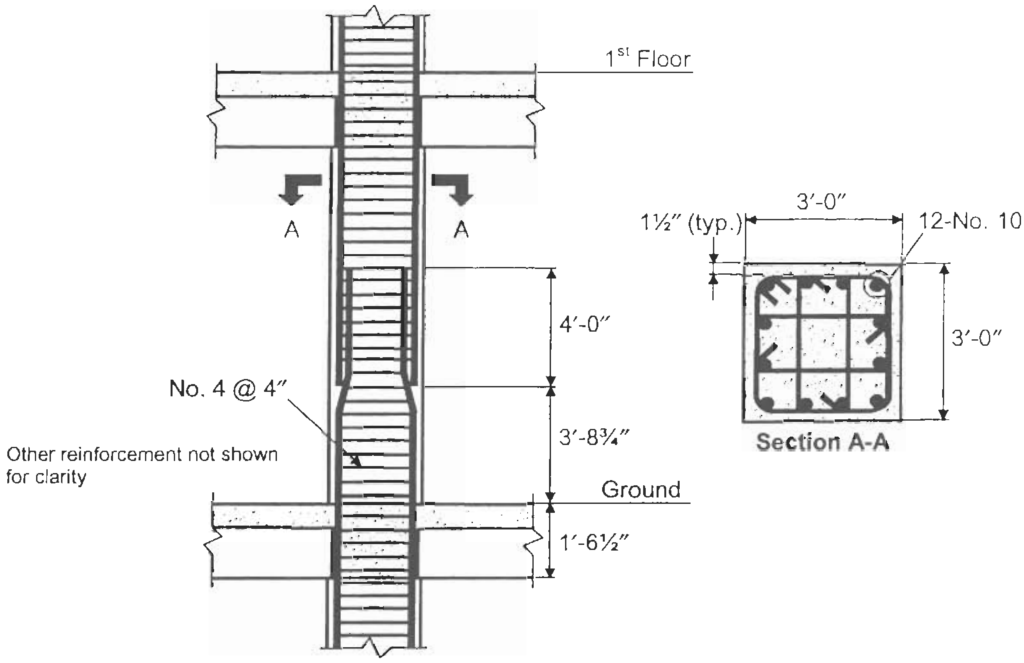


Figure 4-27 Reinforcement Details for Column C3 Supporting the 1st Floor Level (SDC D)

At an interior joint in the example building, beams frame into two sides of the column (joists do not provide confinement in perpendicular direction). According to ACI 21.5.3, these beams provide confinement to the joint, since the beam width (30 in.) is greater than 75 percent of the column face (27 in.). Therefore, transverse reinforcement must be provided in accordance with ACI 21.4.4 as noted above. Use the transverse reinforcement required at the column ends (No. 4 @ 4 in.) through the joint.

Shear strength of joint. Figure 4-28 shows the interior beam-column joint at the first floor level. The shear strength is checked in the N-S direction in accordance with ACI 21.5.3. The shear force at section *x-x* is obtained by subtracting the column shear force from the sum of the tensile force in the top beam reinforcement and the compressive force at the top of the beam on the opposite face of the column. Since development of inelastic rotations at the joint face is associated with strains in the beam flexural reinforcement significantly greater than the yield strain, joint shear forces generated by beam reinforcement are calculated based on a stress in the reinforcement equal to 1.25 f_y (ACI 21.5.1.1):

$$T_1 \text{ (9 - No. 9)} = A_s(1.25 f_y) = (9 \times 1.00) \times (1.25 \times 60) = 675 \text{ kips}$$

$$T_2 \text{ (5 - No. 9)} = (5 \times 1.00) \times (1.25 \times 60) = 375 \text{ kips}$$

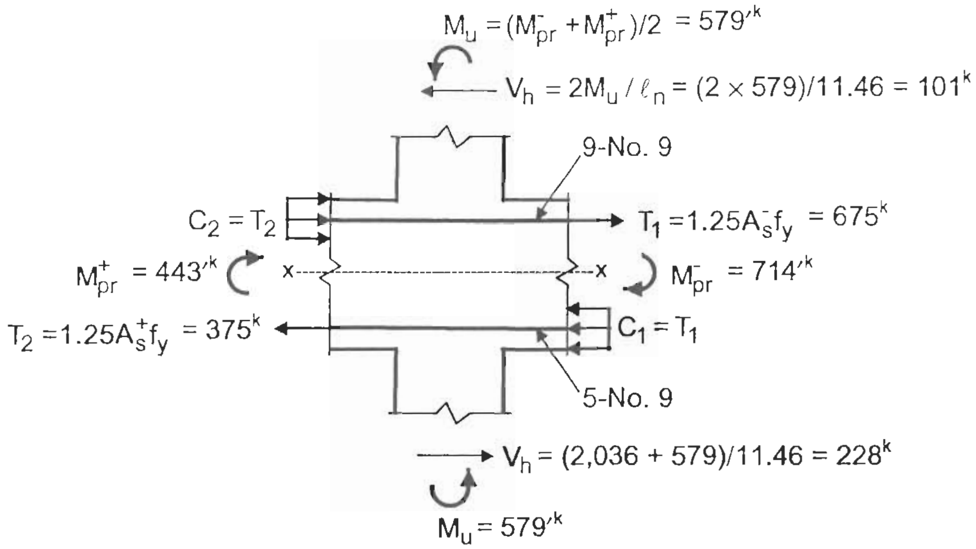


Figure 4-28 Shear Analysis of Interior Beam-Column Joint in N-S Direction (SDC D)

Column horizontal shear force V_h can be obtained by assuming that adjoining floors are deformed so that plastic hinges form at the ends of the beams. For the beams in this example, $M_{pr}^- = 714$ ft-kips and $M_{pr}^+ = 443$ ft-kips (see Figure 4-21).

Since the lengths of the columns above and below the joint are equal, moments M_u in the columns above and below the joint are (714 + 443)/2 = 579 ft-kips. Shear force V_h in the column at the top of the joint is:

$$V_h = \frac{2M_u}{\ell_n} = \frac{2 \times 579}{13 - (18.5/12)} = 101 \text{ kips}$$

Shear force V_h in the column at the bottom of the joint is (see Section 4.5.4.3 of this publication for calculation of moments in column):

$$V_h = \frac{M_u}{\ell_n} = \frac{2{,}036 + 579}{13 - (18.5/12)} = 228 \text{ kips}$$

The maximum net shear force at section *x-x* is $T_1 + C_2 - V_h = 675 + 375 - 101 = 949$ kips

For a joint confined on two opposite faces, nominal shear strength ϕV_c is determined from ACI 21.5.3.1:

$$\phi V_c = \phi 15\sqrt{f_c'}A_j = 0.85 \times 15\sqrt{4{,}000} \times 1{,}296/1{,}000 = 1{,}045 \text{ kips} > 949 \text{ kips} \quad \text{O.K.}$$

where $A_j = 36 \times 36 = 1{,}296 \text{ in.}^2$

Reinforcement details at an interior joint in this example are very similar to those shown in Figure 4-18.

Exterior Joint.

It is assumed for purposes of this example that the beam framing into the edge column along column line 3 has the same flexural reinforcement as the interior beams along this column line.

Transverse reinforcement for confinement. Since an exterior joint is confined on less than four sides, transverse hoop reinforcement in accordance with ACI 21.4.4 must be provided within the joint (ACI 21.5.2.1). For detailing simplicity, use the transverse reinforcement required at the column ends through the joint.

Shear strength of joint. Figure 4-29 shows the exterior joint at the 1st floor level. In this case, the shear force at section *x-x* is determined by subtracting the column shear force from the tensile force in the top beam reinforcement.

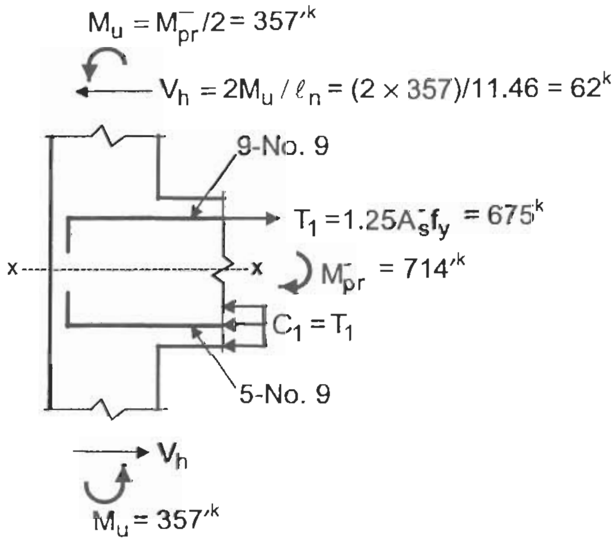


Figure 4-29 Shear Analysis of Exterior Beam-Column Joint in N-S Direction (SDC D)

Since the lengths of the columns above and below the joint are equal, moments M_u in the columns above and below the joint are 714/2 = 357 ft-kips. Shear force V_h in the column at the op of the joint is:

$$V_h = \frac{2M_u}{\ell_n} = \frac{2 \times 357}{13 - (18.5/12)} = 62 \text{ kips}$$

The net shear force at section *x-x* is $T_1 - V_h = 675 - 62 = 613$ kips

For a joint confined on one face, the nominal shear strength ϕV_c is determined from ACI 21.5.3.1:

$$\phi V_c = \phi 12 \sqrt{f'_c} A_j = 0.85 \times 12 \sqrt{4{,}000} \times 32^2 / 1{,}000 = 661 \text{ kips} > 613 \text{ kips} \quad \text{O.K.}$$

Beam flexural reinforcement terminated in a column must extend to the far face of the confined column core and must be anchored in tension and compression according to ACI 21.5.4 and Chapter 12, respectively (ACI 21.5.1.3). The development length ℓ_{dh} for a bar with a standard 90-degree hook in normal weight concrete is the largest of (ACI 21.5.4.1):

- 8(diameter of longitudinal bar) = 8 × 1.128 = 9.0 in.
- 6 in.
- $\dfrac{f_y d_b}{65\sqrt{f'_c}} = \dfrac{60{,}000 \times 1.128}{65\sqrt{4{,}000}} = 16.5$ in. (governs)

Required development lengths for both the flexural reinforcement can be accommodated within the 32-in. deep column.

Reinforcement details at the exterior joint in this example are similar to those shown in Figure 4-20.

4.6 REFERENCES

4.1 International Conference of Building Officials, *Code Central – Earthquake Spectral Acceleration Maps*, prepared in conjunction with U.S. Geological Survey; Building Seismic Safety Council; Federal Emergency Management Agency; and E.V. Leyendecker, A.D. Frankel, and K.S. Rukstales, Whittier, CA (CD-ROM).

4.2 Computers and Structures, Inc., *SAP2000 – Integrated Finite Element Analysis and Design of Structures,* Berkeley, CA, 1999.

4.3 American Society of Civil Engineers, *ASCE Standard Minimum Design Loads for Buildings and Other Structures*, ASCE 7-98, Reston, VA, 2000.

4.4 International Code Council, *2002 Accumulative Supplement to the International Codes*, Falls Chuch, VA, 2002.

CHAPTER 5

RESIDENTIAL BUILDING WITH BEARING WALL SYSTEM

5.1 INTRODUCTION

A bearing wall system is a structural system without an essentially complete space frame providing support for gravity loads. Bearing walls provide support for all or most of the gravity loads. Resistance to lateral loads is provided by the same bearing walls acting as shear walls.

According to IBC Table 1617.6, ordinary reinforced concrete shear walls can be used in buildings with bearing walls systems assigned to SDC A, B, or C (IBC 1910.3, 1910.4). In such cases, the walls can be designed according to ACI 318, exclusive of Chapter 21, with no special seismic detailing required. Special reinforced concrete shear walls are required for buildings assigned to SDC D and higher, subject to the limitations in Table 1617.6 (IBC 1910.5). Design and detailing of the shear walls must conform to ACI 21.6 as well as all of the other applicable sections in ACI 318.

This chapter illustrates the design and detailing of a bearing wall system in a structure assigned to SDC A, B, C, D, and E.

5.2 DESIGN FOR SDC A

5.2.1 Design Data

A typical plan and elevation of a 7-story school building is shown in Figure 5-1. The computation of wind and seismic forces according to the 2000 IBC is illustrated below. Typical walls are designed and detailed for combined effects of gravity, wind, and seismic forces.

As noted above, ordinary reinforced concrete shear walls may be used for structures assigned to SDC A without any limitations according to IBC Table 1617.6. This type of system is utilized in this example.

It is assumed mainly for simplicity that walls have constant cross-sections throughout the height of the building, and that the bases of the lowest story segments are fixed. Although the wall dimensions in the following sections are within the practical range, the structure itself is a hypothetical one, and has been chosen mainly for illustrative purposes.

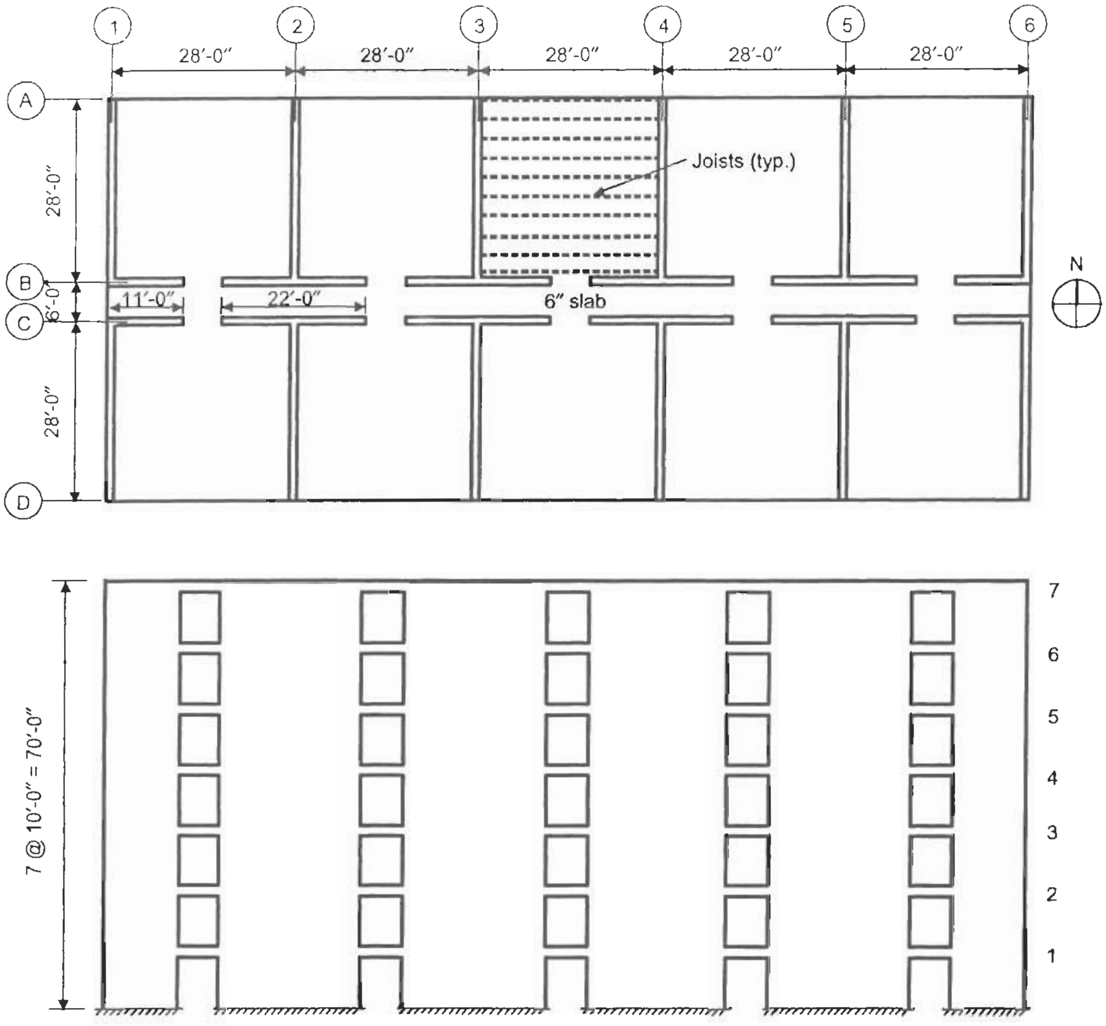


Figure 5-1 Typical Plan and Elevation of Example Building

- Building Location: Miami, FL (zip code 33122)
- Material Properties

Concrete: $f'_c = 4{,}000$ psi, $w_c = 150$ pcf
Reinforcement: $f_y = 60{,}000$ psi

- Service Loads

Live loads: roof = 20 psf
floor = 100 psf for corridors, 40 psf elsewhere

Superimposed dead loads: roof = 10 psf
floor = 30 psf (20 psf permanent partitions + 10 psf ceiling, etc.)

- Seismic Design Data

For zip code 33122: $S_S = 0.065g, S_1 = 0.024g$ [5.1]
Site Class D (stiff soil profile; IBC Table 1615.1.1)
For Category (Seismic Use Group) I occupancy, $I_E = 1.0$ (IBC Table 1604.5)

- Wind Design Data

Basic wind speed = 145 mph (IBC Figure 1609)
Exposure B (IBC 1609.4)
For Category I occupancy, $I_W = 1.0$ (IBC Table 1604.5)

- Member Dimensions

Joists: 12 + 4.5 × 5 + 30 (82 psf)
Wall thickness: 6 in.

5.2.2 Seismic Load Analysis

Exception 4 of IBC 1614.1 states that structures located where $S_S \leq 0.15g$ and $S_1 \leq 0.04g$ need only comply with IBC 1616.4, which are design requirements for SDC A. Thus, the minimum lateral force F_x applied simultaneously at each floor level is computed according to IBC Eq. 16-27:

$$F_x = 0.01 w_x$$

where w_x is the portion of the total gravity load W located or assigned to level x and W is defined in IBC 1616.4.1. The seismic forces F_x are summarized in Table 5-1.

Note that the seismic forces are the same in both the N-S and E-W directions, and may be applied separately in each of the two orthogonal directions of the building, i.e., orthogonal effects may be neglected (IBC 1616.4.1).

5.2.3 Wind Load Analysis

According to IBC 1609.1.1, wind loads shall be determined in accordance with Section 6 of ASCE 7 [5.2]. Since the building has a mean roof height greater than 30 ft, the simplified procedure (Method 1) given in ASCE 6.4 cannot be used to determine the wind forces. Similarly, the simplified procedure of IBC 1609.6 must not be used, since the building is taller than 60 ft.

Table 5-1 Seismic Forces for SDC A

Level	Story weight, w_x (kips)	Lateral force, $F_x = 0.01 w_x$ (kips)
7	978	9.8
6	1,360	13.6
5	1,360	13.6
4	1,360	13.6
3	1,360	13.6
2	1,360	13.6
1	1,360	13.6
	Σ	91.4

The example building is regular shaped by the definition in ASCE 6.2, i.e., it has no unusual geometrical irregularity in spatial form. Also, the building does not have response characteristics making it subject to across wind loading, vortex shedding, or instability due to galloping or flutter. It is assumed that the site location is such that channeling effects or buffeting in the wake of upwind obstructions need not be considered. Thus, the analytical procedure (Method 2) of ASCE 6.5 may be used to determine the wind forces.

Design Procedure.

The design procedure outlined in Section 6.5.3 of ASCE 7 is used to determine the wind forces on the building in the N-S and E-W directions.

1. Basic wind speed, *V*, and wind directionality factor, K_d. Both quantities are determined in accordance with ASCE 6.5.4. As noted above, *V* is equal to 145 mph for Miami according to IBC Figure 1609 or ASCE Figure 6-1.

The wind directionality factor K_d is equal to 0.85 for main wind-force-resisting systems per ASCE Table 6-6 when load combinations specified in Sections 2.3 and 2.4 are used. Note that these load combinations are essentially the same as those in Sections 1605.2 and 1605.3 of the 2000 IBC. It is important to note exception 1 to IBC 1605.2.1, Basic load combinations: load combinations of ACI 9.2 shall be used for concrete structures where combinations do not include seismic forces. The load factors in the ACI 318 combinations are different than those in ASCE 7 and the IBC. The exception goes on to state that for concrete structures designed for wind in accordance with ASCE 7, wind forces are to be divided by the directionality factor. Thus, in the following computations, instead of multiplying and then subsequently dividing the external wind pressures/forces by 0.85, K_d is taken equal to 1.0.

2. Importance factor, I_W. As noted above, I_W is equal to 1.0 for Category I occupancy according to IBC Table 1604.5 and Category II occupancy according to ASCE Table 1-1 (note that IBC Category I and ASCE 7 Category II are the same).

3. Velocity pressure exposure coefficient, K_z. According to ASCE 6.5.6.4, values of K_z are to be determined from ASCE Table 6-5. In lieu of linear interpolation, K_z may be calculated at any height z above ground level from the equations given at the bottom of Table 6-5:

$$K_z = \begin{cases} 2.01\left(\dfrac{15}{z_g}\right)^{2/\alpha} & \text{for } z < 15 \text{ ft} \\ 2.01\left(\dfrac{z}{z_g}\right)^{2/\alpha} & \text{for } 15 \text{ ft} \le z \le z_g \end{cases}$$

where α = 3-second gust speed power law exponent from ASCE Table 6-4
= 7.0 for Exposure B
z_g = nominal height of the atmospheric boundary layer from ASCE Table 6-4
= 1,200 ft for Exposure B

Values of K_z are summarized in Table 5-2 at the various story heights for the example building.

Table 5-2 Velocity Pressure Exposure Coefficient K_z

Level	Height above ground level, z (ft)	K_z
7	70	0.893
6	60	0.854
5	50	0.811
4	40	0.761
3	30	0.701
2	20	0.624
1	10	0.575

4. Topographic factor, K_{zt}. The topographic factor is to be determined in accordance with ASCE 6.5.7, Eq. 6-1. Assuming the example building is situated on level ground and not on a hill, ridge, or escarpment, K_{zt} is equal to 1.

5. Gust effect factors, G and G_f. Effects due to wind gust depend on whether a building is rigid or flexible (ASCE 6.5.8). A rigid building has a fundamental natural frequency n_1 greater than or equal to 1 Hz, while a flexible building has a fundamental natural frequency less than 1 Hz (ASCE 6.2).

In lieu of a more exact method, an approximate fundamental period T_a is determined using Eq. 16-39 in IBC 1617.4.2.1. The natural frequency is computed by taking the inverse of the period.

$$T_a = C_T (h_n)^{3/4}$$

where C_T = building period coefficient
= 0.030 for moment-resisting frame systems of concrete
= 0.020 for other types of building systems

Thus,

$$T_a = 0.020 \times (70)^{3/4} = 0.48 \text{ sec} \quad \text{or} \quad n_1 = 1/0.48 = 2.1 \text{ Hz}$$

Since n_1 > 1.0 Hz, the building is considered rigid, and G may be taken equal to 0.85 or may be calculated by Eq. 6-2 (ASCE 6.5.8.1). For simplicity, G is taken as 0.85.

6. Enclosure classification. It is assumed in this example that the building is enclosed per ASCE 6.5.9 or IBC 1609.2.

7. Internal pressure coefficient, GC_{pi}. According to ASCE 6.5.11.1, internal pressure coefficients are to be determined from ASCE Table 6-7 based on building enclosure classification. The building is in a hurricane-prone region. Glazing in the bottom 60 ft of the building is assumed to be debris resistant (ASCE 6.5.9.3). Therefore, for an enclosed building, $GC_{pi} = \pm 0.18$.

8. External pressure coefficients, C_p. External pressure coefficients for main wind-force-resisting systems are given in Figure 6-3 for this example building. For wind in the N-S direction:

Windward wall: $C_p = 0.8$

Leeward wall (L/B = 62/140.5 = 0.44): $C_p = -0.5$

Side wall: $C_p = -0.7$

Roof (h/L = 70/62 = 1.13):

$C_p = -1.3$ from 0 to $h/2$ = 35 ft from windward edge (may be reduced to 0.8 × (-1.3) = -1.04 over applicable area)

$C_p = -0.7$ from 35 ft from windward edge to 62 ft

For wind in the E-W direction:

Windward wall: $C_p = 0.8$

Leeward wall (L/B = 140.5/62 = 2.27): $C_p = -0.29$

Side wall: $C_p = -0.7$

Roof (h/L = 70/140.5 = 0.50):

$C_p = -0.9$ from 0 to h = 70 ft from windward edge

$C_p = -0.5$ from 70 ft from windward edge to 140.5 ft

9. Velocity pressure, q_z. The velocity pressure at height z is determined from Eq. 6-13 in ASCE 6.5.10:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I$$

where all terms have been defined previously. Table 5-3 contains a summary of the velocity pressures for the example building.

Table 5-3 Velocity Pressure q_z (V = 145 mph)

Level	Height above ground level, z (ft)	K_z	q_z (psf)
7	70	0.893	48.1
6	60	0.854	46.0
5	50	0.811	43.7
4	40	0.761	41.0
3	30	0.701	37.7
2	20	0.624	33.6
1	10	0.575	31.0

10. Design wind pressure, *p*. Design wind pressures on the main wind-force-resisting systems of enclosed and partially enclosed buildings are determined in accordance with ASCE 6.5.12. For rigid buildings of all heights, design wind pressures are calculated from Eq. 6-15:

$$p = qGC_p - q_i(GC_{pi})$$

Tables 5-4 and 5-5 contain summaries of design pressures and forces, respectively, for wind in the N-S direction. It has been assumed that the design wind pressure is constant over the tributary height of the floor level. Tables 5-6 and 5-7 contain the pressures and forces for wind in the E-W direction, respectively.

Table 5-4 Design Wind Pressures in N-S Direction (V = 145 mph)

Location	Level	Height above ground level, *z* (ft)	External Pressure				Internal Pressure		
			q (psf)	G	C_p	qGC_p (psf)	q_i (psf)	GC_{pi}	q_iGC_{pi} (psf)
Windward	7	70	48.1	0.85	0.80	32.7	48.1	± 0.18	± 8.7
	6	60	46.0	0.85	0.80	31.3	48.1	± 0.18	± 8.7
	5	50	43.7	0.85	0.80	29.7	48.1	± 0.18	± 8.7
	4	40	41.0	0.85	0.80	27.9	48.1	± 0.18	± 8.7
	3	30	37.7	0.85	0.80	25.6	48.1	± 0.18	± 8.7
	2	20	33.6	0.85	0.80	22.9	48.1	± 0.18	± 8.7
	1	10	31.0	0.85	0.80	21.1	48.1	± 0.18	± 8.7
Leeward	---	All	48.1	0.85	-0.50	-20.4	48.1	± 0.18	± 8.7
Side	---	All	48.1	0.85	-0.70	-28.6	48.1	± 0.18	± 8.7
Roof	---	70*	48.1	0.85	-1.30	-53.2	48.1	± 0.18	± 8.7
	---	70†	48.1	0.85	-0.70	-28.6	48.1	± 0.18	± 8.7

* from windward edge to 35 ft
† from 35 ft to 62 ft

Table 5-5 Design Wind Forces in N-S Direction (V = 145 mph)

Level	Height above ground level, *z* (ft)	Tributary Height (ft)	Windward		Leeward		Total Design Wind Force (kips)
			External Design Wind Pressure, q_zGC_p (psf)	Design Wind Force, *P* * (kips)	External Design Wind Pressure, q_hGC_p (psf)	Design Wind Force, *P* * (kips)	
7	70	5.0	32.7	23.0	-20.4	14.3	37.3
6	60	10.0	31.3	44.0	-20.4	28.7	72.7
5	50	10.0	29.7	41.7	-20.4	28.7	70.4
4	40	10.0	27.9	39.2	-20.4	28.7	67.9
3	30	10.0	25.6	36.0	-20.4	28.7	64.7
2	20	10.0	22.9	32.2	-20.4	28.7	60.9
1	10	10.0	21.1	29.7	-20.4	28.7	58.4
						Σ	432.3

$^*P = qGC_p \times$ Tributary height $\times 140.5$

5.2.4 Design for Combined Load Effects

It is evident from Tables 5-1, 5-5, and 5-7 that wind forces will govern the design of the shear walls for SDC A. It is important to note, however, that IBC 1609.1.5 requires that seismic detailing requirements of the code be satisfied, even when wind load effects are greater than seismic load effects.

Table 5-6 Design Wind Pressures in E-W Direction (V = 145 mph)

Location	Level	Height above ground level, z (ft)	External Pressure: q (psf)	G	C_p	qGC_p (psf)	Internal Pressure: q_i (psf)	GC_{pi}	q_iGC_{pi} (psf)
Windward	7	70	48.1	0.85	0.80	32.7	48.1	± 0.18	± 8.7
	6	60	46.0	0.85	0.80	31.3	48.1	± 0.18	± 8.7
	5	50	43.7	0.85	0.80	29.7	48.1	± 0.18	± 8.7
	4	40	41.0	0.85	0.80	27.9	48.1	± 0.18	± 8.7
	3	30	37.7	0.85	0.80	25.6	48.1	± 0.18	± 8.7
	2	20	33.6	0.85	0.80	22.9	48.1	± 0.18	± 8.7
	1	10	31.0	0.85	0.80	21.1	48.1	± 0.18	± 8.7
Leeward	---	All	48.1	0.85	-0.29	-11.9	48.1	± 0.18	± 8.7
Side	---	All	48.1	0.85	-0.70	-28.6	48.1	± 0.18	± 8.7
Roof	---	70*	48.1	0.85	-0.90	-36.8	48.1	± 0.18	± 8.7
	---	70†	48.1	0.85	-0.50	-20.4	48.1	± 0.18	± 8.7

* from windward edge to 70 ft
† from 70 ft to 140.5 ft

Table 5-7 Design Wind Forces in E-W Direction (V = 145 mph)

Level	Height above ground level, z (ft)	Tributary Height (ft)	Windward: External Design Wind Pressure, q_zGC_p (psf)	Windward: Design Wind Force, P* (kips)	Leeward: External Design Wind Pressure, q_hGC_p (psf)	Leeward: Design Wind Force, P* (kips)	Total Design Wind Force (kips)
7	70	5.0	32.7	10.1	-11.9	3.7	13.8
6	60	10.0	31.3	19.4	-11.9	7.4	26.8
5	50	10.0	29.7	18.4	-11.9	7.4	25.8
4	40	10.0	27.9	17.3	-11.9	7.4	24.7
3	30	10.0	25.6	15.9	-11.9	7.4	23.3
2	20	10.0	22.9	14.2	-11.9	7.4	21.6
1	10	10.0	21.1	13.1	-11.9	7.4	20.5
						Σ	156.5

*$P = qGC_p$ × Tributary height × 62

A three-dimensional analysis of the building was performed in the N-S and E-W directions for the wind loads contained in Tables 5-5 and 5-7 using SAP2000 [5.3]. In the model, rigid diaphragms were assigned at each floor level. The stiffness properties of the walls were input assuming cracked sections. In lieu of a more accurate analysis, the cracked section property of the shear walls was taken as $I_{eff} = 0.5I_g$ where I_g is the gross moment of inertia of the section.

According to ASCE 6.5.12.3, the main wind-force-resisting systems of buildings with mean roof height h greater than 60 ft must be designed for the full and partial wind load cases of Figure 6-9 (Cases 1 through 4). These four cases were considered in the three-dimensional analysis.

5.2.4.1 Load Combinations

Basic load combinations for strength design are given in IBC 1605.2.1. As noted above, the first exception in this section requires that the non-seismic load combinations of ACI 9.2 be used for concrete structures. Thus, the following load combinations are applicable:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)

where D, L, and W are the effects due to dead, live, and wind loads, respectively. It is important to reiterate that the wind loads were computed utilizing a directionality factor equal to one (IBC 1605.2.1, exception 1).

5.2.4.2 Design of Shear Wall on Line 5

Table 5-8 contains a summary of the design axial forces, bending moments, and shear forces at the base of the stem segment of the wall. Note that for wind load cases 1 and 2, wind in the N-S direction will causes appreciable reactions in the stem. Similarly, Table 5-9 contains a summary of the reactions at the base of the flange segment of the wall. In this case, wind in the E-W direction causes appreciable reactions in the flange.

Wall Stem Segment.

Design for shear. The shear strength of the concrete is determined in accordance with ACI 11.10.5 for walls subjected to axial compression:

$$V_c = 2\sqrt{f'_c}hd = 2\sqrt{4{,}000} \times 6 \times 271.2/1{,}000 = 205.8 \text{ kips}$$

where d is permitted to be taken equal to $0.8\ell_w = 0.8 \times 339 = 271.2$ in. (ACI 11.10.4).

The maximum factored shear force is 40 kips from wind load case 1 in the second and third load combinations (see Table 5-8). Since $V_u = 40$ kips $< \phi V_c/2 = 0.85 \times 205.8/2 =$ 87.5 kips, reinforcement shall be provided in accordance with ACI 11.10.9 or Chapter 14 (ACI 11.10.8).

Table 5-8 Summary of Design Axial Forces, Bending Moments, and Shear Forces in Stem Segment at Base of Shear Wall on Line 5 (SDC A)

Load Case	Full and Partial Wind Load Cases*	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (*D*)		739	0	0
Live (*L*)		95	0	0
Wind (*W*)	1	± 42	± 652	± 31
	2	± 41	± 626	± 30
	3	± 33	± 490	± 24
	4	± 32	± 476	± 23
Load Combinations				
1.4*D* + 1.7*L*		1,196	0	0
0.75(1.4*D* + 1.7*L* + 1.7*W*)	1	951	831	40
	2	949	798	38
	3	939	625	31
	4	938	607	29
0.9*D* + 1.3*W*	1	611	-848	-40
	2	612	-814	-39
	3	622	-637	-31
	4	624	-619	-30

*Cases defined in ASCE 7 Figure 6-9

Minimum vertical reinforcement area = 0.0012 × 12 × 6 = 0.09 in.^{2}/ft (ACI 14.3.2). The maximum bar spacing = 3h = 3 × 6 = 18 in. or 18 in. (ACI 14.3.5). Use 1 layer of No. 4 bars spaced at 18 in. (A_s = 0.13 in.^{2}/ft).

Minimum horizontal reinforcement area = 0.0020 × 12 × 6 = 0.14 in.^{2}/ft (ACI 14.3.2). Use 1 layer of No. 4 bars spaced at 16 in. (A_s = 0.15 in.^{2}/ft).

The shear strength V_n at any horizontal section must be less than or equal to $10\sqrt{f_c'}\,hd$ = 1,029 kips (ACI 11.10.3). In this case,

$$V_n = V_c + V_s = 205.8 + \frac{0.2 \times 60 \times 271.2}{16} = 409 \text{ kips} < 1{,}029 \text{ kips} \quad \text{O.K.}$$

Design for axial force and bending. ACI 14.4 requires that walls subjected to axial load or combined flexure and axial load shall be designed as compression members in accordance with ACI 10.2, 10.3, 10.10 through 10.14, 10.17, 14.2, and 14.3 unless the empirical design method of ACI 14.5 or the alternative design method of ACI 14.8 can be used. Clearly, the empirical and alternative methods cannot be used in this case, and the wall is designed in accordance with ACI 14.4.

Table 5-9 Summary of Design Axial Forces, Bending Moments, and Shear Forces in Flange Segment at Base of Shear Wall on Line 5 (SDC A)

Load Case	Full and Partial Wind Load Cases*	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (*D*)		176	0	0
Live (*L*)		39	0	0
Wind (*W*)	1	± 1	± 235	± 14
	2	0	± 202	± 12
	3	± 33	± 172	± 11
	4	± 32	± 151	± 9
Load Combinations				
1.4*D* + 1.7*L*		313	0	0
0.75(1.4*D* + 1.7*L* + 1.7*W*)	1	236	300	18
	2	235	258	15
	3	277	219	14
	4	275	193	12
0.9*D* + 1.3*W*	1	157	-306	-18
	2	158	-263	-16
	3	116	-224	-14
	4	117	-196	-12

*Cases defined in ASCE 7 Figure 6-9

Figure 5-2 contains the interaction diagram of the stem segment of the wall. The stem is reinforced with 19-No. 4 vertical bars. As seen from the figure, the stem segment is adequate for the load combinations in Table 5-8.

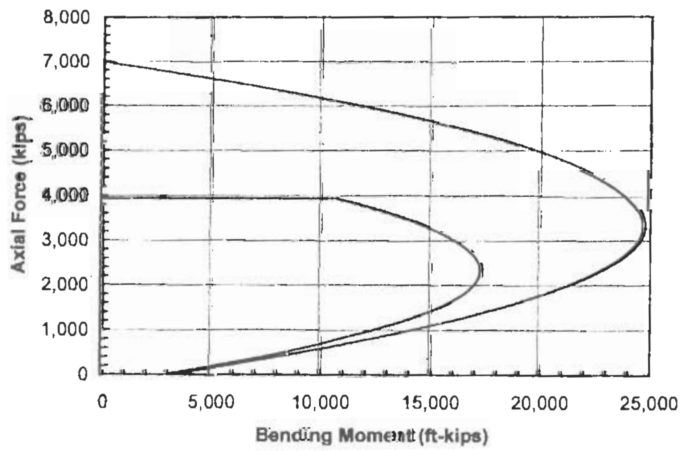


Figure 5-2 Design and Nominal Strength Interaction Diagrams for Stem Segment of Shear Wall Along Line 5 (SDC A)

Splice length of reinforcement. Class A lap splices are utilized for the vertical bars in the stem. No splices are required for the No. 4 horizontal bars in the stem, since full length bars weigh approximately $0.668 \times 28 = 19$ lbs. and are easily installed.

For the No. 4 vertical bars in the stem, ℓ_d is determined from ACI Eq. (12-1):

$$\frac{\ell_d}{d_b} = \frac{3}{40} \frac{f_y}{\sqrt{f_c'}} \frac{\alpha\beta\gamma\lambda}{\left(\frac{c + K_{tr}}{d_b}\right)}$$

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 0.8 for No. 6 and smaller bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} \frac{1}{2} \times 6 = 3.0 \text{ in.} \quad \text{(governs)} \\ \\ \frac{1}{2} \times 18 = 9.0 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index = 0

$$\frac{c + K_{tr}}{d_b} = \frac{3.0 + 0}{0.5} = 6.0 > 2.5, \text{ use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{0.8 \times 1.0 \times 1.0 \times 1.0}{2.5} = 22.8$$

$$\ell_d = 22.8 \times 0.5 = 11.4 \text{ in.} < 12 \text{ in., use } 12 \text{ in.}$$

$$\text{Class A splice length} = 1.0\ell_d = 1.0 \text{ ft}$$

Use a 1 ft-0 in. splice length for the No. 4 bars.

Wall Flange Segment.

Design for shear. The shear strength of the concrete is determined in accordance with ACI 11.10.5 for walls subjected to axial compression:

$$V_c = 2\sqrt{f'_c}hd = 2\sqrt{4{,}000} \times 6 \times 211.2/1{,}000 = 160.3 \text{ kips}$$

where $d = 0.8\ell_w = 0.8 \times 264 = 211.2$ in. (ACI 11.10.4).

The maximum factored shear force is 18 kips from wind load case 1 in the second and third load combinations (see Table 5-9). Since $V_u = 18$ kips $< \phi V_c/2 = 0.85 \times 160.3/2 =$ 68.1 kips, reinforcement shall be provided in accordance with ACI 11.10.9 or Chapter 14 (ACI 11.10.8).

The minimum reinforcement requirements are the same as those for the stem segment determined above. Therefore, use 1 layer of No. 4 vertical bars spaced at 18 in. (A_s = 0.13 in.^{2}/ft) and 1 layer of No. 4 horizontal bars spaced at 16 in. (A_s = 0.15 in.^{2}/ft) in the flange segment.

Design for axial force and bending. Figure 5-3 contains the interaction diagram of the flange segment of the wall. The flange is reinforced with 15-No. 4 vertical bars. As seen from the figure, the flange segment is adequate for the load combinations in Table 5-9.

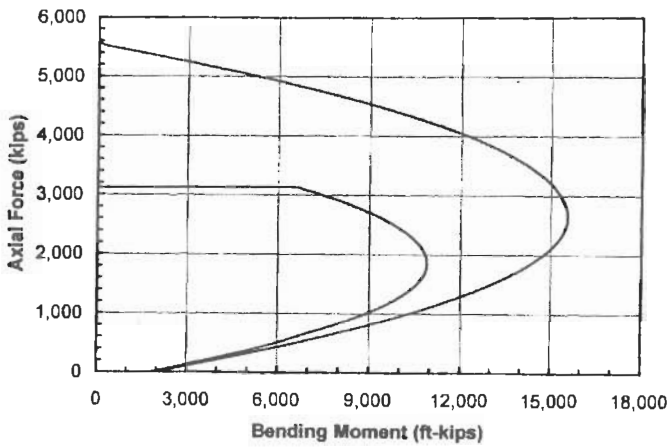


Figure 5-3 Design and Nominal Strength Interaction Diagrams for Flange Segment of Shear Wall Along Line 5 (SDC A)

Splice length of reinforcement. The splice length of the vertical bars in the flange segment is calculated the same as shown above for the stem segment. In this case, a Class B splice is required. Therefore, use a 1 ft-4 in. splice length for the No. 4 vertical bars in the flange.

Reinforcement details for the wall are shown in Figure 5-4.

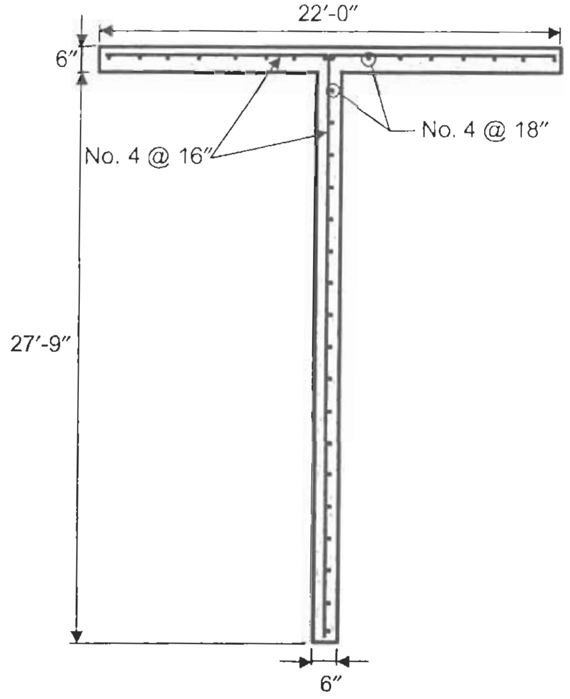


Figure 5-4 Reinforcement Details for Shear Wall Along Line 5 (SDC A)

5.3 DESIGN FOR SDC B

To illustrate the design requirements for Seismic Design Category (SDC) B, the residential building in Figure 5-1 is assumed to be located in Atlanta, GA. Walls are designed and detailed for combined effects of gravity, wind, and seismic forces.

5.3.1 Design Data

- Building Location: Atlanta, GA (zip code 30350)

- Material Properties

  Concrete: $f_c' = 4{,}000$ psi, $w_c = 150$ pcf
  Reinforcement: $f_y = 60{,}000$ psi

- Service Loads

  Live loads: roof = 20 psf
  floor = 100 psf for corridors, 40 psf elsewhere

Superimposed dead loads: roof = 10 psf
floor = 30 psf (20 psf permanent partitions + 10 psf ceiling, etc.)

- Seismic Design Data

For zip code 30350: $S_S = 0.276g, S_1 = 0.117g$ [5.1]
Site Class C (very dense soil / soft rock soil profile; IBC Table 1615.1.1)
For Category (Seismic Use Group) I occupancy, $I_E = 1.0$ (IBC Table 1604.5)

- Wind Design Data

Basic wind speed = 90 mph (IBC Figure 1609)
Exposure B (IBC 1609.4)
For Category I occupancy, $I_W = 1.0$ (IBC Table 1604.5)

- Member Dimensions

Joists: 12 + 4.5 × 5 + 30 (82 psf)
Wall thickness: 6 in.

5.3.2 Seismic Load Analysis

5.3.2.1 Seismic Design Category (SDC)

Analysis procedures for seismic design are given in IBC 1616.6. The appropriate procedure to use depends on the Seismic Design Category (SDC), which is determined in accordance with IBC 1616.3. Structures are assigned to a SDC based on their Seismic Use Group and the design spectral response acceleration parameters S_{DS} and S_{D1}. These parameters can be computed from Eqs. 16-18 and 16-19 in IBC 1615.1.3 or can be obtained from the provisions of IBC 1615.2.5 where site-specific procedures are used as required or permitted by IBC 1615.

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1-second period S_{M1} are determined from IBC Eqs. 16-16 and 16-17, respectively:

$$S_{MS} = F_a S_S = 1.2 \times 0.276 = 0.33g$$

$$S_{M1} = F_v S_1 = 1.68 \times 0.117 = 0.20g$$

where F_a and F_v are the values of site coefficients as a function of site class and of mapped spectral response acceleration at short periods S_S and at 1-second period S_1, respectively. The values of these coefficients for Site Class C in the equations above are contained in IBC Table 1615.1.2(1) for F_a and Table 1615.1.2(2) for F_v. Straight-line interpolation was used to determine F_v.

Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from Eqs. 16-18 and 16-19:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 0.33 = 0.22g$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.20 = 0.13g$$

Tables 1616.3(1) and 1616.3(2) are utilized to determine the SDC. From Table 1616.3(1) for Seismic Use Group I and S_{DS} = 0.22g, the SDC is B. Similarly, from Table 1616.3(2), the SDC is B for $S_{D1} = 0.13g$. Thus, the SDC is B for this building.

5.3.2.2 Seismic Forces

According to IBC 1616.6.2, the equivalent lateral force procedure in IBC 1617.4 may be used to compute the seismic base shear V for structures assigned to SDC B. In a given direction, V is determined from Eq. 16-34:

$$V = C_s W$$

where C_s is the seismic response coefficient determined in accordance with IBC 1617.4.1.1 and W is the effective weight of the structure defined in IBC 1617.4. For the member sizes and superimposed dead loads given above, W = 9,138 kips (see Table 5-10 below).

In both directions, a bearing wall system with ordinary reinforced concrete shear walls is utilized, which is permitted for structures assigned to SDC B without any limitations (see IBC Table 1617.6 and IBC 1910.3). The response modification coefficient R = 4.5 and the deflection amplification factor $C_d = 4$ (IBC Table 1617.6).

Approximate period (T_a). The fundamental period of the building T is determined in accordance with IBC 1617.4.2. In lieu of a more exact analysis, an approximate fundamental period T_a is computed from Eq. 16-39:

Building height $h_n = 70$ ft

Building period coefficient $C_T = 0.02$

Period $T_a = C_T (h_n)^{3/4} = 0.020 \times (70)^{3/4} = 0.48$ sec

Seismic base shear (*V*). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.13}{\left(\frac{4.5}{1.0}\right)\times 0.48} = 0.060$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{0.22}{\left(\frac{4.5}{1.0}\right)} = 0.049$$

Also, C_s must not be less than the following value from Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 0.22 \times 1.0 = 0.010$$

Thus, the value of C_s from Eq. 16-35 governs so that the base shear V in the N-S and E-W directions is:

$$V = C_s W = 0.049 \times 9{,}138 = 448 \text{ kips}$$

Vertical distribution of seismic forces. The total base shear is distributed over the height of the building in conformance with Eqs. 16-41 and 16-42:

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

where F_x is the lateral force induced at level x, w_x and w_i are the portions of W assigned to levels x or i, and k is the distribution exponent defined in IBC 1617.4.3. For $T = 0.48$ sec, $k = 1.0$. The lateral forces F_x and the story shears V_x are contained in Table 5-10.

Table 5-10 Seismic Forces and Story Shears (SDC B)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
7	978	70	68,460	87	87
6	1,360	60	81,600	103	190
5	1,360	50	68,000	86	276
4	1,360	40	54,440	69	345
3	1,360	30	40,800	52	397
2	1,360	20	27,200	34	431
1	1,360	10	13,600	17	448
Σ	9,138		354,060	448	

5.3.2.3 Method of Analysis

A three-dimensional analysis of the building was performed in both the N-S and E-W directions for the seismic forces contained in Table 5-10 using SAP2000 [5.3]. In the model, rigid diaphragms were assigned at each floor level. The stiffness properties of the walls were input assuming cracked sections. In lieu of a more accurate analysis, the cracked section property of the shear walls was taken as $I_{eff} = 0.5 I_g$ where I_g is the gross moment of inertia of the section. P-delta effects were also considered in the analysis.

In accordance with IBC 1617.4.4.4, the center of mass was displaced each way from its actual location a distance equal to 5 percent of the building dimension perpendicular to the applied forces to account for accidental torsion in seismic design. Torsional effects need not be amplified, since the building is assigned to SDC B (IBC 1617.4.4.5).

5.3.2.4 Story Drift and P-delta Effects

Story drift determination. Table 5-11 contains the displacements δ_{xe} in the N-S and E-W directions obtained from the 3-D static, elastic analysis using the design seismic forces, including accidental torsional effects. The table also contains the design earthquake displacement δ_x computed by IBC Eq. 16-46:

$$\delta_x = \frac{C_d \delta_{xe}}{I_E}$$

where C_d is the deflection amplification factor in Table 1617.6, which depends on the seismic-force-resisting system. As noted above, C_d is equal to 4 for this system.

Table 5-11 Lateral Displacements and Interstory Drifts due to Seismic Forces in N-S and E-W Directions (SDC B)

Story	N-S Direction			E-W Direction		
	δ_{xe} (in.)	δ_x (in.)	Δ (in.)	δ_{xe} (in.)	δ_x (in.)	Δ (in.)
7	0.076	0.304	0.048	0.088	0.352	0.040
6	0.064	0.256	0.052	0.078	0.312	0.044
5	0.051	0.204	0.052	0.067	0.268	0.056
4	0.038	0.152	0.052	0.053	0.212	0.064
3	0.025	0.100	0.044	0.037	0.148	0.060
2	0.014	0.056	0.036	0.022	0.088	0.056
1	0.005	0.020	0.020	0.008	0.032	0.032

The interstory drifts Δ computed from the δ_x are also contained in the table. For this structure that does not have plan irregularity Type 1a or 1b of Table 1616.5.1, the drift at story level x is determined by subtracting the design earthquake displacement at the center of mass at the bottom of the story from the design earthquake displacement at the center of mass at the top of the story (IBC 1617.4.6.1):

$$\Delta = \delta_x - \delta_{x-1}$$

The design story drifts Δ must not exceed the allowable story drift Δ_a from Table 1617.3 (IBC 1617.3). For Seismic Use Group I, $\Delta_a = 0.020 h_{sx}$ where h_{sx} is the story height below level x. Thus, for the 10-ft story heights, $\Delta_a = 0.020 \times 10 \times 12 = 2.40$ in. It is evident from Table 5-11 that for all stories, the lateral drifts obtained from the prescribed lateral forces in the N-S and E-W directions are less than the limiting values.

P-delta effects. As noted above, P-delta effects were automatically considered in the analysis using SAP2000.

5.3.3 Wind Load Analysis

5.3.3.1 Wind Forces

The analytical procedure (Method 2) of ASCE 6.5 is used to determine the wind forces.

Details on how to compute the wind forces in both the N-S and E-W directions are given in Section 5.2.3 of this publication. A summary of the design wind forces in both directions is contained in Table 5-12. Once again it is important to note that the wind directionality factor K_d has been taken equal to 1.0 (see Exception 1 in IBC 1605.2.1).

Table 5-12 Design Wind Forces in N-S and E-W Directions (V = 90 mph)

Level	Height above ground level, z (ft)	Total Design Wind Force N-S (kips)	Total Design Wind Force E-W (kips)
7	70	14.4	5.3
6	60	28.0	10.3
5	50	27.1	9.9
4	40	26.2	9.5
3	30	24.9	9.0
2	20	23.5	8.3
1	10	22.5	7.9
	Σ	166.6	60.2

5.3.3.2 Method of Analysis

Similar to the seismic analysis, a three-dimensional analysis of the building was performed in the N-S and E-W directions for the wind forces contained in Table 5-12 using SAP2000. The modeling assumptions utilized for the seismic analysis were also used for the wind analysis.

According to ASCE 6.5.12.3, main wind-force-resisting systems of buildings with mean roof height h greater than 60 ft must be designed for the full and partial wind load cases of Figure 6-9 (Cases 1 through 4). These four cases were considered in the three-dimensional analysis.

5.3.4 Design for Combined Load Effects

5.3.4.1 Load Combinations

Basic load combinations for strength design are given in IBC 1605.2.1. As noted above, the first exception in this section requires that the non-seismic load combinations of ACI 9.2 be used for concrete structures. Thus, the following load combinations are utilized in the design of the structural members:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)
4. $1.2D + f_1 L + 1.0E$ (Formula 16-5)
5. $0.9D + 1.0E$ (Formula 16-6)

where D, L, W, and E are the effects due to dead, live, wind, and seismic loads, respectively, and f_1 is equal to either 1.0 for places of public assembly, for live loads in excess of 100 psf, and for parking garage live load, or is equal to 0.5 for other live loads.

The seismic load effect E for use in Formula 16-5, which is the combined effect of horizontal and vertical earthquake-induced forces, is computed from Eq. 16-28 where the effects of gravity and seismic ground motion are additive:

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of horizontal seismic forces
ρ = redundancy coefficient
= 1.0 for structures assigned to SDC A, B, or C (IBC 1617.2.1)

Similarly, E for use in Formula 16-6 is computed from Eq. 16-29 where the effects of gravity and seismic ground motion counteract:

$$E = \rho Q_E - 0.2 S_{DS} D$$

Substituting $S_{DS} = 0.22g$ and $\rho = 1.0$ into the equations for E, and then substituting E along with $f_1 = 0.5$ into load combinations 4 and 5 above results in the following:

4a. $1.2D + 0.5L + 1.0Q_E + (0.2 \times 0.22)D = 1.24D + 0.5L + Q_E$

4b. $1.2D + 0.5L + 1.0Q_E - (0.2 \times 0.22)D = 1.16D + 0.5L + Q_E$

5a. $0.9D + 1.0Q_E + (0.2 \times 0.22)D = 0.94D + Q_E$

5b. $0.9D + 1.0Q_E - (0.2 \times 0.22)D = 0.86D + Q_E$

Values of Q_E are obtained from the structural analysis for both sidesway to the left and to the right based on the code-prescribed horizontal seismic forces acting on the building. Also, the special seismic load combinations in IBC 1605.4 are not applicable in this example.

5.3.4.2 Design of Shear Wall on Line 5

Comparing the seismic forces in Table 5-10 to the wind forces in Table 5-12, it is clear that the seismic forces will govern the design of the shear walls. Table 5-13 contains a summary of the design axial forces, bending moments, and shear forces at the base of the stem segment of the wall. Similarly, Table 5-14 contains a summary of the reactions at the base of the flange segment of the wall.

Wall Stem Segment.

Design for shear. The shear strength of the concrete is determined in accordance with ACI 11.10.5 for walls subjected to axial compression:

$$V_c = 2\sqrt{f'_c}hd = 2\sqrt{4{,}000} \times 6 \times 271.2 / 1{,}000 = 205.8 \text{ kips}$$

where d is permitted to be taken equal to $0.8\ell_w = 0.8 \times 339 = 271.2$ in. (ACI 11.10.4).

Table 5-13 Summary of Design Axial Forces, Bending Moments, and Shear Forces in Stem Segment at Base of Shear Wall on Line 5 (SDC B)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (*D*)	739	0	0
Live (*L*)	95	0	0
Seismic (Q_E)	± 81	± 1,172	± 46
Load Combinations			
1.4*D* + 1.7*L*	1,196	0	0
1.24*D* + 0.5*L* + Q_E	1,045	1,172	46
0.86*D* + Q_E	555	-1,172	-46

Table 5-14 Summary of Design Axial Forces, Bending Moments, and Shear Forces in Flange Segment at Base of Shear Wall on Line 5 (SDC B)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (*D*)	176	0	0
Live (*L*)	39	0	0
Seismic (Q_E)	± 1	± 940	± 49
Load Combinations			
1.4*D* + 1.7*L*	313	0	0
1.24*D* + 0.5*L* + Q_E	239	940	49
0.86*D* + Q_E	150	-940	-49

The maximum factored shear force is 46 kips from the second and third load combinations (see Table 5-13). Since $V_u = 46$ kips $< \phi V_c/2 = 0.85 \times 205.8/2 = 87.5$ kips, reinforcement shall be provided in accordance with ACI 11.10.9 or Chapter 14 (ACI 11.10.8).

Minimum vertical reinforcement area = $0.0012 \times 12 \times 6 = 0.09$ in.^{2}/ft (ACI 14.3.2). The maximum bar spacing = $3h = 3 \times 6 = 18$ in. or 18 in. (ACI 14.3.5). Use 1 layer of No. 4 bars spaced at 18 in. ($A_s = 0.13$ in.^{2}/ft).

Minimum horizontal reinforcement area = $0.0020 \times 12 \times 6 = 0.14$ in.^{2}/ft (ACI 14.3.2). Use 1 layer of No. 4 bars spaced at 16 in. ($A_s = 0.15$ in.^{2}/ft).

The shear strength V_n at any horizontal section must be less than or equal to $10\sqrt{f_c'}hd$ = 1,029 kips (ACI 11.10.3). In this case,

$$V_n = V_c + V_s = 205.8 + \frac{0.2 \times 60 \times 271.2}{16} = 409 \text{ kips} < 1{,}029 \text{ kips} \quad \text{O.K.}$$

Design for axial force and bending. ACI 14.4 requires that walls subjected to axial load or combined flexure and axial load shall be designed as compression members in accordance with ACI 10.2, 10.3, 10.10 through 10.14, 10.17, 14.2, and 14.3 unless the empirical design method of ACI 14.5 or the alternative design method of ACI 14.8 can be used. Clearly, the empirical and alternative methods cannot be used in this case, and the wall will be designed in accordance with ACI 14.4.

Figure 5-2 contains the interaction diagram of the stem segment of the wall. The stem is reinforced with 19-No. 4 vertical bars. As seen from the figure, the stem segment is adequate for the load combinations in Table 5-13.

IBC 1620.1.7 requires that bearing walls and shear walls and their anchorage in buildings assigned to SDC B and above be designed for an out-of-plane force F_p that is the greater of 10 percent of the weight of the wall w_w or the value obtained from Eq. 16-63:

$$F_p = 0.40 I_E S_{DS} w_w$$

Out-of-plane seismic forces are usually computed for a 1-ft width of the wall length, assuming a uniformly distributed out-of-plane loading.

For the stem segment of the wall:

$$w_w = \frac{6}{12} \times 10 \times 0.15 = 0.75 \text{ kips/ft width of stem}$$

$$F_p = \begin{cases} 0.10 w_w = 0.10 \times 0.75 = 0.075 \text{ kips/ft} \quad \text{(governs)} \\ 0.40 I_E S_{DS} w_w = 0.40 \times 1.0 \times 0.22 \times 0.75 = 0.066 \text{ kips/ft} \end{cases}$$

Distributed load = 0.075/10 = 0.0075 klf/ft

Assuming that the wall is simply supported at the floor levels, the maximum bending moment due to the out-of-plane seismic force is:

$$M_u = \frac{0.0075 \times 10^2}{8} = 0.1 \text{ ft-kips/ft}$$

The design strength of a 1-ft wide segment of the stem reinforced with No. 4 vertical bars spaced at 18 in. is equal to ϕM_n = 1.7 ft-kips/ft > 0.1 ft-kips/ft O.K.

Splice length of reinforcement. The splice length of the No. 4 vertical bars in the stem segment was computed in Section 5.2.4.2 of this publication. Use a 1 ft-0 in. splice length.

Wall Flange Segment.

Design for shear. The shear strength of the concrete is determined in accordance with ACI 11.10.5 for walls subjected to axial compression:

$$V_c = 2\sqrt{f_c'}hd = 2\sqrt{4{,}000} \times 6 \times 211.2 / 1{,}000 = 160.3 \text{ kips}$$

where $d = 0.8\ell_w = 0.8 \times 264 = 211.2$ in. (ACI 11.10.4).

The maximum factored shear force is 49 kips from the second and third load combinations (see Table 5-14). Since $V_u = 49$ kips $< \phi V_c / 2 = 0.85 \times 160.3/2 = 68.1$ kips, reinforcement shall be provided in accordance with ACI 11.10.9 or Chapter 14 (ACI 11.10.8).

The minimum reinforcement requirements are the same as those for the stem segment determined above. Therefore, use 1 layer of No. 4 vertical bars spaced at 18 in. ($A_s = 0.13$ in.^{2}/ft) and 1 layer of No. 4 horizontal bars spaced at 16 in. ($A_s = 0.15$ in.^{2}/ft).

Design for axial force and bending. Figure 5-3 contains the interaction diagram of the flange segment of the wall. The flange is reinforced with 15-No. 4 vertical bars. As seen from the figure, the flange segment is adequate for the load combinations in Table 5-14.

The vertical reinforcement is also adequate to resist the required out-of-plane seismic force prescribed in IBC 1620.1.7.

Splice length of reinforcement. The splice length of the No. 4 vertical bars in the flange segment was computed in Section 5.2.4.2 of this publication. Use a 1 ft-4 in. splice length.

Reinforcement details are the same as shown in Figure 5-4 for SDC A.

5.4 DESIGN FOR SDC C

To illustrate the design requirements for Seismic Design Category (SDC) C, the residential building in Figure 5-1 is assumed to be located in New York, NY.

5.4.1 Design Data

- Building Location: New York, NY (zip code 10013)

- Material Properties

  Concrete: $f_c' = 4{,}000$ psi, $w_c = 150$ pcf
  Reinforcement: $f_y = 60{,}000$ psi

- Service Loads

  Live loads: roof = 20 psf
  floor = 100 psf for corridors, 40 psf elsewhere

  Superimposed dead loads: roof = 10 psf
  floor = 30 psf (20 psf permanent partitions + 10 psf ceiling, etc.)

- Seismic Design Data

  For zip code 10013: $S_S = 0.424g, S_1 = 0.094g$ [5.1]
  Site Class D (stiff soil profile; IBC Table 1615.1.1)
  For Category (Seismic Use Group) I occupancy, $I_E = 1.0$ (IBC Table 1604.5)

- Wind Design Data

  Basic wind speed = 110 mph (IBC Figure 1609)
  Exposure B (IBC 1609.4)
  For Category I occupancy, $I_W = 1.0$ (IBC Table 1604.5)

- Member Dimensions

  Joists: 12 + 4.5 × 5 + 30 (82 psf)
  Wall thickness: 6 in.

5.4.2 Seismic Load Analysis

5.4.2.1 Seismic Design Category (SDC)

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1-second period S_{M1} are determined from IBC Eqs. 16-16 and 16-17, respectively:

$$S_{MS} = F_a S_S = 1.46 \times 0.424 = 0.62g$$

$$S_{M1} = F_v S_1 = 2.4 \times 0.094 = 0.23g$$

where F_a and F_v are the values of site coefficients as a function of site class and of mapped spectral response acceleration at short periods S_S and at 1-second period S_1, respectively. The values of these coefficients for Site Class D in the equations above are contained in IBC Table 1615.1.2(1) for F_a and Table 1615.1.2(2) for F_v. Straight-line interpolation was used to determine F_a.

Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from Eqs. 16-18 and 16-19:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 0.62 = 0.41g$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.23 = 0.15g$$

Tables 1616.3(1) and 1616.3(2) are utilized to determine the SDC. From Table 1616.3(1) for Seismic Use Group I and S_{DS} = 0.41g, the SDC is C. Similarly, from Table 1616.3(2), the SDC is C for $S_{D1} = 0.15g$. Thus, the SDC is C for this building.

5.4.2.2 Seismic Forces

According to IBC 1616.6.2, the equivalent lateral force procedure in IBC 1617.4 may be used to compute the seismic base shear V for structures assigned to SDC C. In a given direction, V is determined from Eq. 16-34:

$$V = C_s W$$

where C_s is the seismic response coefficient determined in accordance with IBC 1617.4.1.1 and W is the effective weight of the structure defined in IBC 1617.4. For the member sizes and superimposed dead loads given above, W = 9,138 kips (see Table 5-15 below).

In both directions, a bearing wall system with ordinary reinforced concrete shear walls is utilized, which is permitted for structures assigned to SDC C without any limitations (see IBC Table 1617.6 and IBC 1910.3). The response modification coefficient R = 4.5 and the deflection amplification factor C_d = 4 are found in IBC Table 1617.6.

Approximate period (T_a). The fundamental period of the building T is determined in accordance with IBC 1617.4.2. In lieu of a more exact analysis, an approximate fundamental period T_a is computed from Eq. 16-39:

Building height h_n = 70 ft

Building period coefficient $C_T = 0.02$

Period $T_a = C_T (h_n)^{3/4} = 0.020 \times (70)^{3/4} = 0.48$ sec

Seismic base shear (V). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.15}{\left(\frac{4.5}{1.0}\right)\times 0.48} = 0.069$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{0.41}{\left(\frac{4.5}{1.0}\right)} = 0.091$$

Also, C_s must not be less than the following value from Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 0.41 \times 1.0 = 0.018$$

Thus, the value of C_s from Eq. 16-36 governs so that the base shear V in the N-S and E-W directions is:

$$V = C_s W = 0.069 \times 9{,}138 = 631 \text{ kips}$$

Vertical distribution of seismic forces. The total base shear is distributed over the height of the building in conformance with Eqs. 16-41 and 16-42:

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

where F_x is the lateral force induced at level x, w_x and w_i are the portions of W assigned to levels x or i, and k is the distribution exponent defined in IBC 1617.4.3. For T = 0.48 sec, k = 1.0. The lateral forces F_x and the story shears V_x are contained in Table 5-15.

Table 5-15 Seismic Forces and Story Shears (SDC C)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
7	978	70	68,460	122	122
6	1,360	60	81,600	145	267
5	1,360	50	68,000	121	388
4	1,360	40	54,440	97	485
3	1,360	30	40,800	73	558
2	1,360	20	27,200	49	607
1	1,360	10	13,600	24	631
Σ	9,138		354,060	631	

5.4.2.3 Method of Analysis

A three-dimensional analysis of the building was performed in both the N-S and E-W directions for the seismic forces contained in Table 5-15 using SAP2000 [5.3]. In the model, rigid diaphragms were assigned at each floor level. The stiffness properties of the walls were input assuming cracked sections. In lieu of a more accurate analysis, the cracked section property of the shear walls was taken as $I_{eff} = 0.5I_g$ where I_g is the gross moment of inertia of the section. P-delta effects were also considered in the analysis.

In accordance with IBC 1617.4.4.4, the center of mass was displaced each way from its actual location a distance equal to 5 percent of the building dimension perpendicular to the applied forces to account for accidental torsion in seismic design.

5.4.2.4 Story Drift and P-delta Effects

Story drift determination. Table 5-16 contains the displacements δ_{xe} in the N-S and E-W directions obtained from the 3-D static, elastic analysis using the design seismic forces, including accidental torsional effects. The table also contains the design earthquake displacement δ_x computed by IBC Eq. 16-46:

$$\delta_x = \frac{C_d \delta_{xe}}{I_E}$$

where C_d is the deflection amplification factor in Table 1617.6, which depends on the seismic-force-resisting system. As noted above, C_d is equal to 4 for this system.

The interstory drifts Δ computed from the δ_x are also contained in the table. For this structure that does not have plan irregularity Types 1a or 1b of Table 1616.5.1, the drift at story level x is determined by subtracting the design earthquake displacement at the center of mass at the bottom of the story from the design earthquake displacement at the center of mass at the top of the story (IBC 1617.4.6.1):

$$\Delta = \delta_x - \delta_{x-1}$$

Table 5-16 Lateral Displacements and Interstory Drifts due to Seismic Forces in N-S and E-W Directions (SDC C)

Story	N-S Direction			E-W Direction		
	δ_{xe} (in.)	δ_x (in.)	Δ (in.)	δ_{xe} (in.)	δ_x (in.)	Δ (in.)
7	0.095	0.380	0.064	0.110	0.440	0.048
6	0.079	0.316	0.064	0.098	0.392	0.060
5	0.063	0.252	0.064	0.083	0.332	0.068
4	0.047	0.188	0.064	0.066	0.264	0.080
3	0.031	0.124	0.056	0.046	0.184	0.076
2	0.017	0.068	0.044	0.027	0.108	0.068
1	0.006	0.024	0.024	0.010	0.040	0.040

The design story drifts Δ must not exceed the allowable story drift Δ_a from Table 1617.3 (IBC 1617.3). For Seismic Use Group I, $\Delta_a = 0.020h_{sx}$ where h_{sx} is the story height below level x. Thus, for the 10-ft story heights, $\Delta_a = 0.020 \times 10 \times 12 =$ 2.40 in. It is evident from Table 5-16 that for all stories, the lateral drifts obtained from the prescribed lateral forces in the N-S and E-W directions are less than the limiting values.

P-delta effects. As noted above, P-delta effects were automatically considered in the analysis using SAP2000.

5.4.3 Wind Load Analysis

5.4.3.1 Wind Forces

The analytical procedure (Method 2) of ASCE 6.5 is used to determine the wind forces.

Details on how to compute the wind forces in both the N-S and E-W directions are given in Section 5.2.3 of this publication. A summary of the design wind forces in both directions is contained in Table 5-17. Once again it is important to note that the wind directionality factor K_d has been taken equal to 1.0 (see Exception 1 in IBC 1605.2.1).

5.4.3.2 Method of Analysis

Similar to the seismic analysis, a three-dimensional analysis of the building was performed in the N-S and E-W directions for the wind forces contained in Table 5-17 using SAP2000. The modeling assumptions utilized for the seismic analysis were also used for the wind analysis.

Table 5-17 Design Wind Forces in N-S and E-W Directions (V = 110 mph)

Level	Height above ground level, *z* (ft)	Total Design Wind Force N-S (kips)	Total Design Wind Force E-W (kips)
7	70	21.5	7.9
6	60	41.8	15.4
5	50	40.5	14.8
4	40	39.1	14.2
3	30	37.2	13.4
2	20	35.1	12.4
1	10	33.6	11.8
	Σ	248.8	89.9

According to ASCE 6.5.12.3, main wind-force-resisting systems of buildings with mean roof height h greater than 60 ft must be designed for the full and partial wind load cases of Figure 6-9 (Cases 1 through 4). These four cases were considered in the three-dimensional analysis.

5.4.4 Design for Combined Load Effects

5.4.4.1 Load Combinations

Basic load combinations for strength design are given in IBC 1605.2.1. As noted above, the first exception in this section requires that the non-seismic load combinations of ACI 9.2 be used for concrete structures. Thus, the following load combinations are applicable:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)
4. $1.2D + f_1 L + 1.0E$ (Formula 16-5)
5. $0.9D + 1.0E$ (Formula 16-6)

where D, L, W, and E are the effects due to dead, live, wind, and seismic loads, respectively, and f_1 is equal to either 1.0 for places of public assembly, for live loads in excess of 100 psf, and for parking garage live load, or is equal to 0.5 for other live loads.

The seismic load effect E for use in Formula 16-5, which is the combined effect of horizontal and vertical earthquake-induced forces, is computed from Eq. 16-28 where the effects of gravity and seismic ground motion are additive:

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of horizontal seismic forces
ρ = redundancy coefficient
= 1.0 for structures assigned to SDC A, B, or C (IBC 1617.2.1)

Similarly, E for use in Formula 16-6 is computed from Eq. 16-29 where the effects of gravity and seismic ground motion counteract:

$$E = \rho Q_E - 0.2 S_{DS} D$$

Substituting $S_{DS} = 0.41g$ and $\rho = 1.0$ into the equations for E, and then substituting E along with $f_1 = 0.5$ into load combinations 4 and 5 above results in the following:

4a. $1.2D + 0.5L + 1.0Q_E + (0.2 \times 0.41)D = 1.28D + 0.5L + Q_E$

4b. $1.2D + 0.5L + 1.0Q_E - (0.2 \times 0.41)D = 1.12D + 0.5L + Q_E$

5a. $0.9D + 1.0Q_E + (0.2 \times 0.41)D = 0.98D + Q_E$

5b. $0.9D + 1.0Q_E - (0.2 \times 0.41)D = 0.82D + Q_E$

Values of Q_E are obtained from the structural analysis for both sidesway to the left and to the right based on the code-prescribed horizontal seismic forces acting on the building.

5.4.4.2 Design of Shear Wall on Line 5

Comparing the seismic forces in Table 5-15 to the wind forces in Table 5-17, it is clear that the seismic forces will govern the design of the shear walls. Table 5-18 contains a summary of the design axial forces, bending moments, and shear forces at the base of the stem segment of the wall. Similarly, Table 5-19 contains a summary of the reactions at the base of the flange segment of the wall.

Wall Stem Segment.

Design for shear. The shear strength of the concrete is determined in accordance with ACI 11.10.5 for walls subjected to axial compression:

$$V_c = 2\sqrt{f_c'}hd = 2\sqrt{4{,}000} \times 6 \times 271.21{,}000 = 205.8 \text{ kips}$$

where d is equal to $0.8\ell_w = 0.8 \times 339 = 271.2$ in. (ACI 11.10.4).

Table 5-18 Summary of Design Axial Forces, Bending Moments, and Shear Forces in Stem Segment at Base of Shear Wall on Line 5 (SDC C)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (*D*)	739	0	0
Live (*L*)	95	0	0
Seismic (Q_E)	± 100	± 1,456	± 57
Load Combinations			
1.4*D* + 1.7*L*	1,196	0	0
1.28*D* + 0.5*L* + Q_E	1,093	1,456	57
0.82*D* + Q_E	506	-1,456	-57

Table 5-19 Summary of Design Axial Forces, Bending Moments, and Shear Forces in Flange Segment at Base of Shear Wall on Line 5 (SDC C)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (*D*)	176	0	0
Live (*L*)	39	0	0
Seismic (Q_E)	± 1	± 1,167	± 61
Load Combinations			
1.4*D* + 1.7*L*	313	0	0
1.28*D* + 0.5*L* + Q_E	246	1,167	61
0.82*D* + Q_E	143	-1,167	-61

The maximum factored shear force is 57 kips from the second and third load combinations (see Table 5-18). Since V_u = 57 kips < $\phi V_c/2$ = 0.85 × 205.8/2 = 87.5 kips, reinforcement shall be provided in accordance with ACI 11.10.9 or Chapter 14 (ACI 11.10.8).

Minimum vertical reinforcement area = 0.0012 × 12 × 6 = 0.09 in.^{2}/ft (ACI 14.3.2). The maximum bar spacing = 3*h* = 3 × 6 = 18 in. or 18 in. (ACI 14.3.5). Use 1 layer of No. 4 bars spaced at 18 in. (A_s = 0.13 in.^{2}/ft).

Minimum horizontal reinforcement area = 0.0020 × 12 × 6 = 0.14 in.^{2}/ft (ACI 14.3.2). Use 1 layer of No. 4 bars spaced at 16 in. (A_s = 0.15 in.^{2}/ft).

The shear strength V_n at any horizontal section must be less than or equal to $10\sqrt{f'_c}hd$ = 1,029 kips (ACI 11.10.3). In this case,

$$V_n = V_c + V_s = 205.8 + \frac{0.2 \times 60 \times 271.2}{16} = 409 \text{ kips} < 1{,}029 \text{ kips} \quad \text{O.K.}$$

Design for axial force and bending. Figure 5-2 contains the interaction diagram of the stem segment of the wall. The stem is reinforced with 19-No. 4 vertical bars. As seen from the figure, the stem segment is adequate for the load combinations in Table 5-18.

IBC 1620.1.7 requires that bearing walls and shear walls and their anchorage in buildings assigned to SDC B and above be designed for an out-of-plane force F_p that is the greater of 10 percent of the weight of the wall w_w or the value obtained from Eq. 16-63:

$$F_p = 0.40 I_E S_{DS} w_w$$

Calculations given in Section 5.3.4.2 of this publication show that the design strength of the stem segment is greater than the required strength when the stem is subjected to F_p.

Splice length of reinforcement. The splice length of the No. 4 vertical bars in the stem segment was computed in Section 5.2.4.2 of this publication. In this example, a Class B splice is required. Use a 1 ft-4 in. splice length.

Wall Flange Segment.

Design for shear. The shear strength of the concrete is determined in accordance with ACI 11.10.5 for walls subjected to axial compression:

$$V_c = 2\sqrt{f'_c}hd = 2\sqrt{4{,}000} \times 6 \times 211.2 / 1{,}000 = 160.3 \text{ kips}$$

where $d = 0.8\ell_w = 0.8 \times 264 = 211.2$ in. (ACI 11.10.4).

The maximum factored shear force is 61 kips from the second and third load combinations (see Table 5-19). Since $V_u = 61$ kips $< \phi V_c / 2 = 0.85 \times 160.3/2 =$ 68.1 kips, reinforcement shall be provided in accordance with ACI 11.10.9 or Chapter 14 (ACI 11.10.8).

The minimum reinforcement requirements are the same as those for the stem segment determined above. Therefore, use 1 layer of No. 4 vertical bars spaced at 18 in. ($A_s =$ 0.13 in.^{2}/ft) and 1 layer of No. 4 horizontal bars spaced at 16 in. ($A_s = 0.15$ in.^{2}/ft).

Design for axial force and bending. Figure 5-3 contains the interaction diagram of the flange segment of the wall. The flange is reinforced with 15-No. 4 vertical bars. As seen from the figure, the flange segment is adequate for the load combinations in Table 5-19.

The vertical reinforcement is also adequate to resist the required out-of-plane seismic force prescribed in IBC 1620.1.7.

Splice length of reinforcement. The splice length of the No. 4 vertical bars in the flange segment was computed in Section 5.2.4.2 of this publication. Use a 1 ft-4 in. splice length.

Reinforcement details are the same as shown in Figure 5-4 for SDC A.

5.5 DESIGN FOR SDC D

To illustrate the design requirements for Seismic Design Category (SDC) D, the residential building in Figure 5-1 is assumed to be located in San Francisco.

5.5.1 Design Data

- Building Location: San Francisco, CA (zip code 94105)

- Material Properties

  Concrete: $f_c' = 4{,}000$ psi, $w_c = 150$ pcf
  Reinforcement: $f_y = 60{,}000$ psi

- Service Loads

  Live loads: roof = 20 psf
  floor = 100 psf for corridors, 40 psf elsewhere

  Superimposed dead loads: roof = 10 psf
  floor = 30 psf (20 psf permanent partitions + 10 psf ceiling, etc.)

- Seismic Design Data

  For zip code 94105: $S_S = 1.50g, S_1 = 0.61g$ [5.1]
  Site Class D (stiff soil profile; IBC Table 1615.1.1)
  For Category (Seismic Use Group) I occupancy, $I_E = 1.0$ (IBC Table 1604.5)

- Wind Design Data

  Basic wind speed = 85 mph (IBC Figure 1609)
  Exposure B (IBC 1609.4)
  For Category I occupancy, $I_W = 1.0$ (IBC Table 1604.5)

- Member Dimensions

  Joists: 12 + 4.5 × 5 + 30 (82 psf)
  Wall thickness: 8 in.

5.5.2 Seismic Load Analysis

5.5.2.1 Seismic Design Category (SDC)

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1-second period S_{M1} are determined from IBC Eqs. 16-16 and 16-17, respectively:

$$S_{MS} = F_a S_S = 1.0 \times 1.50 = 1.50g$$

$$S_{M1} = F_v S_1 = 1.5 \times 0.61 = 0.92g$$

where F_a and F_v are contained in IBC Table 1615.1.2(1) and Table 1615.1.2(2), respectively.

Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from Eqs. 16-18 and 16-19:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 1.50 = 1.00g$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.92 = 0.61g$$

Tables 1616.3(1) and 1616.3(2) are utilized to determine the SDC. From Table 1616.3(1) for Seismic Use Group I and S_{DS} = 1.00g, the SDC is D. Similarly, from Table 1616.3(2), the SDC is D for S_{D1} = 0.61g. Thus, the SDC is D for this building.

5.5.2.2 Seismic Forces

Since the building does not have plan irregularity Type 1a, 1b, or 4 of Table 1616.5.1 or vertical irregularity Type 1a, 1b, 4, or 5 of Table 1616.5.2, it can be considered regular (IBC 1616.6.3). For this regular building that is less than 240 ft in height, Table 1616.6.3 allows the equivalent lateral force procedure in IBC 1617.4 to be used to compute the seismic base shear V (see Eq. 16-34):

$$V = C_s W$$

where C_s is the seismic response coefficient determined in accordance with IBC 1617.4.1.1 and W is the effective weight of the structure defined in IBC 1617.4. For the member sizes and superimposed dead loads given above, W = 10,035 kips (see Table 5-20 below).

In both directions, a bearing wall system with special reinforced concrete shear walls is utilized, which is permitted for structures assigned to SDC D with a height less than or equal to 160 ft (see IBC Table 1617.6 and IBC 1910.5). The response modification coefficient $R = 5.5$ and the deflection amplification factor $C_d = 5$ (IBC Table 1617.6).

Approximate period (T_a). The fundamental period of the building T is determined in accordance with IBC 1617.4.2. In lieu of a more exact analysis, an approximate fundamental period T_a is computed from Eq. 16-39:

Building height $h_n = 70$ ft

Building period coefficient $C_T = 0.02$

Period $T_a = C_T (h_n)^{3/4} = 0.020 \times (70)^{3/4} = 0.48$ sec

Seismic base shear (V). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.61}{\left(\frac{5.5}{1.0}\right)\times 0.48} = 0.230$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{1.00}{\left(\frac{5.5}{1.0}\right)} = 0.182$$

Also, C_s must not be less than the following value from Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 1.00 \times 1.0 = 0.044$$

For buildings assigned to SDC E or F and for those buildings for which $S_1 \geq 0.6g$, C_s shall not be taken less than that computed from Eq. 16-38. Since $S_1 = 0.61g$, Eq. 16-38 is applicable, even though the SDC is D:

$$C_s = \frac{0.5 S_1}{R / I_E} = \frac{0.5 \times 0.61}{5.5/1.0} = 0.056$$

In this case, the lower limit is 0.056 from Eq. 16-38.

Thus, the value of C_s from Eq. 16-35 governs so that the base shear V in the N-S and E-W directions is:

$$V = C_s W = 0.182 \times 10{,}035 = 1{,}826 \text{ kips}$$

Vertical distribution of seismic forces. The total base shear is distributed over the height of the building in conformance with Eqs. 16-41 and 16-42:

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

where F_x is the lateral force induced at level x, w_x and w_i are the portions of W assigned to levels x or i, and k is the distribution exponent defined in IBC 1617.4.3. For T = 0.48 sec, k = 1.0. The lateral forces F_x and the story shears V_x are contained in Table 5-20.

Table 5-20 Seismic Forces and Story Shears (SDC D)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
7	1,047	70	73,290	345	345
6	1,498	60	89,880	423	768
5	1,498	50	74,900	353	1,121
4	1,498	40	59,920	282	1,403
3	1,498	30	44,940	212	1,615
2	1,498	20	29,960	141	1,756
1	1,498	10	14,980	70	1,826
Σ	10,035		387,870	1,826	

5.5.2.3 Method of Analysis

A three-dimensional analysis of the building was performed in both the N-S and E-W directions for the seismic forces contained in Table 5-20 using SAP2000 [5.3]. In the model, rigid diaphragms were assigned at each floor level. The stiffness properties of the walls were input assuming cracked sections. In lieu of a more accurate analysis, the cracked section property of the shear walls was taken as $I_{eff} = 0.5 I_g$ where I_g is the gross moment of inertia of the section. P-delta effects were also considered in the analysis.

In accordance with IBC 1617.4.4.4, the center of mass was displaced each way from its actual location a distance equal to 5 percent of the building dimension perpendicular to the applied forces to account for accidental torsion in seismic design.

5.5.2.4 Story Drift and P-delta Effects

Story drift determination. Table 5-21 contains the displacements δ_{xe} in the N-S and E-W directions obtained from the 3-D static, elastic analysis using the design seismic forces, including accidental torsional effects. The table also contains the design earthquake displacement δ_x computed by IBC Eq. 16-46:

$$\delta_x = \frac{C_d \delta_{xe}}{I_E}$$

where C_d is the deflection amplification factor in Table 1617.6, which depends on the seismic-force-resisting system. As noted above, C_d is equal to 5 for this system.

The interstory drifts Δ computed from the δ_x are also contained in the table. For this structure that does not have plan irregularity Types 1a or 1b of Table 1616.5.1, the drift at story level x is determined by subtracting the design earthquake displacement at the center of mass at the bottom of the story from the design earthquake displacement at the center of mass at the top of the story (IBC 1617.4.6.1):

$$\Delta = \delta_x - \delta_{x-1}$$

The design story drifts Δ must not exceed the allowable story drift Δ_a from Table 1617.3 (IBC 1617.3). For Seismic Use Group I, $\Delta_a = 0.020 h_{sx}$ where h_{sx} is the story height below level x. Thus, for the 10-ft story heights, $\Delta_a = 0.020 \times 10 \times 12 = 2.40$ in. It is evident from Table 5-21 that for all stories, the lateral drifts obtained from the prescribed lateral forces in the N-S and E-W directions are less than the limiting values.

P-delta effects. As noted above, P-delta effects were automatically considered in the analysis using SAP2000.

5.5.3 Wind Load Analysis

5.5.3.1 Wind Forces

The analytical procedure (Method 2) of ASCE 6.5 is used to determine the wind forces.

Details on how to compute the wind forces in both the N-S and E-W directions are given in Section 5.2.3 of this publication. A summary of the design wind forces in both directions is contained in Table 5-22. Once again it is important to note that the wind directionality factor K_d has been taken equal to 1.0 (see Exception 1 in IBC 1605.2.1).

Table 5-21 Lateral Displacements and Interstory Drifts due to Seismic Forces in N-S and E-W Directions (SDC D)

Story	N-S Direction			E-W Direction		
	δ_{xe} (in.)	δ_x (in.)	Δ (in.)	δ_{xe} (in.)	δ_x (in.)	Δ (in.)
7	0.214	1.070	0.175	0.247	1.235	0.130
6	0.179	0.895	0.185	0.221	1.105	0.165
5	0.142	0.710	0.185	0.188	0.940	0.200
4	0.105	0.525	0.175	0.148	0.740	0.215
3	0.070	0.350	0.155	0.105	0.525	0.220
2	0.039	0.195	0.125	0.061	0.305	0.190
1	0.014	0.070	0.070	0.023	0.115	0.115

Table 5-22 Design Wind Forces in N-S and E-W Directions (V = 85 mph)

Level	Height above ground level, z (ft)	Total Design Wind Force N-S (kips)	Total Design Wind Force E-W (kips)
7	70	12.9	4.7
6	60	25.0	9.2
5	50	24.2	8.8
4	40	23.4	8.5
3	30	22.3	8.0
2	20	21.0	7.4
1	10	20.1	7.0
	Σ	148.9	53.6

5.5.3.2 Method of Analysis

Similar to the seismic analysis, a three-dimensional analysis of the building was performed in the N-S and E-W directions for the wind forces contained in Table 5-22 using SAP2000. The modeling assumptions utilized for the seismic analysis were also used for the wind analysis.

According to ASCE 6.5.12.3, main wind-force-resisting systems of buildings with mean roof height h greater than 60 ft must be designed for the full and partial wind load cases of Figure 6-9 (Cases 1 through 4). These four cases were considered in the three-dimensional analysis.

5.5.4 Design for Combined Load Effects

5.5.4.1 Load Combinations

The following load combinations are applicable:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)
4. $1.2D + f_1 L + 1.0E$ (Formula 16-5)
5. $0.9D + 1.0E$ (Formula 16-6)

where D, L, W, and E are the effects due to dead, live, wind, and seismic loads, respectively, and f_1 is equal to either 1.0 for places of public assembly, for live loads in excess of 100 psf, and for parking garage live load, or is equal to 0.5 for other live loads.

The seismic load effect E for use in Formula 16-5, which is the combined effect of horizontal and vertical earthquake-induced forces, is computed from Eq. 16-28 where the effects of gravity and seismic ground motion are additive:

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of horizontal seismic forces
ρ = redundancy coefficient determined in accordance with IBC 1617.2.2 for SDC D, E, or F

Similarly, E for use in Formula 16-6 is computed from Eq. 16-29 where the effects of gravity and seismic ground motion counteract:

$$E = \rho Q_E - 0.2 S_{DS} D$$

According to IBC 1617.2.2, the redundancy coefficient ρ, which shall not be less than 1.0 and need not exceed 1.5, is the largest of the values of ρ_i calculated at each story i from Equation 16-32:

$$\rho_i = 2 - \frac{20}{r_{\max_i} \sqrt{A_i}}$$

For shear walls:

$$r_{\max_i} = (\text{maximum wall shear} \times 10/\ell_w)/\text{total story shear}$$

ℓ_w = length of the wall in feet

In the N-S direction, the most heavily loaded shear wall has the largest shear force at its base. Therefore, for a maximum wall shear force of 172 kips and total story shear of 1,991 kips (see Table 5-20):

$$r_{max_1} = \frac{172 \times \dfrac{10}{28.33}}{1{,}826} = 0.03$$

$$\rho_{max} = 2 - \frac{20}{0.03\sqrt{140.67 \times 62}} < 0$$

Use $\rho = 1.0$.

Similar calculations in the E-W direction also result in $\rho = 1.0$.

Substituting $S_{DS} = 1.00g$ and $\rho = 1.0$ into the equations for E, and then substituting E along with $f_1 = 0.5$ into load combinations 4 and 5 above results in the following:

4a. $1.2D + 0.5L + 1.0Q_E + (0.2 \times 1.00)D = 1.4D + 0.5L + Q_E$

4b. $1.2D + 0.5L + 1.0Q_E - (0.2 \times 1.00)D = D + 0.5L + Q_E$

5a. $0.9D + 1.0Q_E + (0.2 \times 1.00)D = 1.1D + Q_E$

5b. $0.9D + 1.0Q_E - (0.2 \times 1.00)D = 0.7D + Q_E$

Values of Q_E are obtained from the structural analysis for both sidesway to the left and to the right based on the code-prescribed horizontal seismic forces acting on the building. Also, the special seismic load combinations in IBC 1605.4 are not applicable in this example.

5.5.4.2 Design of Shear Wall on Line 5

Comparing the seismic forces in Table 5-20 to the wind forces in Table 5-22, it is clear that the seismic forces will govern the design of the shear walls. Table 5-23 contains a summary of the design axial forces, bending moments, and shear forces in the stem segment of the wall at the base. Similarly, Table 5-24 contains a summary of the reactions in the flange segment of the wall at the base.

Table 5-23 Summary of Design Axial Forces, Bending Moments, and Shear Forces in Stem Segment at Base of Shear Wall on Line 5 (SDC D)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (*D*)	787	0	0
Live (*L*)	95	0	0
Seismic (Q_E)	± 302	± 4,388	± 172
Load Combinations			
$1.4D + 1.7L$	1,263	0	0
$1.4D + 0.5L + Q_E$	1,451	4,388	172
$0.7D + Q_E$	249	-4,388	-172

Table 5-24 Summary of Design Axial Forces, Bending Moments, and Shear Forces in Flange Segment at Base of Shear Wall on Line 5 (SDC D)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (*D*)	214	0	0
Live (*L*)	39	0	0
Seismic (Q_E)	± 5	± 3,508	± 185
Load Combinations			
$1.4D + 1.7L$	366	0	0
$1.4D + 0.5L + Q_E$	324	3,508	185
$0.7D + Q_E$	145	-3,508	-185

Wall Stem Segment.

Design for shear – reinforcement requirements. Special reinforced structural walls are to be provided with reinforcement in two orthogonal directions in the plane of the wall in accordance with ACI 21.6.2. The minimum reinforcement ratio in both directions is 0.0025, unless the design shear force is less than or equal to $A_{cv}\sqrt{f'_c}$, where A_{cv} is the gross area of concrete bounded by the web thickness and the length of wall in the direction of analysis. In such cases, minimum reinforcement in accordance with ACI 14.3 for ordinary walls must be provided. For the wall in this example, $A_{cv} = 8 \times 340 =$ 2,720 in.^{2}, so that

$$A_{cv}\sqrt{f'_c} = 2{,}720 \times \sqrt{4{,}000} / 1{,}000 = 172 \text{ kips} = V_u = 172 \text{ kips}$$

Therefore, minimum reinforcement per ACI 14.3 may be provided.

Two curtains of reinforcement are required in a wall when the in-plane factored shear force exceeds $2A_{cv}\sqrt{f_c'}$ = 2 × 172 = 344 kips. In this case, two curtains need not be provided, since 172 kips < 344 kips.

The minimum required reinforcement per foot of wall is:

Minimum vertical reinforcement area = 0.0012 × 12 × 8 = 0.12 in.^{2}/ft

Minimum horizontal reinforcement area = 0.0020 × 12 × 8 = 0.19 in.^{2}/ft

Try 1 layer of No. 6 bars spaced at 18 in. (A_s = 0.29 in.^{2}/ft) in the vertical and horizontal directions.

Design for shear – shear strength requirements. ACI Eq. 21-7 is used to determine nominal shear strength V_n of structural walls:

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

where $\alpha_c = 2$ for ratio of wall height to length h_w/ℓ_w = 70/28.33 = 2.5 > 2 (ACI 21.6.4.1).

For 1 curtain of No. 6 horizontal bars spaced at 18 in. ($\rho_n = 0.44/(8\times18) = 0.0031$):

$$\phi V_n = 0.85\times2{,}720\times[2\sqrt{4{,}000} + (0.0031\times60{,}000)]/1{,}000$$

$$= 723 \text{ kips} > V_u = 172 \text{ kips} \quad \text{O.K.}$$

where $\phi = 0.85$ for walls with $h_w/\ell_w > 2$ (ACI 9.3.4(a)). Note that V_n = 851 kips is less than the upper limit on shear strength, which is $8A_{cv}\sqrt{f_c'} = 8\times172 = 1{,}376$ kips (ACI 21.6.4.4). Therefore, use 1 curtain of No. 6 bars @ 18 in. on center in horizontal direction.

Reinforcement ratio ρ_v for the vertical reinforcement must not be less than ρ_n when $h_w/\ell_w \leq 2.0$ (ACI 21.6.4.3). Since h_w/ℓ_w = 2.5 > 2, use minimum reinforcement ratio of 0.0012.

Use 1 curtain of No. 6 bars spaced at 18 in. on center in the vertical direction (ρ_v = 0.0031 > 0.0012).

Design for axial force and bending. Figure 5-5 contains the interaction diagram of the stem segment of the wall. The stem is reinforced with 19-No. 6 vertical bars. As seen from the figure, the stem segment is adequate for the load combinations in Table 5-23.

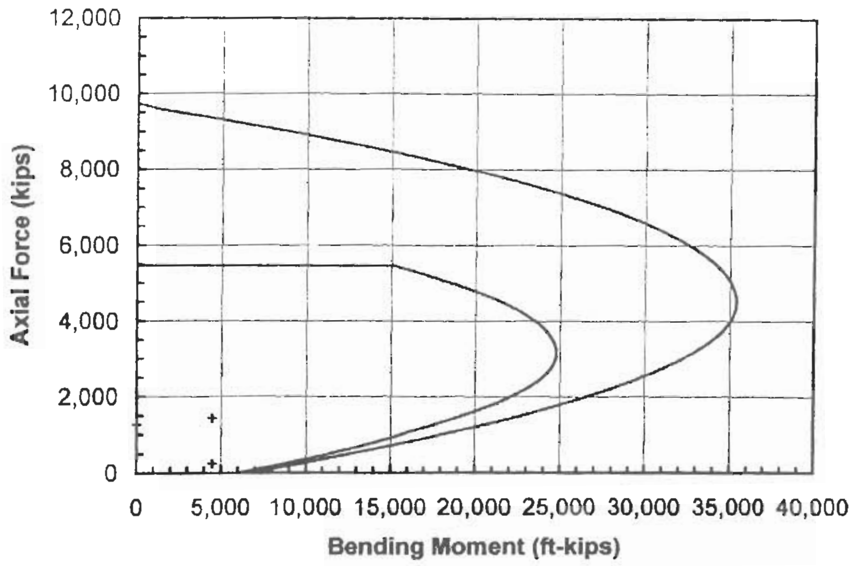


Figure 5-5 Design and Nominal Strength Interaction Diagrams for Stem Segment of Shear Wall Along Line 5 (SDC D)

Special boundary elements. The need for special boundary elements at the edges of structural walls is evaluated in accordance with ACI 21.6.6.2 or 21.6.6.3. The displacement-based approach in ACI 21.6.6.2 is utilized in this example. In this method, the wall is displaced at the top an amount equal to the expected design displacement; special boundary elements are required to confine the concrete when the strain in the extreme compression fiber of the wall exceeds a critical value. This method is applicable to walls or wall piers that are essentially continuous in cross-section over the entire height and designed to have one critical section for flexure and axial loads.

Compression zones are to be reinforced with special boundary elements where (Eq. 21-8):

$$c \geq \frac{\ell_w}{600(\delta_u / h_w)}, \quad \delta_u / h_w \geq 0.007$$

where c = distance from extreme compression fiber to the neutral axis per ACI 10.2.7 calculated for the factored axial force and nominal moment strength, consistent with the design displacement δ_u, resulting in the largest neutral axis depth

ℓ_w = length of entire wall or segment of wall considered in the direction of the shear force

δ_u = design displacement

= total lateral displacement expected for the design-basis earthquake as specified by the governing code

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

The lower limit on the quantity δ_u / h_w is specified to require moderate wall deformation capacity for stiff buildings.

In this example, ℓ_w = 28.33 ft = 340 in., h_w = 70 ft = 840 in., δ_u is equal to δ_x from Table 5-21, which is 1.07 in. at the top of the wall, and δ_u / h_w = 0.0013 < 0.007 (use 0.007). Therefore, special boundary elements are required if c is greater than or equal to 340/(600 × 0.007) = 81.0 in.

The distance c to be used in Eq. 21-8 is the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with the design displacement δ_u. From a strain compatibility analysis, the largest c is equal to 75.0 in. corresponding to a factored axial load of 1,451 kips and nominal moment strength of 22,263 ft-kips, which is less than 81.0 in. Therefore, special boundary elements are not required.

Where special boundary elements are not required according to ACI 21.6.6.2, the provisions of ACI 21.6.5.5 must be satisfied. These provisions require boundary transverse reinforcement for walls with moderate amounts of boundary longitudinal reinforcement to help prevent buckling of the longitudinal bars.

Boundary transverse reinforcement in accordance with ACI 21.4.4.1(c), 21.4.4.3, and 21.6.6.4(a) must be provided at the ends of walls where the longitudinal reinforcement ratio at the wall boundary is greater than $400/f_y$. In this example, the reinforcement layout is similar to that shown in the lower portion of ACI Figure R21.6.6.5, i.e., uniformly distributed vertical bars of the same size. The longitudinal reinforcement ratio ρ at the wall boundary is:

$$\rho = \frac{0.44}{8 \times 18} = 0.0031 < \frac{400}{60,000} = 0.0067$$

Thus, no special transverse reinforcement is required in the stem segment of the wall. However, according to ACI 21.6.6.5(b), the horizontal reinforcement in the stem terminating at the edges of the wall must have a standard hook engaging the edge reinforcement since V_u = 172 kips $= A_{cv}\sqrt{f_c'}$.

IBC 1620.1.7 requires that bearing walls and shear walls and their anchorage in buildings assigned to SDC B and above be designed for an out-of-plane force F_p that is the greater of 10 percent of the weight of the wall w_w or the value obtained from Eq. 16-63:

$$F_p = 0.40 I_E S_{DS} w_w$$

Out-of-plane seismic forces are usually computed for a 1-ft width of the wall length, assuming a uniformly distributed out-of-plane loading.

For the stem segment of the wall:

$$w_w = \frac{8}{12} \times 10 \times 0.15 = 1.0 \text{ kips/ft width of stem}$$

$$F_p = \begin{cases} 0.10 w_w = 0.10 \times 1.0 = 0.10 \text{ kips/ft} \\ 0.40 I_E S_{DS} w_w = 0.40 \times 1.0 \times 1.0 \times 1.0 = 0.4 \text{ kips/ft} \quad \text{(governs)} \end{cases}$$

Distributed load = 0.4/10 = 0.04 klf/ft

Assuming that the wall is simply supported at the floor levels, the maximum bending moment due to the out-of-plane seismic force is:

$$M_u = \frac{0.04 \times 10^2}{8} = 0.5 \text{ ft-kips/ft}$$

The design strength of a 1-ft wide segment of the stem reinforced with No. 6 vertical bars spaced at 18 in. is equal to $\phi M_n = 4.9$ ft-kips/ft > 0.5 ft-kips/ft O.K.

Splice length of reinforcement. Class B lap splices are utilized for the vertical bars in the stem. No splices are required for the No. 6 horizontal bars in the stem, since full length bars weigh approximately 1.502 × 28 = 42 lbs. and are easily installed.

For the No. 6 vertical bars in the stem, ℓ_d is determined from ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 0.8 for No. 6 and smaller bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} \frac{1}{2} \times 8 = 4.0 \text{ in.} \quad \text{(governs)} \\ \\ \frac{1}{2} \times 18 = 9.0 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index = 0

$$\frac{c + K_{tr}}{d_b} = \frac{4.0 + 0}{0.75} = 5.3 > 2.5, \text{ use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{0.8 \times 1.0 \times 1.0 \times 1.0}{2.5} = 22.8$$

$$\ell_d = 22.8 \times 0.75 = 17.1 \text{ in.} = 1.4 \text{ ft}$$

Class B splice length = $1.3\ell_d$ = 1.8 ft

Use a 1 ft-10 in. splice length for the No. 6 bars.

Wall Flange Segment.

Design for shear – reinforcement requirements. The minimum reinforcement ratio in both directions is 0.0025, unless the design shear force is less than or equal to $A_{cv}\sqrt{f'_c}$. In such cases, minimum reinforcement in accordance with ACI 14.3 for ordinary walls must be provided. For the wall in this example, A_{cv} = 8 × 264 = 2,112 in.^{2}, so that

$$A_{cv}\sqrt{f'_c} = 2{,}112 \times \sqrt{4{,}000} / 1{,}000 = 134 \text{ kips} < V_u = 185 \text{ kips}$$

Therefore, the minimum reinforcement ratio is 0.0025 and the maximum spacing is 18 in. (ACI 21.6.2.1).

Two curtains of reinforcement are required in a wall when the in-plane factored shear force exceeds $2A_{cv}\sqrt{f'_c}$ = 2 × 134 = 268 kips. In this case, two curtains need not be provided, since 185 kips < 268 kips.

The minimum required reinforcement in each direction per foot of wall is $0.0025 \times 8 \times 12 = 0.24$ in.^{2} Assuming No. 6 bars in one curtain, required spacing s is:

$$s = \frac{0.44}{0.24} \times 12 = 22 \text{ in.} > 18 \text{ in.}$$

Try 1 curtain of No. 6 bars spaced at 18 in.

Design for shear – shear strength requirements. ACI Eq. 21-7 is used to determine nominal shear strength V_n of structural walls:

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

where $\alpha_c = 2$ for ratio of wall height to length $h_w / \ell_w =$ 70/22 = 3.2 > 2 (ACI 21.6.4.1).

For 1 curtain of No. 6 horizontal bars spaced at 18 in. ($\rho_n = 0.44/(8 \times 18) = 0.0031$):

$$\phi V_n = 0.85 \times 2{,}112 \times [2\sqrt{4{,}000} + (0.0031 \times 60{,}000)]/1{,}000$$

$$= 561 \text{ kips} > V_u = 185 \text{ kips} \quad \text{O.K.}$$

where $\phi = 0.85$ for walls with $h_w / \ell_w > 2$ (ACI 9.3.4(a)). Note that V_n = 660 kips is less than the upper limit on shear strength, which is $8A_{cv}\sqrt{f_c'} = 8 \times 134 = 1{,}072$ kips (ACI 21.6.4.4). Therefore, use 1 curtain of No. 6 bars @ 18 in. on center in horizontal direction.

Reinforcement ratio ρ_v for the vertical reinforcement must not be less than ρ_n when $h_w / \ell_w \leq 2.0$ (ACI 21.6.4.3). Since h_w / ℓ_w = 3.2 > 2, use minimum reinforcement ratio of 0.0025.

Use 1 curtain of No. 6 bars spaced at 18 in. on center in the vertical direction (ρ_v = 0.0031 > 0.0025).

Design for axial force and bending. Figure 5-6 contains the interaction diagram of the flange segment of the wall. The flange is reinforced with 15-No. 6 vertical bars. As seen from the figure, the flange segment is adequate for the load combinations in Table 5-24.

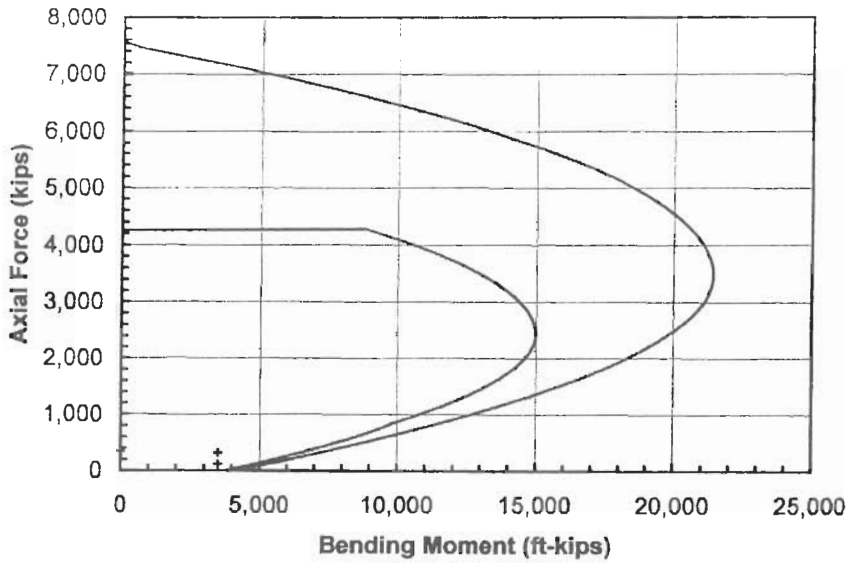


Figure 5-6 Design and Nominal Strength Interaction Diagrams for Flange Segment of Shear Wall Along Line 5 (SDC D)

Special boundary elements. Compression zones are to be reinforced with special boundary elements where (Eq. 21-8):

$$c \geq \frac{\ell_w}{600(\delta_u / h_w)}, \quad \delta_u / h_w \geq 0.007$$

In this example, ℓ_w = 22 ft = 264 in., h_w = 70 ft = 840 in., δ_u is equal to δ_x from Table 5-21, which is 1.235 in. at the top of the wall, and δ_u / h_w = 0.0015 < 0.007 (use 0.007). Therefore, special boundary elements are required if c is greater than or equal to 264/(600 × 0.007) = 62.9 in.

The distance c to be used in Eq. 21-8 is the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with the design displacement δ_u. From a strain compatibility analysis, the largest c is equal to 27.4 in. corresponding to a factored axial load of 324 kips and nominal moment strength of 7,200 ft-kips, which is less than 62.9 in. Therefore, special boundary elements are not required.

Boundary transverse reinforcement in accordance with ACI 21.4.4.1(c), 21.4.4.3, and 21.6.6.4(a) must be provided at the ends of walls where the longitudinal reinforcement ratio at the wall boundary is greater than $400/f_y$. The longitudinal reinforcement ratio ρ at the wall boundary is:

$$\rho = \frac{0.44}{8 \times 18} = 0.0031 < \frac{400}{60{,}000} = 0.0067$$

Thus, no special transverse reinforcement is required in the flange segment of the wall. However, according to ACI 21.6.6.5(b), the horizontal reinforcement in the flange terminating at the edges of the wall must have a standard hook engaging the edge reinforcement since $V_u = 185$ kips > $A_{cv}\sqrt{f_c'} = 134$ kips.

The vertical reinforcement is also adequate to resist the required out-of-plane seismic force prescribed in IBC 1620.1.7.

Splice length of reinforcement. Class B lap splices are utilized for the vertical bars in the flange. No splices would be required for the No. 6 horizontal bars in the flange, since full length bars weigh approximately $1.502 \times 22 = 33$ lbs. and are easily installed.

Class B splice length was determined above for the No. 6 bars in the stem and is equal to 1 ft-10 in.

Reinforcement details for the wall are shown in Figure 5-7.

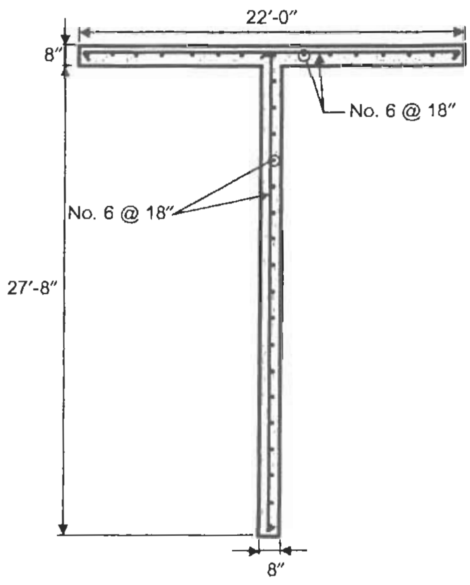


Figure 5-7 Reinforcement Details for Shear Wall Along Line 5 (SDC D)

5.6 DESIGN FOR SDC E

To illustrate the design requirements for Seismic Design Category (SDC) E, the residential building in Figure 5-1 is assumed to be located in Berkeley, CA.

5.6.1 Design Data

- Building Location: Berkeley, CA (zip code 94705)

- Material Properties

  Concrete: $f_c' = 4{,}000$ psi, $w_c = 150$ pcf
  Reinforcement: $f_y = 60{,}000$ psi

- Service Loads

  Live loads: roof = 20 psf
  floor = 100 psf for corridors, 40 psf elsewhere

  Superimposed dead loads: roof = 10 psf
  floor = 30 psf (20 psf permanent partitions + 10 psf ceiling, etc.)

- Seismic Design Data

  For zip code 94705: $S_S = 2.08g, S_1 = 0.92g$ [5.1]
  Site Class D (stiff soil profile; IBC Table 1615.1.1)
  For Category (Seismic Use Group) I occupancy, $I_E = 1.0$ (IBC Table 1604.5)

- Wind Design Data

  Basic wind speed = 85 mph (IBC Figure 1609)
  Exposure B (IBC 1609.4)
  For Category I occupancy, $I_W = 1.0$ (IBC Table 1604.5)

- Member Dimensions

  Joists: 12 + 4.5 × 5 + 30 (82 psf)
  Wall thickness: 10 in.

5.6.2 Seismic Load Analysis

5.6.2.1 Seismic Design Category (SDC)

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1-second period S_{M1} are determined from IBC Eqs. 16-16 and 16-17, respectively:

$$S_{MS} = F_a S_s = 1.0 \times 2.08 = 2.08g$$

$$S_{M1} = F_v S_1 = 1.5 \times 0.92 = 1.38g$$

where F_a and F_v are the values of site coefficients as a function of site class and of mapped spectral response acceleration at short periods S_S and at 1-second period S_1, respectively. The values of these coefficients for Site Class D in the equations above are contained in IBC Table 1615.1.2(1) for F_a and Table 1615.1.2(2) for F_v.

Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from Eqs. 16-18 and 16-19:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 2.08 = 1.39g$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 1.38 = 0.92g$$

Tables 1616.3(1) and 1616.3(2) are utilized to determine the SDC. From the footnote to Table 1616.3(1), Seismic Use Group I structures located on sites with $S_1 \geq 0.75g$ shall be assigned to SDC E when $S_{DS} \geq 0.50g$. Similarly, from the footnote to Table 1616.3(2), the SDC is E when $S_{D1} \geq 0.20g$. Thus, the SDC is E for this building.

5.6.2.2 Seismic Forces

Since the building does not have plan irregularity Type 1a, 1b, or 4 of Table 1616.5.1 or vertical irregularity Type 1a, 1b, 4, or 5 of Table 1616.5.2, it can be considered regular (IBC 1616.6.3). Note that structures assigned to SDC E or F are not permitted to have plan irregularity Type 1b and vertical irregularity Type 1b or 5 (IBC 1620.4.1). For this regular building that is less than 240 ft in height, Table 1616.6.3 allows the equivalent lateral force procedure in IBC 1617.4 to be used to compute the seismic base shear V (see Eq. 16-34):

$$V = C_s W$$

where C_s is the seismic response coefficient determined in accordance with IBC 1617.4.1.1 and W is the effective weight of the structure defined in IBC 1617.4. For the member sizes and superimposed dead loads given above, W = 10,919 kips (see Table 5-25 below).

In both directions, a bearing wall system with special reinforced concrete shear walls is utilized, which is permitted for structures assigned to SDC E with a height less than or equal to 160 ft (see IBC Table 1617.6 and IBC 1910.5). The response modification coefficient R = 5.5 and the deflection amplification factor C_d = 5 (IBC Table 1617.6).

Approximate period (T_a). The fundamental period of the building T is determined in accordance with IBC 1617.4.2. In lieu of a more exact analysis, an approximate fundamental period T_a is computed from Eq. 16-39:

Building height $h_n = 70$ ft

Building period coefficient $C_T = 0.02$

Period $T_a = C_T(h_n)^{3/4} = 0.020 \times (70)^{3/4} = 0.48$ sec

Seismic base shear (V). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.92}{\left(\frac{5.5}{1.0}\right)\times 0.48} = 0.349$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{1.39}{\left(\frac{5.5}{1.0}\right)} = 0.253$$

Also, C_s must not be less than the following value from Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 1.39 \times 1.0 = 0.061$$

For buildings assigned to SDC E or F and for those buildings for which $S_1 \geq 0.6g$, C_s shall not be taken less than that computed from Eq. 16-38:

$$C_s = \frac{0.5 S_1}{R/I_E} = \frac{0.5 \times 0.92}{5.5/1.0} = 0.084$$

In this case, the lower limit is 0.084 from Eq. 16-38.

Thus, the value of C_s from Eq. 16-35 governs so that the base shear V in the N-S and E-W directions is:

$$V = C_s W = 0.253 \times 10{,}919 = 2{,}763 \text{ kips}$$

Vertical distribution of seismic forces. The total base shear is distributed over the height of the building in conformance with Eqs. 16-41 and 16-42:

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

where F_x is the lateral force induced at level x, w_x and w_i are the portions of W assigned to levels x or i, and k is the distribution exponent defined in IBC 1617.4.3. For $T = 0.48$ sec, $k = 1.0$. The lateral forces F_x and the story shears V_x are contained in Table 5-25.

Table 5-25 Seismic Forces and Story Shears (SDC E)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
7	1,115	70	78,050	512	512
6	1,634	60	98,040	643	1,155
5	1,634	50	81,700	536	1,691
4	1,634	40	65,360	429	2,120
3	1,634	30	49,020	322	2,442
2	1,634	20	32,680	214	2,656
1	1,634	10	16,340	107	2,763
Σ	10,919		421,190	2,763	

5.6.2.3 Method of Analysis

A three-dimensional analysis of the building was performed in both the N-S and E-W directions for the seismic forces contained in Table 5-20 using SAP2000 [5.3]. In the model, rigid diaphragms were assigned at each floor level. The stiffness properties of the walls were input assuming cracked sections. In lieu of a more accurate analysis, the cracked section property of the shear walls was taken as $I_{eff} = 0.5 I_g$ where I_g is the gross moment of inertia of the section. P-delta effects were also considered in the analysis.

In accordance with IBC 1617.4.4.4, the center of mass was displaced each way from its actual location a distance equal to 5 percent of the building dimension perpendicular to the applied forces to account for accidental torsion in seismic design.

5.6.2.4 Story Drift and P-delta Effects

Story drift determination. Table 5-26 contains the displacements δ_{xe} in the N-S and E-W directions obtained from the 3-D static, elastic analysis using the design seismic forces, including accidental torsional effects. The table also contains the design earthquake displacement δ_x computed by IBC Eq. 16-46:

$$\delta_x = \frac{C_d \delta_{xe}}{I_E}$$

where C_d is the deflection amplification factor in Table 1617.6, which depends on the seismic-force-resisting system. As noted above, C_d is equal to 5 for this system.

Table 5-26 Lateral Displacements and Interstory Drifts due to Seismic Forces in N-S and E-W Directions (SDC E)

Story	N-S Direction			E-W Direction		
	δ_{xe} (in.)	δ_x (in.)	Δ (in.)	δ_{xe} (in.)	δ_x (in.)	Δ (in.)
7	0.255	1.275	0.210	0.344	1.720	0.180
6	0.213	1.065	0.220	0.308	1.540	0.230
5	0.169	0.845	0.220	0.262	1.310	0.280
4	0.125	0.625	0.210	0.206	1.030	0.300
3	0.083	0.415	0.185	0.146	0.730	0.305
2	0.046	0.230	0.145	0.085	0.425	0.265
1	0.017	0.085	0.085	0.032	0.160	0.160

The interstory drifts Δ computed from the δ_x are also contained in the table. For this structure that does not have plan irregularity Type 1a or 1b of Table 1616.5.1, the drift at story level x is determined by subtracting the design earthquake displacement at the center of mass at the bottom of the story from the design earthquake displacement at the center of mass at the top of the story (IBC 1617.4.6.1):

$$\Delta = \delta_x - \delta_{x-1}$$

The design story drifts Δ must not exceed the allowable story drift Δ_a from Table 1617.3 (IBC 1617.3). For Seismic Use Group I, $\Delta_a = 0.020 h_{sx}$ where h_{sx} is the story height below level x. Thus, for the 10-ft story heights, $\Delta_a = 0.020 \times 10 \times 12 =$ 2.40 in. It is evident from Table 5-26 that for all stories, the lateral drifts obtained from the prescribed lateral forces in the N-S and E-W directions are less than the limiting values.

P-delta effects. As noted above, P-delta effects were automatically considered in the analysis using SAP2000.

5.6.3 Wind Load Analysis

Wind forces in this example are the same as those computed in Section 5.5.3.1 of this publication, since the wind velocity and exposure are the same in the two examples. Thus, wind forces in the N-S and E-W directions are contained in Table 5-22.

5.6.4 Design for Combined Load Effects

5.6.4.1 Load Combinations

The following load combinations are applicable:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)
4. $1.2D + f_1 L + 1.0E$ (Formula 16-5)
5. $0.9D + 1.0E$ (Formula 16-6)

where D, L, W, and E are the effects due to dead, live, wind, and seismic loads, respectively, and f_1 is equal to either 1.0 for places of public assembly, for live loads in excess of 100 psf, and for parking garage live load, or is equal to 0.5 for other live loads.

The seismic load effect E for use in Formula 16-5, which is the combined effect of horizontal and vertical earthquake-induced forces, is computed from Eq. 16-28 where the effects of gravity and seismic ground motion are additive:

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of horizontal seismic forces
ρ = redundancy coefficient determined in accordance with IBC 1617.2.2 for SDC D, E, or F

Similarly, E for use in Formula 16-6 is computed from Eq. 16-29 where the effects of gravity and seismic ground motion counteract:

$$E = \rho Q_E - 0.2 S_{DS} D$$

According to IBC 1617.2.2, the redundancy coefficient ρ, which shall not be less than 1.0 and need not exceed 1.5, is the largest of the values of ρ_i calculated at each story i from Equation 16-32:

$$\rho_i = 2 - \frac{20}{r_{\max_i}\sqrt{A_i}}$$

For shear walls:

$r_{\max_i}$ = (maximum wall shear × $10/\ell_w$)/total story shear

ℓ_w = length of the wall in feet

In the N-S direction, the most heavily loaded shear wall has the largest shear force at its base. Therefore, for a maximum wall shear force of 243 kips and total story shear of 2,763 kips (see Table 5-25):

$$r_{\max_1} = \frac{243 \times \frac{10}{28.42}}{2,763} = 0.03$$

$$\rho_{\max} = 2 - \frac{20}{0.03\sqrt{140.83 \times 62}} < 0$$

Use $\rho = 1.0$.

Similar calculations in the E-W direction also result in $\rho = 1.0$.

Substituting $S_{DS} = 1.39g$ and $\rho = 1.0$ into the equations for E, and then substituting E along with $f_1 = 0.5$ into load combinations 4 and 5 above results in the following:

4a. $1.2D + 0.5L + 1.0Q_E + (0.2 \times 1.39)D = 1.48D + 0.5L + Q_E$

4b. $1.2D + 0.5L + 1.0Q_E - (0.2 \times 1.39)D = 0.92D + 0.5L + Q_E$

5a. $0.9D + 1.0Q_E + (0.2 \times 1.39)D = 1.18D + Q_E$

5b. $0.9D + 1.0Q_E - (0.2 \times 1.39)D = 0.62D + Q_E$

Values of Q_E are obtained from the structural analysis for both sidesway to the left and to the right based on the code-prescribed horizontal seismic forces acting on the building. Also, the special seismic load combinations in IBC 1605.4 are not applicable in this example.

5.6.4.2 Design of Shear Wall on Line 5

Table 5-27 contains a summary of the design axial forces, bending moments, and shear forces at the base of the stem segment of the wall. Similarly, Table 5-28 contains a summary of the reactions at the base of the flange segment of the wall.

Table 5-27 Summary of Design Axial Forces, Bending Moments, and Shear Forces in Stem Segment at Base of Shear Wall on Line 5 (SDC E)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	835	0	0
Live (L)	95	0	0
Seismic (Q_E)	± 398	± 6,539	± 243
Load Combinations			
$1.4D + 1.7L$	1,331	0	0
$1.48D + 0.5L + Q_E$	1,681	6,539	243
$0.62D + Q_E$	120	-6,539	-243

Table 5-28 Summary of Design Axial Forces, Bending Moments, and Shear Forces in Flange Segment at Base of Shear Wall on Line 5 (SDC E)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	253	0	0
Live (L)	39	0	0
Seismic (Q_E)	± 7	± 4,901	± 256
Load Combinations			
$1.4D + 1.7L$	421	0	0
$1.48D + 0.5L + Q_E$	401	4,901	256
$0.62D + Q_E$	150	-4,901	-256

Wall Stem Segment.

Design for shear – reinforcement requirements. Special reinforced structural walls are to be provided with reinforcement in two orthogonal directions in the plane of the wall in accordance with ACI 21.6.2. The minimum reinforcement ratio in both directions is 0.0025, unless the design shear force is less than or equal to $A_{cv}\sqrt{f'_c}$, where A_{cv} is the gross area of concrete bounded by the web thickness and the length of wall in the direction of analysis. In such cases, minimum reinforcement in accordance with ACI 14.3 for ordinary walls must be provided. For the wall in this example, $A_{cv} = 10 \times 341 = 3{,}410$ in.^{2}, so that

$$A_{cv}\sqrt{f'_c} = 3{,}410 \times \sqrt{4{,}000} / 1{,}000 = 216 \text{ kips} < V_u = 243 \text{ kips}$$

Therefore, the minimum reinforcement ratio is 0.0025 and the maximum spacing is 18 in. (ACI 21.6.2.1).

Two curtains of reinforcement are required in a wall when the in-plane factored shear force exceeds $2A_{cv}\sqrt{f_c'}$ = 2 × 216 = 432 kips. In this case, two curtains need not be provided, since 243 kips < 432 kips.

The minimum required reinforcement in each direction per foot of wall is 0.0025 × 10 × 12 = 0.30 in.^{2} Assuming No. 6 bars in one curtain, required spacing s is

$$s = \frac{0.44}{0.30} \times 12 = 17.6 \text{ in.} < 18 \text{ in.}$$

Try 1 curtain of No. 6 bars spaced at 16 in.

Design for shear – shear strength requirements. ACI Eq. 21-7 is used to determine nominal shear strength V_n of structural walls:

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

where $\alpha_c = 2$ for ratio of wall height to length h_w / ℓ_w = 70/28.42 = 2.5 > 2 (ACI 21.6.4.1).

For 1 curtain of No. 6 horizontal bars spaced at 16 in. ($\rho_n = 0.44/(10 \times 16) = 0.0028$):

$$\phi V_n = 0.85 \times 3{,}410 \times [2\sqrt{4{,}000} + (0.0028 \times 60{,}000)]/1{,}000$$

$$= 854 \text{ kips} > V_u = 243 \text{ kips} \quad \text{O.K.}$$

where $\phi = 0.85$ for walls with $h_w / \ell_w > 2$ (ACI 9.3.4(a)). Note that V_n = 1,005 kips is less than the upper limit on shear strength, which is $8A_{cv}\sqrt{f_c'} = 8 \times 216 = 1{,}728$ kips (ACI 21.6.4.4).Therefore, use 1 curtain of No. 6 bars @ 16 in. on center in horizontal direction.

Reinforcement ratio ρ_v for the vertical reinforcement must not be less than ρ_n when $h_w / \ell_w \leq 2.0$ (ACI 21.6.4.3). Since h_w / ℓ_w = 2.5 > 2, use minimum reinforcement ratio of 0.0025.

Use 1 curtain of No. 6 bars spaced at 16 in. on center in the vertical direction (ρ_v = 0.0028 > 0.0025).

Design for axial force and bending. Figure 5-8 contains the interaction diagram of the stem segment of the wall. The stem is reinforced with 21-No. 6 vertical bars. As seen from the figure, the stem segment is adequate for the load combinations in Table 5-27.

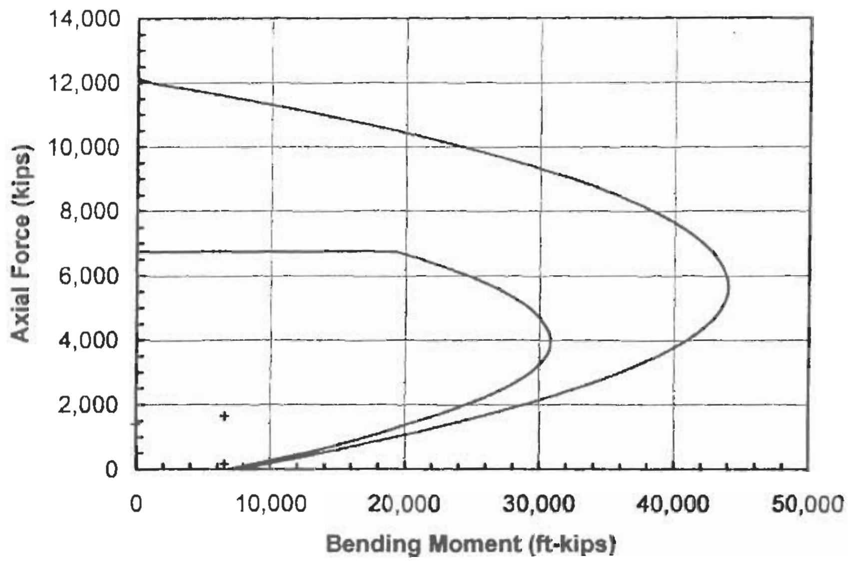


Figure 5-8 Design and Nominal Strength Interaction Diagrams for Stem Segment of Shear Wall Along Line 5 (SDC E)

Special boundary elements. The need for special boundary elements at the edges of structural walls is evaluated in accordance with ACI 21.6.6.2 or 21.6.6.3. The displacement-based approach in ACI 21.6.6.2 is utilized in this example. In this method, the wall is displaced at the top an amount equal to the expected design displacement; special boundary elements are required to confine the concrete when the strain in the extreme compression fiber of the wall exceeds a critical value. This method is applicable to walls or wall piers that are essentially continuous in cross-section over the entire height and designed to have one critical section for flexure and axial loads.

Compression zones are to be reinforced with special boundary elements where (Eq. 21-8):

$$c \geq \frac{\ell_w}{600(\delta_u / h_w)}, \qquad \delta_u / h_w \geq 0.007$$

where c = distance from extreme compression fiber to the neutral axis per ACI 10.2.7 calculated for the factored axial force and nominal moment strength, consistent with the design displacement δ_u, resulting in the largest neutral axis depth

ℓ_w = length of entire wall or segment of wall considered in the direction of the shear force

δ_u = design displacement

= total lateral displacement expected for the design-basis earthquake as specified by the governing code

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

The lower limit on the quantity δ_u / h_w is specified to require moderate wall deformation capacity for stiff buildings.

In this example, ℓ_w = 28.42 ft = 341 in., h_w = 70 ft = 840 in., δ_u is equal to δ_x from Table 5-26, which is 1.275 in. at the top of the wall, and $\delta_u / h_w = 0.0015 < 0.007$ (use 0.007). Therefore, special boundary elements are required if c is greater than or equal to $341/(600 \times 0.007) = 81.2$ in.

The distance c to be used in Eq. 21-8 is the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with the design displacement δ_u. From a strain compatibility analysis, the largest c is equal to 69.9 in. corresponding to a factored axial load of 1,681 kips and nominal moment strength of 26,002 ft-kips, which is less than 81.2 in. Therefore, special boundary elements are not required.

Where special boundary elements are not required according to ACI 21.6.6.2, the provisions of ACI 21.6.5.5 must be satisfied. These provisions require boundary transverse reinforcement for walls with moderate amounts of boundary longitudinal reinforcement to help prevent buckling of the longitudinal bars.

Boundary transverse reinforcement in accordance with ACI 21.4.4.1(c), 21.4.4.3, and 21.6.6.4(a) must be provided at the ends of walls where the longitudinal reinforcement ratio at the wall boundary is greater than $400/f_y$. In this example, the reinforcement layout is similar to that shown in the lower portion of ACI Figure R21.6.6.5, i.e., uniformly distributed vertical bars of the same size. The longitudinal reinforcement ratio ρ at the wall boundary is:

$$\rho = \frac{0.44}{10 \times 16} = 0.0028 < \frac{400}{60{,}000} = 0.0067$$

Thus, no special transverse reinforcement is required in the stem segment of the wall. However, according to ACI 21.6.6.5(b), the horizontal reinforcement in the stem terminating at the edges of the wall must have a standard hook engaging the edge reinforcement since $V_u = 243$ kips $> A_{cv}\sqrt{f_c'} = 216$ kips.

IBC 1620.1.7 requires that bearing walls and shear walls and their anchorage in buildings assigned to SDC B and above be designed for an out-of-plane force F_p that is the greater of 10 percent of the weight of the wall w_w or the value obtained from Eq. 16-63:

$$F_p = 0.40 I_E S_{DS} w_w$$

Out-of-plane seismic forces are usually computed for a 1-ft width of the wall length, assuming a uniformly distributed out-of-plane loading.

For the stem segment of the wall:

$$w_w = \frac{10}{12} \times 10 \times 0.15 = 1.25 \text{ kips/ft width of stem}$$

$$F_p = \begin{cases} 0.10 w_w = 0.10 \times 1.25 = 0.125 \text{ kips/ft} \\ 0.40 I_E S_{DS} w_w = 0.40 \times 1.0 \times 1.39 \times 1.25 = 0.695 \text{ kips/ft (governs)} \end{cases}$$

Distributed load = 0.695/10 = 0.07 klf/ft

Assuming that the wall is simply supported at the floor levels, the maximum bending moment due to the out-of-plane seismic force is:

$$M_u = \frac{0.07 \times 10^2}{8} = 0.9 \text{ ft-kips/ft}$$

The design strength of a 1-ft wide segment of the stem reinforced with No. 6 vertical bars spaced at 16 in. is equal to $\phi M_n = 7.1$ ft-kips/ft > 0.9 ft-kips/ft O.K.

Splice length of reinforcement. Class B lap splices are utilized for the vertical bars in the stem. No splices are required for the No. 6 horizontal bars in the stem, since full length bars weigh approximately 1.502 × 28 = 42 lbs. and are easily installed.

For the No. 6 vertical bars in the stem, ℓ_d is determined from ACI Eq. (12-1):

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 0.8 for No. 6 and smaller bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} \frac{1}{2} \times 10 = 5.0 \text{ in.} \quad \text{(governs)} \\ \\ \frac{1}{2} \times 16 = 8.0 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index = 0

$$\frac{c + K_{tr}}{d_b} = \frac{5.0 + 0}{0.75} = 6.7 > 2.5, \text{ use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{0.8 \times 1.0 \times 1.0 \times 1.0}{2.5} = 22.8$$

$$\ell_d = 22.8 \times 0.75 = 17.1 \text{ in.} = 1.4 \text{ ft}$$

Class B splice length = $1.3\ell_d$ = 1.8 ft

Use a 1 ft-10 in. splice length for the No. 6 bars.

Wall Flange Segment.

Design for shear – reinforcement requirements. The minimum reinforcement ratio in both directions is 0.0025, unless the design shear force is less than or equal to $A_{cv}\sqrt{f'_c}$. In such cases, minimum reinforcement in accordance with ACI 14.3 for ordinary walls must be provided. For the wall in this example, A_{cv} = 10 × 264 = 2,640 in.^{2}, so that

$$A_{cv}\sqrt{f'_c} = 2{,}640 \times \sqrt{4{,}000} / 1{,}000 = 167 \text{ kips} < V_u = 256 \text{ kips}$$

Therefore, the minimum reinforcement ratio is 0.0025 and the maximum spacing is 18 in. (ACI 21.6.2.1).

Two curtains of reinforcement are required in a wall when the in-plane factored shear force exceeds $2A_{cv}\sqrt{f'_c}$ = 2 × 167 = 334 kips. In this case, two curtains need not be provided, since 256 kips < 334 kips.

The minimum required reinforcement in each direction per foot of wall is $0.0025 \times 10 \times 12 = 0.30$ in.^{2} Assuming No. 6 bars in one curtain, required spacing s is

$$s = \frac{0.44}{0.30} \times 12 = 17.6 \text{ in.} < 18 \text{ in.}$$

Try 1 curtain of No. 6 bars spaced at 16 in.

Design for shear – shear strength requirements. ACI Eq. 21-7 is used to determine nominal shear strength V_n of structural walls:

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

where $\alpha_c = 2$ for ratio of wall height to length $h_w / \ell_w =$ 70/22 = 3.2 > 2 (ACI 21.6.4.1).

For 1 curtain of No. 6 horizontal bars spaced at 16 in. ($\rho_n = 0.44/(10 \times 16) = 0.0028$):

$$\phi V_n = 0.85 \times 2{,}640 \times [2\sqrt{4{,}000} + (0.0028 \times 60{,}000)] / 1{,}000$$

$$= 661 \text{ kips} > V_u = 256 \text{ kips} \qquad \text{O.K.}$$

where $\phi = 0.85$ for walls with $h_w / \ell_w > 2$ (ACI 9.3.4(a)). Note that V_n = 778 kips is less than the upper limit on shear strength, which is $8A_{cv}\sqrt{f_c'} = 8 \times 167 = 1{,}336$ kips (ACI 21.6.4.4).Therefore, use 1 curtain of No. 6 bars @ 16 in. on center in horizontal direction.

Reinforcement ratio ρ_v for the vertical reinforcement must not be less than ρ_n when $h_w / \ell_w \leq$ 2.0 (ACI 21.6.4.3). Since $h_w / \ell_w =$ 3.2 > 2, use minimum reinforcement ratio of 0.0025.

Use 1 curtain of No. 6 bars spaced at 16 in. on center in the vertical direction (ρ_v = 0.0028 > 0.0025).

Design for axial force and bending. Figure 5-9 contains the interaction diagram of the flange segment of the wall. The flange is reinforced with 17-No. 6 vertical bars. As seen from the figure, the flange segment is adequate for the load combinations in Table 5-28.

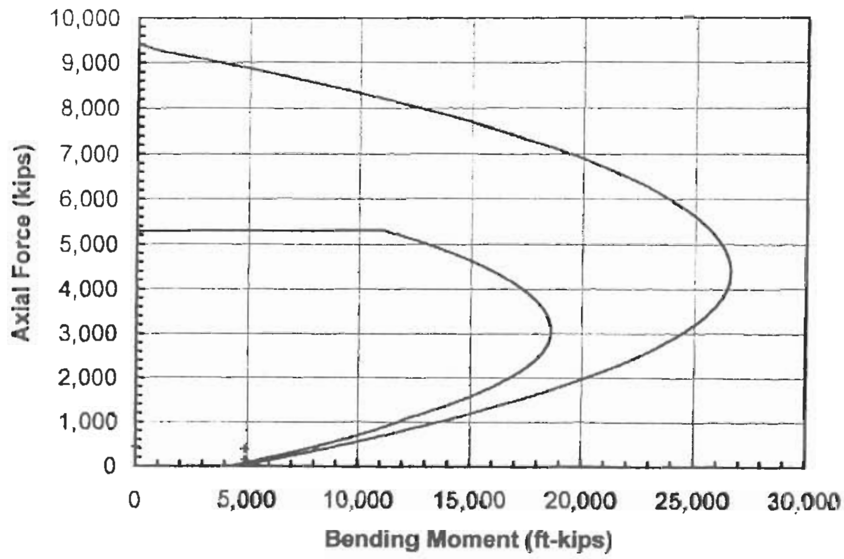


Figure 5-9 Design and Nominal Strength Interaction Diagrams for Flange Segment of Shear Wall Along Line 5 (SDC E)

Special boundary elements. Compression zones are to be reinforced with special boundary elements where (Eq. 21-8):

$$c \geq \frac{\ell_w}{600(\delta_u / h_w)}, \quad \delta_u / h_w \geq 0.007$$

In this example, ℓ_w = 22 ft = 264 in., h_w = 70 ft = 840 in., δ_u is equal to δ_x from Table 5-26, which is 1.72 in. at the top of the wall, and δ_u / h_w = 0.002 < 0.007 (use 0.007). Therefore, special boundary elements are required if c is greater than or equal to 264/(600 × 0.007) = 62.9 in.

The distance c to be used in Eq. 21-8 is the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with the design displacement δ_u. From a strain compatibility analysis, the largest c is equal to 26.1 in. corresponding to a factored axial load of 401 kips and nominal moment strength of 8,544 ft-kips, which is less than 62.9 in. Therefore, special boundary elements are not required.

Boundary transverse reinforcement in accordance with ACI 21.4.4.1(c), 21.4.4.3, and 21.6.6.4(a) must be provided at the ends of walls where the longitudinal reinforcement ratio at the wall boundary is greater than $400/f_y$. The longitudinal reinforcement ratio ρ at the wall boundary is:

$$\rho = \frac{0.44}{10 \times 16} = 0.0028 < \frac{400}{60{,}000} = 0.0067$$

Thus, no special transverse reinforcement is required in the flange segment of the wall. However, according to ACI 21.6.6.5(b), the horizontal reinforcement in the flange terminating at the edges of the wall must have a standard hook engaging the edge reinforcement since $V_u = 256$ kips > $A_{cv}\sqrt{f'_c} = 167$ kips.

The vertical reinforcement is also adequate to resist the required out-of-plane seismic force prescribed in IBC 1620.1.7.

Splice length of reinforcement. Class B lap splices are utilized for the vertical bars in the flange. No splices are required for the No. 6 horizontal bars in the flange, since full length bars weigh approximately $1.502 \times 22 = 33$ lbs. and are easily installed.

Class B splice length was determined above for the No. 6 bars in the stem and is equal to 1 ft-10 in.

Reinforcement details for the wall are shown in Figure 5-10.

5.7 REFERENCES

5.1 International Conference of Building Officials, *Code Central – Earthquake Spectral Acceleration Maps*, prepared in conjunction with U.S. Geological Survey; Building Seismic Safety Council; Federal Emergency Management Agency; and E.V. Leyendecker, A.D. Frankel, and K.S. Rukstales, Whittier, CA (CD-ROM).

5.2 American Society of Civil Engineers, *ASCE Standard Minimum Design Loads for Buildings and Other Structures*, ASCE 7-98, Reston, VA, 2000.

5.3 Computers and Structures, Inc., *SAP2000 – Integrated Finite Element Analysis and Design of Structures*, Berkeley, CA, 1999.

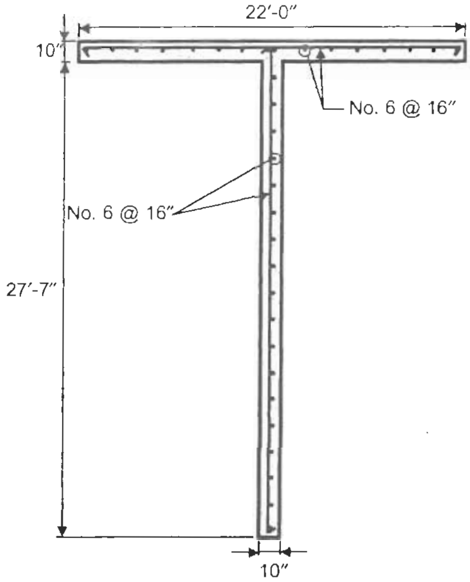


Figure 5-10 Reinforcement Details for Shear Wall Along Line 5 (SDC E)

CHAPTER 6

PARKING STRUCTURE WITH BUILDING FRAME SYSTEM

6.1 INTRODUCTION

Building frame systems have an essentially complete space frame that resists the gravity loads and shear walls that resist the lateral forces. It is essential that the deformation compatibility requirements of IBC 1617.6.4.3 be satisfied for building frame systems assigned to SDC D and above. These provisions recognize that members that are not designated to be part of the seismic-force-resisting system deform with the members of the seismic-force-resisting system when subjected to the code-specified earthquake design forces, since all of the components are connected at every floor level through the floor systems. Thus, members that are not part of the seismic-force-resisting system must be able to continue to carry gravity loads when subjected to earthquake-induced lateral displacements.

This chapter illustrates the design and detailing of structural walls that are part of a building frame system in a parking structure assigned to SDC B, C, and D.

6.2 DESIGN FOR SDC B

6.2.1 Design Data

A typical plan and elevation of a 4-story precast parking structure is shown in Figure 6-1. The computation of wind and seismic forces according to the 2000 IBC is illustrated below. Typical precast walls are designed and detailed for combined effects of gravity, wind, and seismic forces.

A building frame system with ordinary reinforced concrete shear walls may be used for structures assigned to SDC B without any limitations according to IBC Table 1617.6. This type of system is utilized in this example.

- Building Location: Atlanta, GA (zip code 30350)

- Material Properties

  Concrete: $f_c' = 5{,}000$ psi, $w_c = 150$ pcf
  Reinforcement: $f_y = 60{,}000$ psi

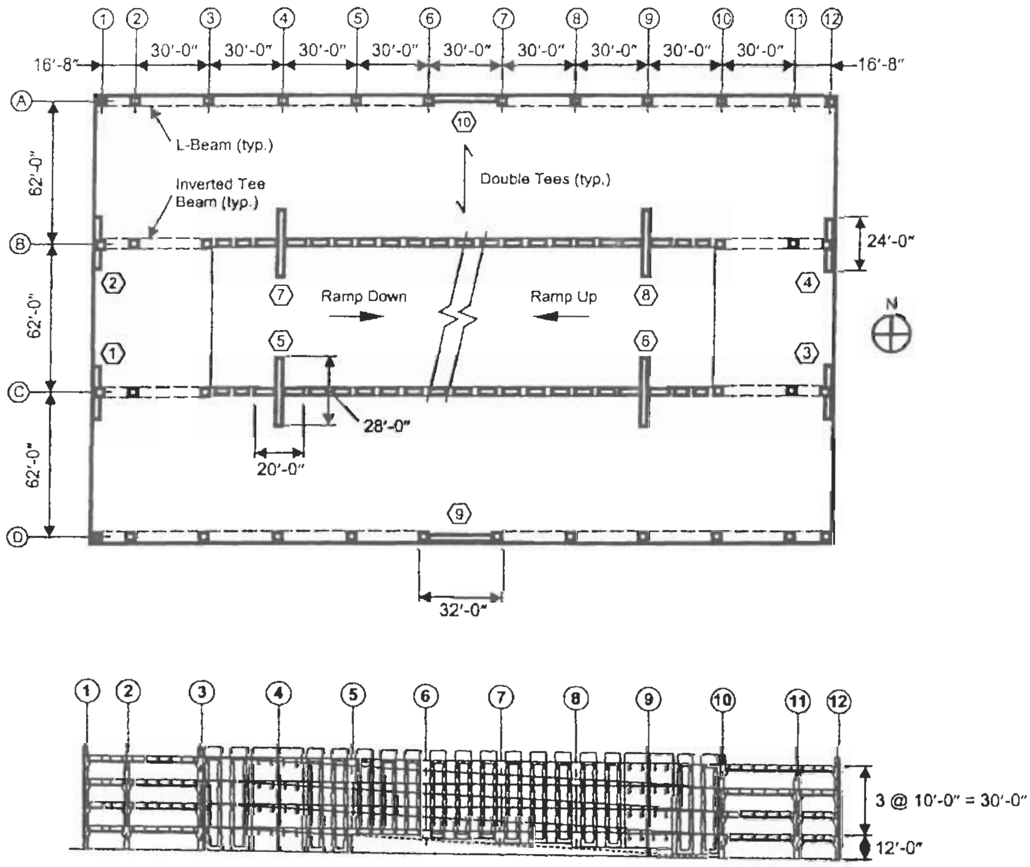


Figure 6-1 Typical Plan and Elevation of Precast Parking Structure (SDC B)

- Service Loads

  Live loads: 50 psf (IBC Table 1607.1)
  Dead loads: 90 psf (weight of structural members + superimposed dead loads)

- Seismic Design Data

  For zip code 30350: $S_S = 0.276g, S_1 = 0.117g$ [6.1]
  Site Class C (very dense soil / soft rock soil profile; IBC Table 1615.1.1)
  For Category (Seismic Use Group) I occupancy, $I_E = 1.0$ (IBC Table 1604.5)

- Wind Design Data

  Basic wind speed = 90 mph (IBC Figure 1609)

Exposure B (IBC 1609.4)
For Category I occupancy, $I_W = 1.0$ (IBC Table 1604.5)

- Member Dimensions

Wall thickness: 10 in.

6.2.2 Seismic Load Analysis

6.2.2.1 Seismic Design Category (SDC)

Analysis procedures for seismic design are given in IBC 1616.6. The appropriate procedure to use depends on the Seismic Design Category (SDC), which is determined in accordance with IBC 1616.3. Structures are assigned to a SDC based on their Seismic Use Group and the design spectral response acceleration parameters S_{DS} and S_{D1}. These parameters can be computed from Eqs. 16-18 and 16-19 in IBC 1615.1.3 or can be obtained from the provisions of IBC 1615.2.5 where site-specific procedures are used as required or permitted by IBC 1615.

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1-second period S_{M1} are determined from IBC Eqs. 16-16 and 16-17, respectively:

$$S_{MS} = F_a S_S = 1.2 \times 0.276 = 0.33g$$

$$S_{M1} = F_v S_1 = 1.68 \times 0.117 = 0.20g$$

where F_a and F_v are the values of site coefficients as a function of site class and of mapped spectral response acceleration at short periods S_S and at 1-second period S_1, respectively. The values of these coefficients for Site Class C in the equations above are contained in IBC Table 1615.1.2(1) for F_a and Table 1615.1.2(2) for F_v. Straight-line interpolation was used to determine F_v.

Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from Eqs. 16-18 and 16-19:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 0.33 = 0.22g$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.20 = 0.13g$$

Tables 1616.3(1) and 1616.3(2) are utilized to determine the SDC. From Table 1616.3(1) for Seismic Use Group I and $S_{DS} = 0.22g$, the SDC is B. Similarly, from Table 1616.3(2), the SDC is B for $S_{D1} = 0.13g$. Thus, the SDC is B for this structure.

6.2.2.2 Seismic Forces

According to IBC 1616.6.2, the equivalent lateral force procedure in IBC 1617.4 may be used to compute the seismic base shear V for structures assigned to SDC B. In a given direction, V is determined from Eq. 16-34:

$$V = C_s W$$

where C_s is the seismic response coefficient determined in accordance with IBC 1617.4.1.1 and W is the effective seismic weight of the structure defined in IBC 1617.4. For this example, W = 22,968 kips (see Table 6-1 below).

In both directions, a building frame system with ordinary reinforced concrete shear walls is utilized, which is permitted for structures assigned to SDC B without any limitations (see IBC Table 1617.6 and IBC 1910.3). The response modification coefficient $R = 5$ and the deflection amplification factor $C_d = 4.5$ (IBC Table 1617.6).

Approximate period (T_a). The fundamental period of the building T is determined in accordance with IBC 1617.4.2. In lieu of a more exact analysis, an approximate fundamental period T_a is computed from Eq. 16-39:

Structure height $h_n = 42$ ft

Period coefficient $C_T = 0.02$

Period $T_a = C_T (h_n)^{3/4} = 0.020 \times (42)^{3/4} = 0.33$ sec

Seismic base shear (V). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.13}{\left(\frac{5}{1.0}\right)\times 0.33} = 0.079$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{0.22}{\left(\frac{5.0}{1.0}\right)} = 0.044$$

Also, C_s must not be less than the following value from Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 0.22 \times 1.0 = 0.010$$

Thus, the value of C_s from Eq. 16-35 governs so that the base shear V in the N-S and E-W directions is:

$$V = C_s W = 0.044 \times 22{,}968 = 1{,}011 \text{ kips}$$

Vertical distribution of seismic forces. The total base shear is distributed over the height of the structure in conformance with Eqs. 16-41 and 16-42:

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

where F_x is the lateral force induced at level x, w_x and w_i are the portions of W assigned to levels x or i, and k is the distribution exponent defined in IBC 1617.4.3. For T= 0.33 sec, k = 1.0. The lateral forces F_x and the story shears V_x are contained in Table 6-1.

Table 6-1 Seismic Forces and Story Shears (SDC B)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
4	5,722	42	240,324	389	389
3	5,910	32	189,120	306	695
2	5,912	22	130,064	211	906
1	5,424	12	65,088	105	1,011
Σ	22,968		624,596	1,011	

6.2.2.3 Method of Analysis

A three-dimensional analysis of the structure was performed in the E-W direction for the seismic forces contained in Table 6-1 using the computer program *BRDLAT* [6.2]. The stiffness properties of the members were input assuming cracked sections. In lieu of a more accurate analysis, the following cracked section properties were used:

- Beams: $I_{eff} = 0.7 I_g$

- Columns: $I_{eff} = 0.5I_g$
- Shear walls: $I_{eff} = 0.4I_g$

where I_g is the gross moment of inertia of the section.

In accordance with IBC 1617.4.4.4, the center of mass was displaced each way from its actual location a distance equal to 5 percent of the structure dimension perpendicular to the applied forces to account for accidental torsion in seismic design. Torsional effects need not be amplified, since the structure is assigned to SDC B (IBC 1617.4.4.5).

6.2.2.4 Story Drift and P-delta Effects

Story drift determination. Table 6-2 contains the displacements δ_{xe} in the E-W direction obtained from the 3-D static, elastic analysis using the design seismic forces, including accidental torsional effects. The table also contains the design earthquake displacement δ_x computed by IBC Eq. 16-46:

$$\delta_x = \frac{C_d \delta_{xe}}{I_E}$$

where C_d is the deflection amplification factor in Table 1617.6, which depends on the seismic-force-resisting system. As noted above, C_d is equal to 4.5 for this system.

Table 6-2 Lateral Displacements and Interstory Drifts due to Seismic Forces in E-W Direction (SDC B)

Story	δ_{xe} (in.)	δ_x (in.)	Δ (in.)
4	0.60	2.70	0.90
3	0.40	1.80	0.81
2	0.22	0.99	0.63
1	0.08	0.36	0.36

The interstory drifts Δ computed from the δ_x are also contained in the table, where the drift at story level x is determined by subtracting the design earthquake displacement at the center of mass at the bottom of the story from the design earthquake displacement at the center of mass at the top of the story (IBC 1617.4.6.1):

$$\Delta = \delta_x - \delta_{x-1}$$

The design story drifts Δ must not exceed the allowable story drift Δ_a from Table 1617.3 (IBC 1617.3). For Seismic Use Group I, $\Delta_a = 0.025h_{sx}$ for structures 4 stories or less in height where h_{sx} is the story height below level x. Thus, for the 10-ft story heights, $\Delta_a = 0.025 \times 10 \times 12 = 3.0$ in., and for the 12-ft story height at the first level, $\Delta_a = 3.6$ in. It is evident from Table 6-2 that for all stories, the lateral drifts obtained from the prescribed lateral forces in the E-W direction are less than the limiting values.

Similar calculations for seismic forces in the N-S direction also show that the lateral drifts are less than the allowable values.

P-delta effects. P-delta effects need not be considered when the stability coefficient θ determined by Equation 16-47 is less than or equal to 0.10 (IBC 1617.4.6.2):

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d}$$

where P_x = total unfactored vertical design load at and above level x

Δ = design story drift occurring simultaneously with V_x

V_x = seismic shear force acting between level x and $x - 1$

h_{sx} = story height below level x

C_d = deflection amplification factor

The stability coefficient θ must not exceed θ_{max} determined from Eq. 16-48:

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25$$

where β is the ratio of shear demand to shear capacity between level x and $x - 1$, which may be taken equal to 1.0 when it is not calculated.

Table 6-3 contains the calculations for the E-W direction. It is clear that P-delta effects need not be considered at any of the floor levels. Note that θ_{max} is equal to 0.111 in the E-W direction using $\beta = 1.0$. Similar calculations for the N-S direction show that P-delta effects may also be neglected.

Table 6-3 P-delta Effects (SDC B)

Level	h_{sx} (ft)	P_x (kips)	V_x (kips)	Δ (in.)	θ
4	10	8,543	389	0.90	0.0366
3	10	16,710	695	0.81	0.0361
2	10	24,879	906	0.63	0.0320
1	12	32,559	1,011	0.36	0.0179

6.2.2.5 Overturning

According to IBC 1617.4.5, the structure must be designed to resist overturning effects caused by the seismic forces distributed over the height of the structure. The increment of overturning moment in any story is to be distributed to the various vertical-force-resisting elements in the same proportion as the distribution of the horizontal shears to those elements.

The overturning moment M_x at level x is determined from Equation 16-45:

$$M_x = \tau \sum_{i=x}^{n} F_i (h_i - h_x)$$

where F_i = portion of the seismic base shear V induced at level i

h_i, h_x = height from the base to level i or x

τ = overturning moment reduction factor
= 1.0 for top 10 stories
= 0.8 for the 20th story from the top and below
= value between 1.0 and 0.8 determined by straight line interpolation for stories between the 20th and 10th stories below the top

Tables 6-4 and 6-5 contain summaries of the overturning and resisting moments at the base of the shear walls in both the N-S and E-W directions, respectively. When computing the resisting moments, the dead load was multiplied by the factor $(0.9 - 0.2S_{DS}) = 0.9 - (0.2 \times 0.22) = 0.86$ in accordance with Equation 16-29, which applies when the effects of gravity and seismic ground motion counteract (IBC 1617.1.1). For all walls, the ratio of resisting moment to overturning moment is greater than 1.

Table 6-4 Resisting and Overturning Moments at Base of Shear Walls for Seismic Forces in the N-S Direction (SDC B)

Wall No.	Length (ft)	Dead Load* (kips)	Resisting Moment M_r † (ft-kips)	Overturning Moment M_x (ft-kips)	M_r/M_x
1	24	437	4,510	2,789	1.6
2	24	437	4,510	2,789	1.6
3	24	437	4,510	2,822	1.6
4	24	437	4,510	2,822	1.6
5	28	645	7,766	4,276	1.8
6	28	577	6,947	5,346	1.3
7	28	645	7,766	4,276	1.8
8	28	577	6,947	5,346	1.3

*Dead load = weight of wall + dead load tributary to wall.

† M_r = 0.86 × dead load × (length/2)

Table 6-5 Resisting and Overturning Moments at Base of Shear Walls for Seismic Forces in the E-W Direction (SDC B)

Wall No.	Length (ft)	Dead Load* (kips)	Resisting Moment M_r † (ft-kips)	Overturning Moment M_x (ft-kips)	M_r / M_x
5	20	645	5,547	3,186	1.7
6	20	577	4,962	4,053	1.2
7	20	645	5,547	3,191	1.7
8	20	577	4,962	4,060	1.2
9	32	861	11,847	8,145	1.5
10	32	861	11,847	8,176	1.5

*Dead load = weight of wall + dead load tributary to wall

† $M_r = 0.86 \times$ dead load $\times$ (length/2)

6.2.3 Wind Load Analysis

Wind forces are determined in accordance with the analytical procedure (Method 2) given in ASCE 6.5 [6.3].

Design Procedure.

The design procedure outlined in Section 6.5.3 of ASCE 7 is used to determine the wind forces on the structure in the E-W direction.

1. Basic wind speed, V, and wind directionality factor, K_d. Both quantities are determined in accordance with ASCE 6.5.4. As noted above, V is equal to 90 mph for Atlanta according to IBC Figure 1609 or ASCE Figure 6-1.

The wind directionality factor K_d is equal to 0.85 for main wind-force-resisting systems per ASCE Table 6-6 when load combinations specified in Sections 2.3 and 2.4 are used. Note that these load combinations are essentially the same as those in Sections 1605.2 and 1605.3 of the 2000 IBC. It is important to note exception 1 to IBC 1605.2.1, Basic load combinations: load combinations of ACI 9.2 shall be used for concrete structures where combinations do not include seismic forces. The load factors in the ACI 318 combinations are different than those in ASCE 7 and the IBC. The exception goes on to state that for concrete structures designed for wind in accordance with ASCE 7, wind forces are to be divided by the directionality factor. Thus, in the following computations, instead of multiplying and then subsequently dividing the external wind pressures/forces by 0.85, K_d is taken equal to 1.0.

2. Importance factor, I_W. As noted above, I_W is equal to 1.0 for Category I occupancy according to IBC Table 1604.5 and Category II occupancy according to ASCE Table 1-1 (note that IBC Category I and ASCE 7 Category II are the same).

3. Velocity pressure exposure coefficient, K_z. According to ASCE 6.5.6.4, values of K_z are to be determined from ASCE Table 6-5. In lieu of linear interpolation, K_z may be calculated at any height z above ground level from the equations given at the bottom of ASCE Table 6-5:

$$K_z = \begin{cases} 2.01\left(\dfrac{15}{z_g}\right)^{2/\alpha} & \text{for } z < 15 \text{ ft} \\ 2.01\left(\dfrac{z}{z_g}\right)^{2/\alpha} & \text{for } 15 \text{ ft} \le z \le z_g \end{cases}$$

where α = 3-second gust speed power law exponent from ASCE Table 6-4
= 7.0 for Exposure B
z_g = nominal height of the atmospheric boundary layer from ASCE Table 6-4
= 1,200 ft for Exposure B

Values of K_z are summarized in Table 6-6 at the various heights for the example structure.

Table 6-6 Velocity Pressure Exposure Coefficient K_z

Level	Height above ground level, z (ft)	K_z
4	42	0.771
3	32	0.714
2	22	0.641
1	12	0.575

4. Topographic factor, K_{zt}. The topographic factor is to be determined in accordance with ASCE 6.5.7, Equation 6-1. Assuming the example structure is situated on level ground and not on a hill, ridge, or escarpment, K_{zt} is equal to 1.

5. Gust effect factors, G and G_f. Effects due to wind gust depend on whether a structure is rigid or flexible (ASCE 6.5.8). A rigid structure has a fundamental natural frequency n_1 greater than or equal to 1 Hz, while a flexible structure has a fundamental natural frequency less than 1 Hz (ASCE 6.2).

In lieu of a more exact method, the approximate fundamental period T_a = 0.33 sec determined in Section 6.2.2.2 of this publication is used. The natural frequency is computed by taking the inverse of the period: $n_1 = 1/0.33 = 3.0 \text{ Hz}$.

Since $n_1 > 1.0$ Hz, the building is considered rigid, and G may be taken equal to 0.85 or may be calculated by Equation 6-2 (ASCE 6.5.8.1). For simplicity, G is taken as 0.85.

6. Enclosure classification. For wind design, the definition of an open building requires that 80% of the area of each external wall be open (ASCE 6.2). The example structure does not have that degree of openness. It also does not comply with the conditions of a partially enclosed building, even though the structure is approximately 30% open on all sides. Thus, the structure is enclosed per ASCE 6.5.9.4 or IBC 1609.2.

7. Internal pressure coefficient, GC_{pi}. According to ASCE 6.5.11.1, internal pressure coefficients are to be determined from ASCE Table 6-7 based on enclosure classification. For an enclosed building, $GC_{pi} = \pm 0.18$.

8. External pressure coefficients, C_p. External pressure coefficients for main wind-force-resisting systems are given in ASCE Figure 6-3 for this example structure. For wind in the E-W direction:

Windward wall: $C_p = 0.8$

Leeward wall (L/B = 303.33/186 = 1.6): $C_p = -0.38$

9. Velocity pressure, q_z. The velocity pressure at height z is determined from Equation 6-13 in ASCE 6.5.10:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I$$

where all terms have been defined previously. Table 6-7 contains a summary of the velocity pressures for the example building.

Table 6-7 Velocity Pressure q_z (V = 90 mph)

Level	Height above ground level, z (ft)	K_z	q_z (psf)
4	42	0.771	16.0
3	32	0.714	14.8
2	22	0.641	13.3
1	12	0.575	11.9

10. Design wind pressure, p. Design wind pressures on the main wind-force-resisting systems of enclosed and partially enclosed buildings are determined in accordance with ASCE 6.5.12. For rigid buildings of all heights, design wind pressures are calculated from Equation 6-15:

$$p = qGC_p - q_i(GC_{pi})$$

In addition to parapets, it is common for spandrel beams and interior ramp walls to project above the roof level in parking structures. The design wind force in such cases is computed from ASCE Equation 6-20:

$$F = q_z G C_f A_f$$

where q_z = velocity pressure evaluated at height z of the centroid of area A_f using the exposure defined in ASCE 6.5.6.3.2

G = gust effect factor from ASCE 6.5.8

C_f = net force coefficients from ASCE Tables 6-9 through 6-12

A_f = projected area normal to wind except where C_f is specified for the actual surface area

In this case, force coefficients C_f in ASCE Table 6-11 for solid freestanding walls and solid signs are applicable. For a ratio of height to width v less than 3, C_f = 1.2. Assuming a 4-ft tall parapet, q_z at 42 + 2 = 44 ft (location of centroid of parapet above ground) is equal to $0.00256 \times 0.782 \times 90^2 = 16.2$ psf. Design wind force F at the centroid of the parapet is:

$$F = 16.2 \times 0.85 \times 1.2 \times 4 \times 186/1{,}000 = 12.3 \text{ kips}$$

Thus, at the roof level, F = 12.3 kips is added to the design wind force computed from the design wind pressure for enclosed buildings from ASCE Equation 6-19. Tables 6-8 and 6-9 contain summaries of design pressures and forces, respectively, for wind in the E-W direction. Wind pressures are applied to the gross projected wall area without discounting the wall openings.

Table 6-8 Design Wind External Pressures in E-W Direction (V = 90 mph)

Location	Level	Height above ground level, z (ft)	q (psf)	G	C_p	qGC_p (psf)
Windward	4	42	16.0	0.85	0.80	10.9
	3	32	14.8	0.85	0.80	10.1
	2	22	13.3	0.85	0.80	9.0
	1	12	11.9	0.85	0.80	8.1
Leeward	---	All	16.0	0.85	-0.38	-5.2

Table 6-9 Design Wind Forces in E-W Direction (V = 90 mph)

Level	Height above ground level, z (ft)	Tributary Height (ft)	Windward: External Design Wind Pressure, q_zGC_p (psf)	Windward: Design Wind Force, P* (kips)	Leeward: External Design Wind Pressure, q_hGC_p (psf)	Leeward: Design Wind Force, P* (kips)	Total Design Wind Force (kips)
4	42	5.0	10.9	10.1	-5.2	4.8	27.2†
3	32	10.0	10.1	18.8	-5.2	9.7	28.5
2	22	10.0	9.0	16.7	-5.2	9.7	26.4
1	12	11.0	8.1	16.6	-5.2	10.6	27.2
						Σ	109.3

*$P = qGC_p \times$ Tributary height $\times 186$

† Total wind force at roof = force from external design wind pressures + force at parapet = (10.1 + 4.8) + 12.3 = 27.2 kips

6.2.4 Design for Combined Load Effects

6.2.4.1 Load Combinations

Basic load combinations for strength design are given in IBC 1605.2.1. The first exception in this section requires that the non-seismic load combinations of ACI 9.2 be used for concrete structures. Thus, the following load combinations are applicable in the design of the structural members:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)
4. $1.2D + f_1 L + 1.0E$ (Formula 16-5)
5. $0.9D + 1.0E$ (Formula 16-6)

where D, L, W, and E are the effects due to dead, live, wind, and seismic loads, respectively, and f_1 is equal to either 1.0 for places of public assembly, for live loads in excess of 100 psf, and for parking garage live load, or is equal to 0.5 for other live loads.

The seismic load effect E for use in Formula 16-5, which is the combined effect of horizontal and vertical earthquake-induced forces, is computed from Eq. 16-28 where the effects of gravity and seismic ground motion are additive:

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of horizontal seismic forces

ρ = redundancy coefficient
= 1.0 for structures assigned to SDC A, B, or C (IBC 1617.2.1)

Similarly, E for use in Formula 16-6 is computed from Eq. 16-29 where the effects of gravity and seismic ground motion counteract:

$$E = \rho Q_E - 0.2 S_{DS} D$$

Substituting $S_{DS} = 0.22g$ and $\rho = 1.0$ into the equations for E, and then substituting E along with $f_1 = 1.0$ into load combinations 4 and 5 above results in the following:

4a. $1.2D + 1.0L + 1.0Q_E + (0.2 \times 0.22)D = 1.24D + L + Q_E$

4b. $1.2D + 1.0L + 1.0Q_E - (0.2 \times 0.22)D = 1.16D + L + Q_E$

5a. $0.9D + 1.0Q_E + (0.2 \times 0.22)D = 0.94D + Q_E$

5b. $0.9D + 1.0Q_E - (0.2 \times 0.22)D = 0.86D + Q_E$

Values of Q_E are obtained from the structural analysis for both sidesway to the left and to the right based on the code-prescribed horizontal seismic forces acting on the structure.

Comparing the seismic forces in Table 6-1 to the wind forces in Table 6-9, it is clear that the seismic forces will govern the design of the members.

6.2.4.2 Design of Shear Wall No. 5

Table 6-10 contains a summary of the design axial forces, bending moments, and shear forces at the base of shear wall number 5 for seismic forces in the E-W direction. The quantities in the table are for a 9 ft-6¼ in. long segment in the E-W direction. Connection details for the cruciform wall at this location are given below.

Table 6-10 Summary of Design Axial Forces, Bending Moments, and Shear Forces at Base of Shear Wall No. 5 (SDC B)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	323	0	0
Live (L)	91	0	0
Seismic (Q_E)	0	± 1,737	± 54
Load Combinations			
$1.4D + 1.7L$	607	0	0
$1.24D + L + Q_E$	492	1,737	54
$0.86D + Q_E$	278	-1,737	-54

Design for shear.

The shear strength of the concrete is determined in accordance with ACI 11.10.5 for walls subjected to axial compression:

$$V_c = 2\sqrt{f_c'}hd$$

$$= 2\sqrt{5{,}000} \times 10 \times 91.4 / 1{,}000 = 129.3 \text{ kips}$$

where d is permitted to be taken equal to $0.8\ell_w = 0.8 \times 114.25 = 91.4$ in. (ACI 11.10.4). The maximum factored shear force is 54 kips from the second or third load combinations (see Table 6-10). Since $\phi V_c / 2 = 0.85 \times 129.3 / 2 = 55 \text{ kips} > V_u = 54 \text{ kips}$, reinforcement shall be provided in accordance with ACI 11.10.9 or Chapter 14 (ACI 11.10.8). In lieu of the minimum reinforcement and maximum spacing requirements given in ACI 14.3, the area of horizontal and vertical reinforcement shall not be less than 0.001 times the gross cross-sectional area of the wall and the maximum bar spacing shall not be greater than 18 in. for exterior precast, nonprestressed walls (ACI 16.4.2).

Minimum horizontal and vertical reinforcement area = 0.001 × 10 × 12 = 0.12 in.^{2}/ft. Try 2 layers of No. 3 bars spaced at 18 in. ($A_s = 0.15$ in.^{2}/ft).

The shear strength V_n at any horizontal section must be less than or equal to $10\sqrt{f_c'}hd$ = 646.3 kips (ACI 11.10.3). In this case,

$$V_n = V_c + V_s = 129.3 + \frac{(2 \times 0.11) \times 60 \times 91.4}{18} = 196.3 \text{ kips} < 646.3 \text{ kips} \qquad \text{O.K.}$$

Try 2-No. 3 horizontal and vertical bars @ 18 in.

Design for axial force and bending.

It can be shown that the wall is not adequate to resist the load combinations in Table 6-10 when it is reinforced with 2 layers of No. 3 bars @ 18 in. Figure 6-2 contains the interaction diagram of the wall reinforced with 2 layers of No. 4 vertical bars @ 18 in. As seen from the figure, the wall with that amount of reinforcement is adequate for the load combinations in Table 6-10.

IBC 1620.1.7 requires that bearing walls and shear walls and their anchorage in buildings assigned to SDC B and above be designed for an out-of-plane force F_p that is the greater of 10 percent of the weight of the wall w_w or the value obtained from Eq. 16-63:

$$F_p = 0.40 I_E S_{DS} w_w$$

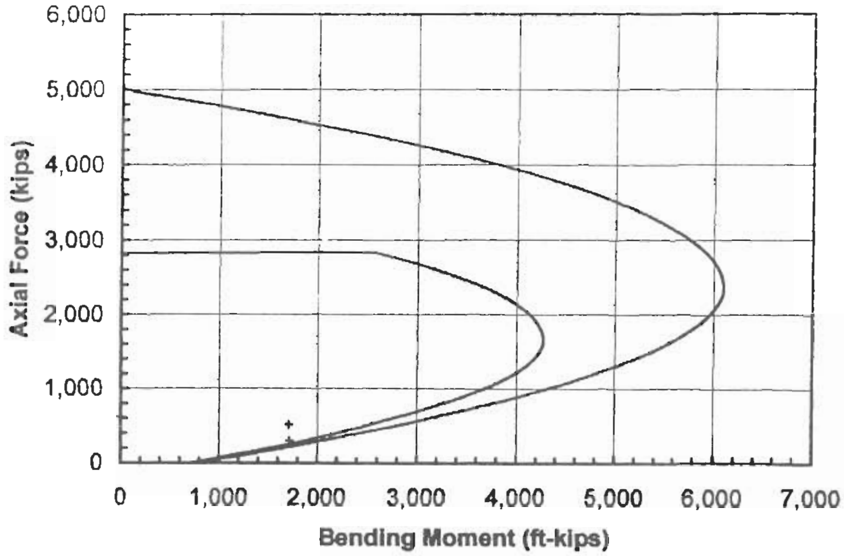


Figure 6-2 Design and Nominal Strength Interaction Diagrams for Shear Wall No. 5 (SDC B)

Out-of-plane seismic forces are usually computed for a 1-ft width of the wall length, assuming a uniformly distributed out-of-plane loading.

For the 10-in. thick wall in the first level:

$$w_w = \frac{10}{12} \times 12 \times 0.15 = 1.5 \text{ kips/ft width}$$

$$F_p = \begin{cases} 0.10 w_w = 0.10 \times 1.5 = 0.15 \text{ kips/ft} \quad \text{(governs)} \\ 0.40 I_E S_{DS} w_w = 0.40 \times 1.0 \times 0.22 \times 1.5 = 0.13 \text{ kips/ft} \end{cases}$$

Distributed load = 0.15/12 = 0.013 klf/ft

Assuming that the wall is simply supported at the floor levels, the maximum bending moment due to the out-of-plane seismic force is:

$$M_u = \frac{0.013 \times 12^2}{8} = 0.23 \text{ ft-kips/ft}$$

The design strength of a 1-ft wide segment of the wall reinforced with 2-No. 4 vertical bars spaced at 18 in. is equal to $\phi M_n = 6$ ft-kips/ft > 0.23 ft-kips/ft O.K.

Splice length of reinforcement.

Class B lap splices are required for the vertical bars in the wall. No splices are needed for the No. 4 horizontal bars, since full length bars weigh approximately 0.668 × 10 = 6.7 lbs. and are easily installed.

For the No. 4 vertical bars:

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 0.8 for No. 6 and smaller bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 0.75 + 0.5 + \frac{0.5}{2} = 1.5 \text{ in.} \quad \text{(governs)} \\ \frac{1}{2} \times 18 = 9.0 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index = 0

$$\frac{c+K_{tr}}{d_b} = \frac{1.5+0}{0.5} = 3.0 > 2.5, \text{ use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{5{,}000}} \times \frac{1.0 \times 1.0 \times 0.8 \times 1.0}{2.5} = 20.4$$

$$\ell_d = 20.4 \times 0.5 = 10.2 \text{ in.} < 12 \text{ in., use } 12 \text{ in.}$$

Class B splice length = $1.3\ell_d$ = 1.3 ft

Use a 1 ft-4 in. splice length for the No. 4 bars.

The No. 4 horizontal bars are developed by providing standard 90-degree hooks at the ends of the bars.

Reinforcement details are shown in Figure 6-3.

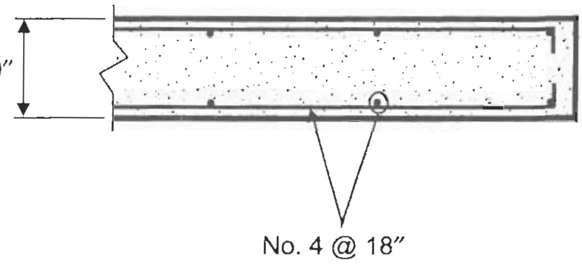


Figure 6-3 Reinforcement Details for Shear Wall No. 5 (SDC B)

Connection details.

The cruciform walls labeled 5 through 8 in Figure 6-1 are comprised of two 9 ft-6¼ in. long load-bearing segments in the E-W direction and one 28 ft long non-load-bearing segment in the N-S direction. A detail of the connection between these segments is contained in Figure 6-4. Information on how to design the components of this connection can be found in Chapter 5 of Reference 6.4. The structural integrity provisions in ACI 16.5 must be satisfied for all precast concrete structures.

6.3 DESIGN FOR SDC C

Design for SDC C is illustrated by assuming that the 4-story precast parking structure depicted in Figure 6-1 is located in Atlanta, GA, on a site designated as Site Class D instead of C, which was assumed previously. All other design data given in Section 6.2.1 are the same.

6.3.1 Seismic Load Analysis

6.3.1.1 Seismic Design Category (SDC)

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1-second period S_{M1} are determined from IBC Eqs. 16-16 and 16-17, respectively:

$$S_{MS} = F_a S_S = 1.58 \times 0.276 = 0.44g$$

$$S_{M1} = F_v S_1 = 2.33 \times 0.117 = 0.27g$$

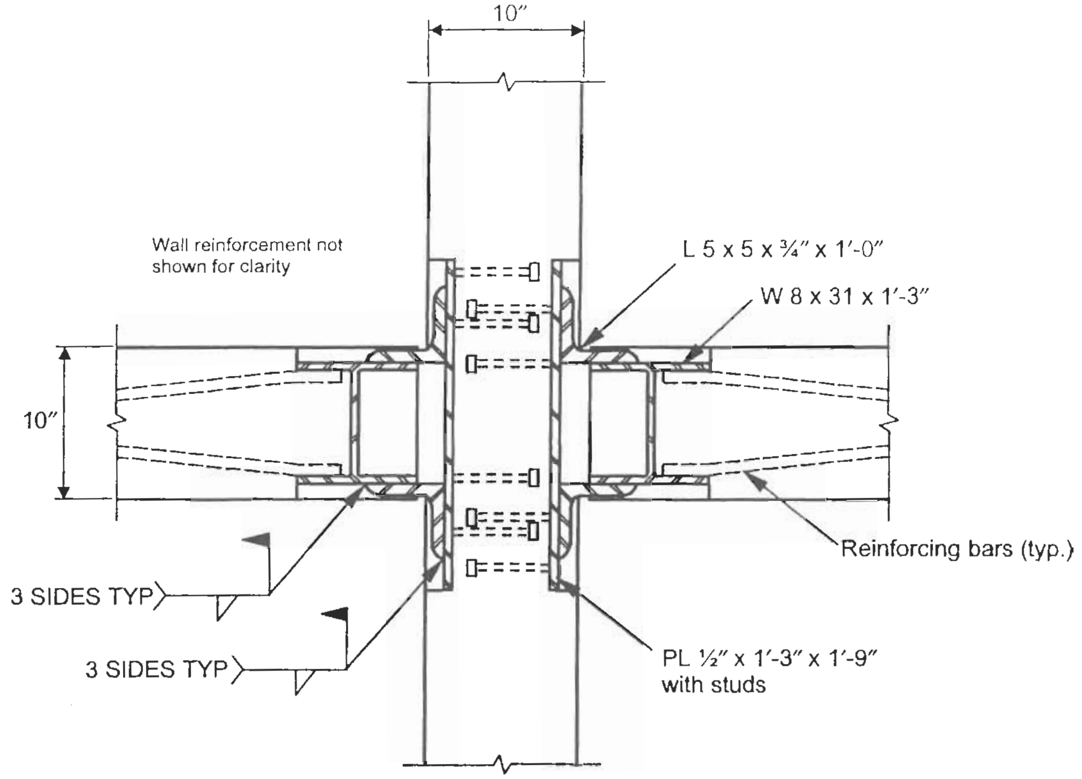


Figure 6-4 Connection Details for Cruciform Wall

where F_a and F_v are the values of site coefficients as a function of site class and of mapped spectral response acceleration at short periods S_S and at 1-second period S_1, respectively. The values of these coefficients for Site Class D in the equations above are contained in IBC Table 1615.1.2(1) for F_a and Table 1615.1.2(2) for F_v. Straight-line interpolation was used to determine F_a and F_v.

Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from Eqs. 16-18 and 16-19:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 0.44 = 0.29g$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.27 = 0.18g$$

Tables 1616.3(1) and 1616.3(2) are utilized to determine the SDC. From Table 1616.3(1) for Seismic Use Group I and S_{DS} = 0.29g, the SDC is B. Similarly, from

Table 1616.3(2), the SDC is C for $S_{D1} = 0.18g$. Since the more severe of the two governs (IBC 1616.3), the SDC is C for this structure.

6.3.1.2 Seismic Forces

According to IBC 1616.6.2, the equivalent lateral force procedure in IBC 1617.4 may be used to compute the seismic base shear V for structures assigned to SDC C. In a given direction, V is determined from Eq. 16-34:

$$V = C_s W$$

where C_s is the seismic response coefficient determined in accordance with IBC 1617.4.1.1 and W is the effective weight of the structure defined in IBC 1617.4. As was determined in the previous example, W = 22,968 kips (see Table 6-11 below).

In both directions, a building frame system with ordinary reinforced concrete shear walls is utilized, which is permitted for structures assigned to SDC C without any limitations (see IBC Table 1617.6 and IBC 1910.4). The response modification coefficient $R = 5$ and the deflection amplification factor $C_d = 4.5$ (IBC Table 1617.6).

Approximate period (T_a). The fundamental period of the building T is determined in accordance with IBC 1617.4.2. In lieu of a more exact analysis, an approximate fundamental period T_a is computed from Eq. 16-39:

Structure height $h_n = 42$ ft
Period coefficient $C_T = 0.02$
Period $T_a = C_T (h_n)^{3/4} = 0.020 \times (42)^{3/4} = 0.33$ sec

Seismic base shear (V). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.18}{\left(\frac{5}{1.0}\right)\times 0.33} = 0.109$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{0.29}{\left(\frac{5.0}{1.0}\right)} = 0.058$$

Also, C_s must not be less than the following value from Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 0.29 \times 1.0 = 0.013$$

Thus, the value of C_s from Eq. 16-35 governs so that the base shear V in the N-S and E-W directions is:

$$V = C_s W = 0.058 \times 22{,}968 = 1{,}332 \text{ kips}$$

Note that the base shear for Site Class D is approximately 1.32 times that determined for Site Class C (see Section 6.2.2.2 of this publication).

Vertical distribution of seismic forces. The total base shear is distributed over the height of the structure in conformance with Eqs. 16-41 and 16-42:

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

where F_x is the lateral force induced at level x, w_x and w_i are the portions of W assigned to levels x or i, and k is the distribution exponent defined in IBC 1617.4.3. For T= 0.33 sec, k = 1.0. The lateral forces F_x and the story shears V_x are contained in Table 6-11.

Table 6-11 Seismic Forces and Story Shears (SDC C)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
4	5,722	42	240,324	513	513
3	5,910	32	189,120	403	916
2	5,912	22	130,064	277	1,193
1	5,424	12	65,088	139	1,332
Σ	22,968		624,596	1,332	

6.3.1.3 Method of Analysis

A three-dimensional analysis of the structure was performed in the E-W direction for the seismic forces contained in Table 6-11 using the computer program *BRDLAT* [6.2]. The stiffness properties of the members were input assuming cracked sections. In lieu of a more accurate analysis, the following cracked section properties were used:

- Beams: $I_{eff} = 0.7 I_g$
- Columns: $I_{eff} = 0.5 I_g$
- Shear walls: $I_{eff} = 0.4 I_g$

where I_g is the gross moment of inertia of the section.

In accordance with IBC 1617.4.4.4, the center of mass was displaced each way from its actual location a distance equal to 5 percent of the structure dimension perpendicular to the applied forces to account for accidental torsion in seismic design. Torsional effects need not be amplified, since the structure does not possess Type 1a or 1b plan irregularities (IBC 1617.4.4.5).

6.3.1.4 Story Drift and P-delta Effects

Story drift determination. Table 6-12 contains the displacements δ_{xe} in the E-W direction obtained from the 3-D static, elastic analysis using the design seismic forces, including accidental torsional effects. The table also contains the design earthquake displacement δ_x computed by IBC Eq. 16-46:

$$\delta_x = \frac{C_d \delta_{xe}}{I_E}$$

where C_d is the deflection amplification factor in Table 1617.6, which depends on the seismic-force-resisting system. As noted above, C_d is equal to 4.5 for this system.

Table 6-12 Lateral Displacements and Interstory Drifts due to Seismic Forces in E-W Direction (SDC C)

Story	δ_{xe} (in.)	δ_x (in.)	Δ (in.)
4	0.78	3.51	1.17
3	0.52	2.34	1.03
2	0.29	1.31	0.81
1	0.11	0.50	0.50

The interstory drifts Δ computed from the δ_x are also contained in the table, where the drift at story level x is determined by subtracting the design earthquake displacement at the center of mass at the bottom of the story from the design earthquake displacement at the center of mass at the top of the story (IBC 1617.4.6.1):

$$\Delta = \delta_x - \delta_{x-1}$$

The design story drifts Δ must not exceed the allowable story drift Δ_a from Table 1617.3 (IBC 1617.3). For Seismic Use Group I, $\Delta_a = 0.025 h_{sx}$ for structures 4 stories or less where h_{sx} is the story height below level x. Thus, for the 10-ft story heights, $\Delta_a = 0.025 \times 10 \times 12 = 3.0$ in., and for the 12-ft story height at the first level, $\Delta_a = 3.6$ in. It is evident from Table 6-12 that for all stories, the lateral drifts obtained from the prescribed lateral forces in the E-W direction are less than the limiting values.

Similar calculations for seismic forces in the N-S direction also show that the lateral drifts are less than the allowable values.

P-delta effects. Calculations for P-delta effects are similar to those shown in Section 6.2.2.4 of this publication. Table 6-13 contains the calculations for the E-W direction. It is clear that P-delta effects need not be considered at any of the floor levels. Note that θ_{max} is equal to 0.111 in the E-W direction using $\beta = 1.0$. Similar calculations for the N-S direction show that P-delta effects may also be neglected.

Table 6-13 P-delta Effects (SDC B)

Level	h_{sx} (ft)	P_x (kips)	V_x (kips)	Δ (in.)	θ
4	10	8,543	513	1.17	0.0361
3	10	16,710	916	1.03	0.0348
2	10	24,879	1,193	0.81	0.0313
1	12	32,559	1,332	0.50	0.0189

6.3.1.5 Overturning

Calculations for overturning are similar to those given in Section 6.2.2.5 of this publication.

Tables 6-14 and 6-15 contain summaries of the overturning and resisting moments at the base of the shear walls in both the N-S and E-W directions, respectively. When computing the resisting moments, the dead load was multiplied by the factor $(0.9 - 0.2 S_{DS}) = 0.9 - (0.2 \times 0.29) = 0.84$ in accordance with Eq. 16-29, which applies when the effects of gravity and seismic ground motion counteract (IBC 1617.1.1).

For all walls except numbers 6 and 8, the ratio of resisting moment to overturning moment is greater than 1. When the overturning-to-resisting moment ratio is less than 1, the foundation must provide resistance to uplift. One viable option is to tie adjacent footings together with a grade beam.

Table 6-14 Resisting and Overturning Moments at Base of Shear Walls for Seismic Forces in the N-S Direction (SDC C)

Wall No.	Length (ft)	Dead Load* (kips)	Resisting Moment M_r † (ft-kips)	Overturning Moment M_x (ft-kips)	M_r / M_x
1	24	437	4,405	3,693	1.19
2	24	437	4,405	3,693	1.19
3	24	437	4,405	3,737	1.18
4	24	437	4,405	3,737	1.18
5	28	645	7,585	5,662	1.34
6	28	577	6,786	7,079	0.96
7	28	645	7,585	5,662	1.34
8	28	577	6,786	7,079	0.96

*Dead load = weight of wall + dead load tributary to wall

† $M_r = 0.84 \times$ dead load $\times$ (length/2)

Table 6-15 Resisting and Overturning Moments at Base of Shear Walls for Seismic Forces in the E-W Direction (SDC C)

Wall No.	Length (ft)	Dead Load* (kips)	Resisting Moment M_r † (ft-kips)	Overturning Moment M_x (ft-kips)	M_r / M_x
5	20	645	5,418	4,220	1.28
6	20	577	4,847	5,367	0.90
7	20	645	5,418	4,220	1.28
8	20	577	4,847	5,367	0.90
9	32	861	11,572	10,827	1.07
10	32	861	11,572	10,827	1.07

*Dead load = weight of wall + dead load tributary to wall

† $M_r = 0.84 \times$ dead load $\times$ (length/2)

6.3.2 Wind Load Analysis

Wind forces are determined in accordance with the analytical procedure (Method 2) given in ASCE 6.5 [6.3]. Table 6-9 in Section 6.2.3 contains the design wind forces in the E-W direction.

6.3.3 Design for Combined Load Effects

6.3.3.1 Load Combinations

The following load combinations are applicable:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)

3. $0.9D + 1.3W$ (Eq. 9-3)

4. $1.2D + f_1 L + 1.0E$ (Formula 16-5)

5. $0.9D + 1.0E$ (Formula 16-6)

where D, L, W, and E are the effects due to dead, live, wind, and seismic loads, respectively, and f_1 is equal to either 1.0 for places of public assembly, for live loads in excess of 100 psf, and for parking garage live load, or is equal to 0.5 for other live loads.

The seismic load effect E for use in Formula 16-5, which is the combined effect of horizontal and vertical earthquake-induced forces, is computed from Eq. 16-28 where the effects of gravity and seismic ground motion are additive:

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of horizontal seismic forces
ρ = redundancy coefficient
= 1.0 for structures assigned to SDC A, B, or C (IBC 1617.2.1)

Similarly, E for use in Formula 16-6 is computed from Eq. 16-29 where the effects of gravity and seismic ground motion counteract:

$$E = \rho Q_E - 0.2 S_{DS} D$$

Substituting S_{DS} = 0.29*g* and ρ = 1.0 into the equations for E, and then substituting E along with f_1 = 1.0 into load combinations 4 and 5 above results in the following:

4a. $1.2D + 1.0L + 1.0Q_E + (0.2 \times 0.29)D = 1.26D + L + Q_E$

4b. $1.2D + 1.0L + 1.0Q_E - (0.2 \times 0.29)D = 1.14D + L + Q_E$

5a. $0.9D + 1.0Q_E + (0.2 \times 0.29)D = 0.96D + Q_E$

5b. $0.9D + 1.0Q_E - (0.2 \times 0.29)D = 0.84D + Q_E$

Values of Q_E are obtained from the structural analysis for both sidesway to the left and to the right based on the code-prescribed horizontal seismic forces acting on the structure.

Comparing the seismic forces in Table 6-11 to the wind forces in Table 6-9, it is clear that the seismic forces will govern the design of the members.

6.3.3.2 Design of Shear Wall No. 5

Table 6-16 contains a summary of the design axial forces, bending moments, and shear forces at the base of shear wall number 5 for seismic forces in the E-W direction. The reactions in the table are for a 9 ft-6¼ in. long segment in the E-W direction. Connection details for the cruciform wall are given in Figure 6-4.

Table 6-16 Summary of Design Axial Forces, Bending Moments, and Shear Forces at Base of Shear Wall No. 5 (SDC C)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	323	0	0
Live (L)	91	0	0
Seismic (Q_E)	0	± 2,256	± 71
Load Combinations			
$1.4D + 1.7L$	607	0	0
$1.26D + L + Q_E$	498	2,256	71
$0.84D + Q_E$	271	-2,256	-71

Design for shear.

The shear strength of the concrete is determined in accordance with ACI 11.10.5 for walls subjected to axial compression:

$$V_c = 2\sqrt{f'_c}hd$$

$$= 2\sqrt{5{,}000} \times 10 \times 91.4 / 1{,}000 = 129.3 \text{ kips}$$

where d is permitted to be taken equal to $0.8\ell_w = 0.8 \times 114.25 = 91.4$ in. (ACI 11.10.4). The maximum factored shear force is 71 kips from the second or third load combinations (see Table 6-16). Since $\phi V_c / 2 = 55 \text{ kips} < V_u = 71 \text{ kips}$, reinforcement shall be provided in accordance with ACI 11.10.9 (ACI 11.10.8).

Minimum horizontal and vertical reinforcement area = 0.0025 × 10 × 12 = 0.30 in.^{2}/ft. Try 2 layers of No. 4 bars spaced at 16 in. (A_s = 0.30 in.^{2}/ft).

Check shear strength:

$$\phi V_n = \phi(V_c + V_s)$$

$$= 0.85\left[129.3 + \frac{(2 \times 0.2) \times 60 \times 91.4}{16}\right] = 226.4 \text{ kips} > V_u = 71 \text{ kips} \quad \text{O.K.}$$

The shear strength V_n at any horizontal section must be less than or equal to $10\sqrt{f_c'}hd$ = 646.3 kips (ACI 11.10.3). In this case, $V_n = 226.4/0.85 = 266.4$ kips < 646.3 kips O.K.

Try 2-No. 4 horizontal and vertical bars @ 16 in.

Design for axial force and bending.

It can be shown that the wall is not adequate to resist the load combinations in Table 6-16 when it is reinforced with 2 layers of No. 4 bars @ 16 in. Figure 6-5 contains the interaction diagram of the wall reinforced with 2 layers of No. 5 vertical bars @ 12 in. As seen from the figure, the wall with that amount of reinforcement is adequate for the load combinations in Table 6-16.

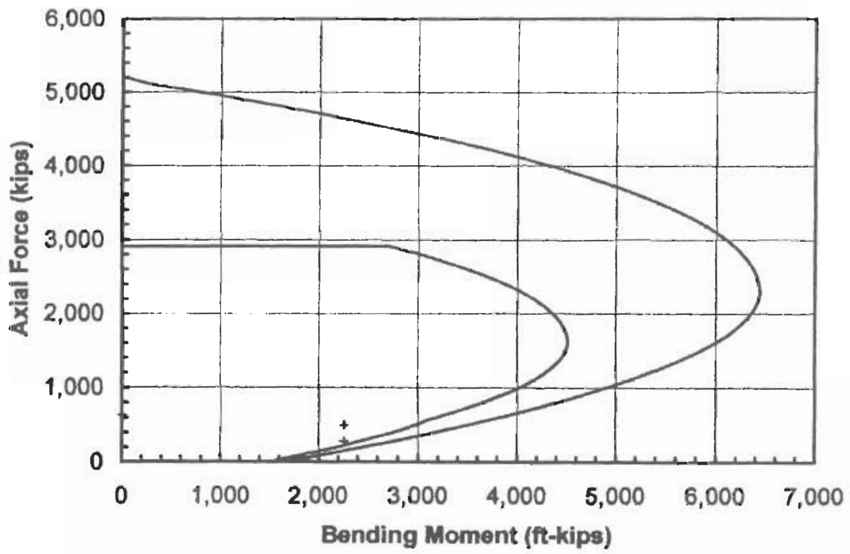


Figure 6-5 Design and Nominal Strength Interaction Diagrams for Shear Wall No. 5 (SDC C)

IBC 1620.1.7 requires that bearing walls and shear walls and their anchorage in buildings assigned to SDC B and above be designed for an out-of-plane force F_p that is the greater of 10 percent of the weight of the wall w_w or the value obtained from Eq. 16-63:

$$F_p = 0.40 I_E S_{DS} w_w$$

Out-of-plane seismic forces are usually computed for a 1-ft width of the wall length, assuming a uniformly distributed out-of-plane loading.

For the 10-in. thick wall in the first level:

$$w_w = \frac{10}{12} \times 12 \times 0.15 = 1.5 \text{ kips/ft width}$$

$$F_p = \begin{cases} 0.10 w_w = 0.10 \times 1.5 = 0.15 \text{ kips/ft} \\ 0.40 I_E S_{DS} w_w = 0.40 \times 1.0 \times 0.29 \times 1.5 = 0.17 \text{ kips/ft} \quad \text{(governs)} \end{cases}$$

Distributed load = 0.17/12 = 0.014 klf/ft

Assuming that the wall is simply supported at the floor levels, the maximum bending moment due to the out-of-plane seismic force is:

$$M_u = \frac{0.014 \times 12^2}{8} = 0.26 \text{ ft-kips/ft}$$

The design strength of a 1-ft wide segment of the wall reinforced with 2-No. 5 vertical bars spaced at 12 in. is equal to ϕM_n = 13.0 ft-kips/ft > 0.26 ft-kips/ft O.K.

Splice length of reinforcement.

Class B lap splices are required for the vertical bars in the wall. No splices are needed for the No. 4 horizontal bars, since full length bars weigh approximately 0.668 × 10 = 6.7 lbs. and are easily installed.

For the No. 5 vertical bars:

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

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 0.8 for No. 6 and smaller bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 0.75 + 0.5 + \frac{0.625}{2} = 1.6 \text{ in.} \quad \text{(governs)} \\ \frac{1}{2} \times 12 = 6.0 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index = 0

$$\frac{c+K_{tr}}{d_b} = \frac{1.6+0}{0.625} = 2.6 > 2.5, \text{ use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{5{,}000}} \times \frac{1.0 \times 1.0 \times 0.8 \times 1.0}{2.5} = 20.4$$

$$\ell_d = 20.4 \times 0.625 = 12.8 \text{ in.}$$

Class B splice length = $1.3\ell_d$ = 1.4 ft

Use a 1 ft-5 in. splice length for the No. 5 bars.

The No. 4 horizontal bars are developed by providing standard 90-degree hooks at the ends of the bars.

Reinforcement details are shown in Figure 6-6.

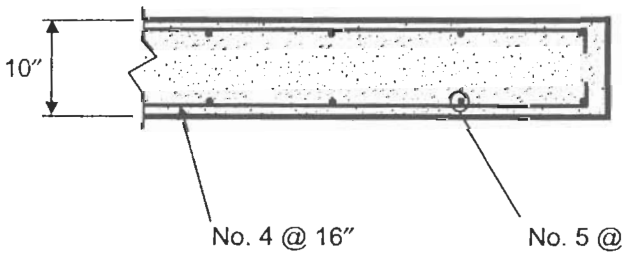


Figure 6-6 Reinforcement Details for Shear Wall No. 5 (SDC C)

6.4 DESIGN FOR SDC D

The 4-story precast parking structure with cast-in-place shear walls depicted in Figure 6-7 is assumed to be located in San Francisco, CA. The elevation of the structure is similar to that shown in Figure 6-1. Typical walls are designed and detailed for combined effects of gravity, wind, and seismic forces.

A building frame system with special reinforced concrete shear walls may be used for structures assigned to SDC D considering the limitations contained in IBC Table 1617.6 (IBC 1910.5.1). Frame members not proportioned to resist forces induced by earthquake motions are to conform to the provisions in ACI 21.9 (IBC 1910.5.2). This type of system is utilized in this example.

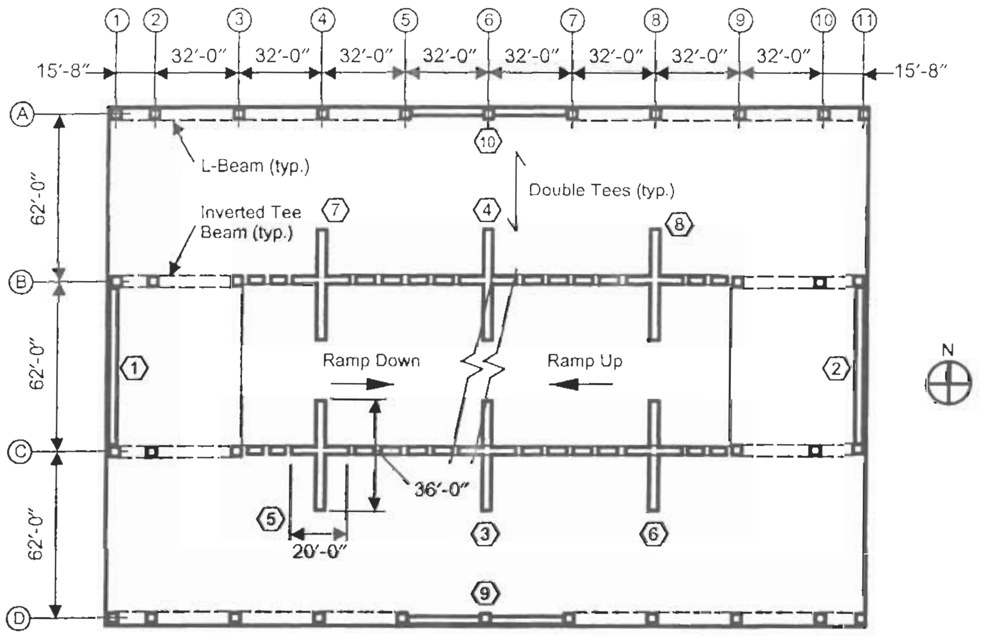


Figure 6-7 Typical Plan of Precast Parking Structure with Cast-in-place Shear Walls (SDC D)

6.4.1 Design Data

- Building Location: San Francisco, CA (zip code 94105)
- Material Properties

  Concrete: $f'_c = 5{,}000$ psi, $w_c = 150$ pcf
  Reinforcement: $f_y = 60{,}000$ psi

- Service Loads

  Live loads: 50 psf
  Dead loads: 90 psf (weight of structural members + superimposed dead loads)

- Seismic Design Data

  For zip code 94105: $S_S = 1.50g, S_1 = 0.61g$ [6.1]
  Site Class D (stiff soil profile; IBC Table 1615.1.1)
  For Category (Seismic Use Group) I occupancy, $I_E = 1.0$ (IBC Table 1604.5)

- Wind Design Data

  Basic wind speed = 85 mph (IBC Figure 1609)
  Exposure B (IBC 1609.4)

For Category I occupancy, $I_W = 1.0$ (IBC Table 1604.5)

- Member Dimensions

Wall thickness: 10 in.

6.4.2 Seismic Load Analysis

6.4.2.1 Seismic Design Category (SDC)

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1-second period S_{M1} are determined from IBC Eqs. 16-16 and 16-17, respectively. According to IBC 1616.6.3, for regular structures 5 stories or less having a period determined in accordance with IBC 1617.4.2 of 0.5 sec or less, S_{DS} and S_{D1} need not exceed the values calculated using values of S_S = 1.50*g* and S_1 = 0.60*g*, respectively. The 4-story example structure is regular according to IBC 1616.6.3, and it is shown below that its period is less than 0.5 sec. Therefore,

$$S_{MS} = F_a S_S = 1.0 \times 1.50 = 1.50g$$

$$S_{M1} = F_v S_1 = 1.5 \times 0.60 = 0.90g$$

where F_a and F_v are contained in IBC Table 1615.1.2(1) and Table 1615.1.2(2), respectively.

Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from Eqs. 16-18 and 16-19:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 1.50 = 1.00g$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.90 = 0.60g$$

Tables 1616.3(1) and 1616.3(2) are utilized to determine the SDC. From Table 1616.3(1) for Seismic Use Group I and S_{DS} = 1.00*g*, the SDC is D. Similarly, from Table 1616.3(2), the SDC is D for S_{D1} = 0.60*g*. Thus, the SDC is D for this structure.

6.4.2.2 Seismic Forces

Since the structure does not have plan irregularity Type 1a, 1b, or 4 of Table 1616.5.1 or vertical irregularity Type 1a, 1b, 4, or 5 of Table 1616.5.2, it can be considered regular (IBC 1616.6.3). For this regular structure that is less than 240 ft in height, Table 1616.6.3

allows the equivalent lateral force procedure in IBC 1617.4 to be used to compute the seismic base shear V (see Eq. 16-34):

$$V = C_s W$$

where C_s is the seismic response coefficient determined in accordance with IBC 1617.4.1.1 and W is the effective weight of the structure defined in IBC 1617.4. For this example, W = 23,890 kips (see Table 6-16 below).

In both directions, a building frame system with special reinforced concrete shear walls is utilized, which is permitted for structures assigned to SDC D with a height less than or equal to 160 ft (see IBC Table 1617.6 and IBC 1910.5). The response modification coefficient $R = 6$ and the deflection amplification factor $C_d = 5$ (IBC Table 1617.6).

Approximate period (T_a). The fundamental period of the building T is determined in accordance with IBC 1617.4.2. In lieu of a more exact analysis, an approximate fundamental period T_a is computed from Eq. 16-39:

Building height $h_n = 42$ ft
Building period coefficient $C_T = 0.02$
Period $T_a = C_T (h_n)^{3/4} = 0.020 \times (42)^{3/4} = 0.33$ sec

Seismic base shear (V). The seismic response coefficient C_s is determined from Eq. 16-36:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.60}{\left(\frac{6}{1.0}\right) \times 0.33} = 0.303$$

The value of C_s need not exceed that from Eq. 16-35:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{1.00}{\left(\frac{6}{1.0}\right)} = 0.167$$

Also, C_s must not be less than the following value from Eq. 16-37:

$$C_s = 0.044 S_{DS} I_E = 0.044 \times 1.00 \times 1.0 = 0.044$$

For buildings assigned to SDC E or F and for those buildings for which $S_1 \geq 0.6g$, C_s shall not be taken less than that computed from Eq. 16-38. Since $S_1 = 0.60g$, Eq. 16-38 is applicable, even though the SDC is D:

$$C_s = \frac{0.5S_1}{R/I_E} = \frac{0.5 \times 0.60}{6/1.0} = 0.050$$

In this case, the lower limit is 0.050 from Eq. 16-38.

Thus, the value of C_s from Eq. 16-35 governs so that the base shear V in the N-S and E-W directions is:

$$V = C_s W = 0.167 \times 23{,}890 = 3{,}990 \text{ kips}$$

Vertical distribution of seismic forces. The total base shear is distributed over the height of the building in conformance with Eqs. 16-41 and 16-42:

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

where F_x is the lateral force induced at level x, w_x and w_i are the portions of W assigned to levels x or i, and k is the distribution exponent defined in IBC 1617.4.3. For T = 0.33 sec, k = 1.0. The lateral forces F_x and the story shears V_x are contained in Table 6-17.

Table 6-17 Seismic Forces and Story Shears (SDC D)

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x (kips)	Story Shear, V_x (kips)
4	5,665	42	237,930	1,481	1,481
3	6,146	32	196,672	1,224	2,705
2	6,146	22	135,212	842	3,547
1	5,933	12	71,196	443	3,990
Σ	23,890		641,010	3,990	

6.4.2.3 Method of Analysis

Shear walls in a building frame system are designed to carry the total lateral forces, and frames provide support for the gravity loads. Also, frame members, which are not designated to be part of the seismic-force-resisting system (SFRS), must be designed and detailed to maintain support of design dead and live loads when subjected to expected deformations caused by seismic forces.

The internal forces in members that are part of the SFRS were determined from a three-dimensional analysis of a structural model comprised of the shear walls subjected to the seismic forces in Table 6-17. In accordance with IBC 1617.4.4.4, the center of mass was displaced each way from its actual location a distance equal to 5 percent of the building dimension perpendicular to the applied forces to account for accidental torsion. Torsional effects need not be amplified, since the building does not possess Type 1a or 1b plan torsional irregularities as defined in Table 1616.5.1 (IBC 1617.4.4.5). In lieu of a more accurate analysis, the following cracked section properties were used:

- Beams: $I_{eff} = 0.7 I_g$
- Columns: $I_{eff} = 0.5 I_g$
- Shear walls: $I_{eff} = 0.4 I_g$

For all structures assigned to SDC D and higher, IBC 1620.3.5 requires that orthogonal effects of the seismic forces be considered for design and detailing of the components of the SFRS. In the 2002 supplement to the 2000 IBC [6.5], the orthogonal combination procedure is required only for columns or walls that form part of two or more intersecting seismic-force-resisting systems and that are subjected to axial load due to seismic forces greater than or equal to 20% of the axial load design strength. Orthogonal effects can be neglected for the shear walls in this example building.

IBC 1617.6.4.3 requires that reinforced concrete members not designed as part of the SFRS comply with ACI 21.9. According to ACI 21.9.1, such members are to be detailed according to ACI 21.9.2 or 21.9.3 depending on the magnitude of bending moments and shear forces in those members when subjected to the design displacements δ_x where $\delta_x = C_d \delta_{xe} / I_E$ and δ_{xe} are the displacements calculated by an elastic analysis of the SFRS subjected to the prescribed seismic forces (IBC 1617.6.4.3). The approximate method that was first introduced in Section 3.4.2.3 of this publication can be used to determine these internal forces. Since calculations for this method were covered in sufficient detail in the applicable examples in Chapter 3 of this publication, similar calculations are not repeated here.

It is important to note that IBC 1908.1.6 contains additional requirements for structures with precast gravity systems. These requirements are modifications to the 1999 ACI code and are contained in new Section 21.2.1.7. The intent of these provisions is to have overall structural performance of a precast gravity system equivalent or at least similar to that provided by a monolithic (cast-in-place) gravity system.

The first item in Section 21.2.1.7 is a limitation on the aspect ratio of horizontal diaphragms, which is applicable to the lower two-thirds of the stories of buildings 3 stories or more in height. The span of the diaphragm or diaphragm segment between lateral-force-resisting systems must be less than or equal to 3 times the width of the diaphragm or diaphragm segment. This requirement seeks to provide backup frame action, which is expected from monolithic gravity frames, by increasing the redundancy of the lateral-force-resisting system. In this example, the longest span between shear walls occurs in the segment between column lines 1 to 4 or 8 to 11 and is equal to 79.67 ft. The corresponding width of the segment is 62 ft, so that the ratio of span to width is equal to 79.67/62 = 1.3 < 3.

The second item in Section 21.2.1.7 must be satisfied when the diaphragm aspect ratio limitations of the first alternative cannot be economically met. This requires that all precast gravity frame beam-to-column connections be partially restrained.

In addition to the above requirements, complete calculations for the deformation compatibility of the gravity load-carrying system must be made in accordance with IBC 1617.6.4.3 using cracked section stiffnesses of the lateral-force-resisting elements and the diaphragm. These calculations are not presented for this example, since similar calculations were performed for the applicable example buildings in Chapter 3 of this publication.

Requirements for gravity columns that are not laterally supported on all sides and for minimum bearing lengths of horizontal members are also contained in Section 21.2.1.7.

6.4.2.4 Story Drift and P-delta Effects

Story drift determination. Table 6-18 contains the displacements δ_{xe} of the SFRS in the E-W direction obtained from the 3-D static, elastic analysis using the design seismic forces, including accidental torsional effects, and the design earthquake displacement δ_x computed by IBC Eq. 16-46. As noted above, C_d is equal to 5 for this system. The interstory drifts Δ computed from the δ_x are also contained in the table.

Table 6-18 Lateral Displacements and Interstory Drifts due to Seismic Forces in E-W Direction (SDC D)

Story	δ_{xe} (in.)	δ_x (in.)	Δ (in.)
4	0.19	0.95	0.35
3	0.12	0.60	0.35
2	0.07	0.35	0.25
1	0.02	0.10	0.10

The design story drifts Δ must not exceed the allowable story drift Δ_a from Table 1617.3 (IBC 1617.3). For the 10-ft story heights, $\Delta_a = 0.025 \times 10 \times 12 = 3.0$ in. and for the 12-ft story height at the first level, $\Delta_a = 3.6$ in. It is evident from Table 6-18 that for all stories, the lateral drifts obtained from the prescribed lateral forces in the E-W direction are less than the limiting values.

Similar calculations for seismic forces in the N-S direction also show that the lateral drifts are less than the allowable values.

P-delta effects. Calculations for P-delta effects are similar to those shown in Section 6.3.1.4 of this publication. Similar calculations for this example in the E-W and N-S directions show that P-delta effects may also be neglected.

6.4.2.5 Overturning

Calculations for overturning are similar to those given in Section 6.2.2.5 of this publication.

Tables 6-19 and 6-20 contain summaries of the overturning and resisting moments at the base of the shear walls in both the N-S and E-W directions, respectively. When computing the resisting moments, the dead load was multiplied by the factor $(0.9 - 0.2S_{DS}) = 0.9 - (0.2 \times 1.0) = 0.7$ in accordance with Eq. 16-29, which applies when the effects of gravity and seismic ground motion counteract (IBC 1617.1.1).

Table 6-19 Resisting and Overturning Moments at Base of Shear Walls for Seismic Forces in the N-S Direction (SDC D)

Wall No.	Length (ft)	Dead Load* (kips)	Resisting Moment M_r† (ft-kips)	Overturning Moment M_x (ft-kips)	M_r / M_x
1	60	997	20,937	23,389	0.90
2	60	997	20,937	25,175	0.83
3	36	577	7,270	12,015	0.61
4	36	577	7,270	12,015	0.61
5	36	645	8,127	9,257	0.88
6	36	577	7,270	10,833	0.67
7	36	645	8,127	9,257	0.88
8	36	577	7,270	10,833	0.67

*Dead load = weight of wall + dead load tributary to wall

† $M_r = 0.7 \times$ dead load $\times$ (length/2)

It is evident that for most of the walls, the ratio of resisting-to-overturning moment is less than one. In these cases, the foundation design must include resistance to uplift. One viable option is to tie adjacent footings together with grade beams.

Table 6-20 Resisting and Overturning Moments at Base of Shear Walls for Seismic Forces in the E-W Direction (SDC D)

Wall No.	Length (ft)	Dead Load* (kips)	Resisting Moment M_r † (ft-kips)	Overturning Moment M_x (ft-kips)	M_r / M_x
3	20	577	4,039	6,328	0.64
4	20	577	4,039	6,385	0.63
5	20	645	4,515	4,280	1.06
6	20	577	4,039	5,186	0.78
7	20	645	4,515	4,319	1.05
8	20	577	4,039	5,233	0.77
9	62	1,271	27,581	42,579	0.65
10	62	1,271	27,581	43,551	0.63

*Dead load = weight of wall + dead load tributary to wall

† M_r = 0.7 × dead load × (length/2)

6.4.3 Wind Load Analysis

In this example, the wind velocity is 85 mph, which produces wind forces that are significantly smaller than the seismic forces computed above. Thus, wind forces are not considered in this example.

6.4.4 Design for Combined Load Effects

6.4.4.1 Load Combinations

The following load combinations are applicable to structural members that are part of the SFRS:

1. $1.4D + 1.7L$ (Eq. 9-1)
2. $0.75(1.4D + 1.7L + 1.7W)$ (Eq. 9-2)
3. $0.9D + 1.3W$ (Eq. 9-3)
4. $1.2D + f_1 L + 1.0E$ (Formula 16-5)
5. $0.9D + 1.0E$ (Formula 16-6)

where D, L, W, and E are the effects due to dead, live, wind, and seismic loads, respectively, and f_1 is equal to either 1.0 for places of public assembly, for live loads in excess of 100 psf, and for parking garage live load, or is equal to 0.5 for other live loads.

The seismic load effect E for use in Formula 16-5, which is the combined effect of horizontal and vertical earthquake-induced forces, is computed from Eq. 16-28 where the effects of gravity and seismic ground motion are additive:

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of horizontal seismic forces
ρ = redundancy coefficient determined in accordance with IBC 1617.2.2 for SDC D, E, or F

Similarly, E for use in Formula 16-6 is computed from Eq. 16-29 where the effects of gravity and seismic ground motion counteract:

$$E = \rho Q_E - 0.2 S_{DS} D$$

According to IBC 1617.2.2, the redundancy coefficient ρ, which shall not be less than 1.0 and need not exceed 1.5, is the largest of the values of ρ_i calculated at each story i from Eq. 16-32:

$$\rho_i = 2 - \frac{20}{r_{\max_i}\sqrt{A_i}}$$

For shear walls:

$r_{\max_i}$ = (maximum wall shear × $10/\ell_w$)/total story shear

ℓ_w = length of the wall in feet

For the structure depicted in Figure 6-7, shear wall number 9 or 10 will have the largest shear force at its base in the E-W direction, depending on which direction the center of mass is displaced. Therefore,

$$r_{\max_1} = \frac{1{,}316 \times \frac{10}{62}}{3{,}990} = 0.05$$

$$\rho_{\max} = 2 - \frac{20}{0.05\sqrt{287.33 \times 186}} = 0.27 < 1.0$$

Use ρ = 1.0.

Substituting S_{DS} = 1.00g and ρ = 1.0 into the equations for E, and then substituting E along with f_1 = 1.0 into load combinations 4 and 5 above results in the following:

4a. $1.2D + 1.0L + 1.0Q_E + (0.2 \times 1.00)D = 1.4D + L + Q_E$

4b. $1.2D + 1.0L + 1.0Q_E - (0.2 \times 1.00)D = D + L + Q_E$

5a. $0.9D + 1.0Q_E + (0.2 \times 1.00)D = 1.1D + Q_E$

5b. $0.9D + 1.0Q_E - (0.2 \times 1.00)D = 0.7D + Q_E$

Values of Q_E are obtained from the structural analysis for both sidesway to the left and to the right based on the code-prescribed horizontal seismic forces acting on the building.

The exception in IBC 1617.6.4.3 requires that reinforced concrete members that are not part of the SFRS comply with ACI 21.9. Therefore, the load combinations for these members are:

- $1.05D + 1.28L + E$
- $0.9D + E$

where the values of E are the forces in the frame members due to the lateral displacements δ_x being applied to the frames.

Also, the special seismic load combinations in IBC 1605.4 are not applicable in this example.

6.4.4.2 Design of Shear Wall No. 5

Table 6-21 contains a summary of the design axial forces, bending moments, and shear forces at the base of the wall for seismic forces in the E-W direction. As noted above, this special reinforced concrete shear wall, as well as all of the other shear walls in the building, is part of the SFRS.

Table 6-21 Summary of Design Axial Forces, Bending Moments, and Shear Forces at Base of Shear Wall on Line 4 (SDC D)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	645	0	0
Live (L)	186	0	0
Seismic (Q_E)	0	± 4,280	± 143
Load Combinations			
$1.4D + 1.7L$	1,219	0	0
$1.4D + L + Q_E$	1,089	4,280	143
$0.7D + Q_E$	452	-4,280	-143

Design for shear.

Reinforcement requirements. Special reinforced structural walls are to be provided with reinforcement in two orthogonal directions in the plane of the wall in accordance with ACI 21.6.2. The minimum reinforcement ratio in both directions is 0.0025, unless the design shear force is less than or equal to $A_{cv}\sqrt{f_c'}$, where A_{cv} is the gross area of concrete bounded by the web thickness and the length of wall in the direction of analysis. In such cases, minimum reinforcement in accordance with ACI 14.3 for ordinary walls must be provided. For the wall in this example, $A_{cv} = 10 \times 240 = 2{,}400$ in.^{2}, so that

$$A_{cv}\sqrt{f_c'} = 2{,}400 \times \sqrt{5{,}000}/1{,}000 = 170 \text{ kips} > V_u = 143 \text{ kips}$$

Therefore, assuming that No. 5 bars or less will be used in the wall, the minimum ratio of vertical reinforcement is 0.0012 and the minimum ratio of horizontal reinforcement is 0.0020 (ACI 14.3). According to ACI 14.3.5, maximum reinforcement spacing is $3h = 3 \times 10 = 30$ in. or 18 in. (governs).

Two curtains of reinforcement are required in a wall when the in-plane factored shear force exceeds $2A_{cv}\sqrt{f_c'} = 2 \times 170 = 340$ kips. In this case, two curtains are not required, since 340 kips > $V_u = 143$ kips. However, two curtains are provided in this example.

The minimum required area of vertical reinforcement per foot of wall is $0.0012 \times 10 \times 12 = 0.14$ in.^{2} Assuming No. 4 bars in two curtains, required spacing s is

$$s = \frac{2 \times 0.20}{0.14} \times 12 = 34.3 \text{ in.} > 18 \text{ in.}$$

Try 2 curtains of No. 4 vertical bars spaced at 18 in.

Similarly, the minimum area of horizontal reinforcement per foot of wall is $0.0020 \times 10 \times 12 = 0.24$ in.^{2} Assuming No. 4 bars in two curtains, required spacing s is

$$s = \frac{2 \times 0.20}{0.24} \times 12 = 20.0 \text{ in.} > 18 \text{ in.}$$

Try 2 curtains of No. 4 horizontal bars spaced at 18 in.

Shear strength requirements. ACI Eq. 21-7 is used to determine nominal shear strength V_n of structural walls:

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

where $\alpha_c = 2$ for ratio of wall height to length $h_w / \ell_w =$ 42/20 = 2.1 > 2 (ACI 21.6.4.1).

For 2 curtains of No. 4 horizontal bars spaced at 18 in. ($\rho_n = 0.40/(10\times18) = 0.0022$):

$$\phi V_n = 0.85\times2{,}400\times[2\sqrt{5{,}000} + (0.0022\times60{,}000)]/1{,}000$$

$$= 558 \text{ kips} > V_u = 143 \text{ kips} \qquad \text{O.K.}$$

where $\phi = 0.85$ for walls with $h_w / \ell_w > 2$ (ACI 9.3.4(a)). Note that V_n = 657 kips is less than the upper limit on shear strength, which is $8A_{cv}\sqrt{f_c'} = 8\times170 = 1{,}360$ kips (ACI 21.6.4.4). Therefore, use 2 curtains of No. 4 bars @ 18 in. on center in horizontal direction.

Reinforcement ratio ρ_v for the vertical reinforcement must not be less than ρ_n when $h_w / \ell_w \leq$ 2.0 (ACI 21.6.4.3). Since h_w / ℓ_w = 2.1 > 2, use minimum reinforcement ratio of 0.0012.

Try 2 curtains of No. 4 bars spaced at 18 in. on center in the vertical direction (ρ_v = 0.0022 > 0.0012).

Design for axial force and bending.

Figure 6-8 contains the interaction diagram of the wall reinforced with 2-No. 4 vertical bars @ 18 in. As seen from the figure, the wall is adequate for the load combinations in Table 6-21.

Special boundary elements.

The need for special boundary elements at the edges of structural walls is evaluated in accordance with ACI 21.6.6.2 or 21.6.6.3. The displacement-based approach in ACI 21.6.6.2 is utilized in this example. In this method, the wall is displaced at the top an amount equal to the expected design displacement; special boundary elements are required to confine the concrete when the strain in the extreme compression fiber of the wall exceeds a critical value. This method is applicable to walls or wall piers that are essentially continuous in cross-section over the entire height and designed to have one critical section for flexure and axial loads.

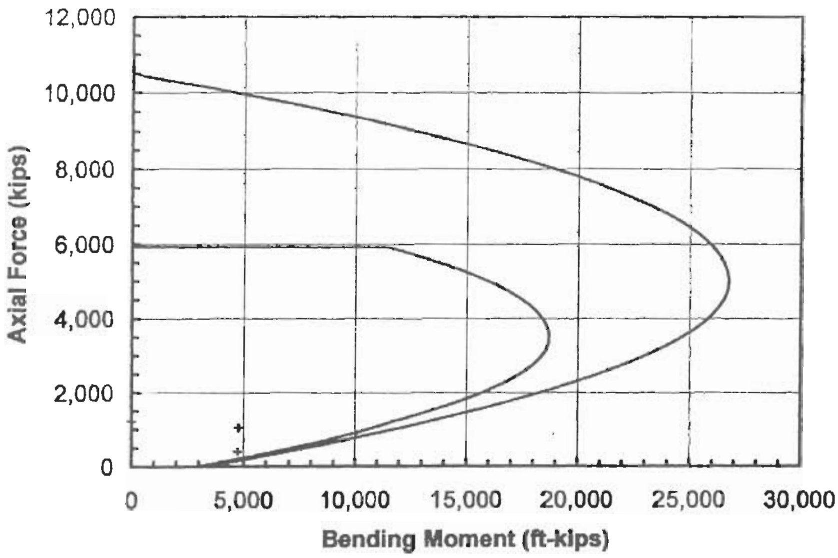


Figure 6-8 Design and Nominal Strength Interaction Diagrams for Shear Wall No. 5 (SDC D)

Compression zones are to be reinforced with special boundary elements where (Eq. 21-8):

$$c \geq \frac{\ell_w}{600(\delta_u / h_w)}, \quad \delta_u / h_w \geq 0.007$$

where c = distance from extreme compression fiber to the neutral axis per ACI 10.2.7 calculated for the factored axial force and nominal moment strength, consistent with the design displacement δ_u, resulting in the largest neutral axis depth

ℓ_w = length of entire wall or segment of wall considered in the direction of the shear force

δ_u = design displacement

= total lateral displacement expected for the design-basis earthquake as specified by the governing code

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

The lower limit on the quantity δ_u / h_w is specified to require moderate wall deformation capacity for stiff buildings.

In this example, ℓ_w = 20 ft = 240 in., h_w = 42 ft = 504 in., δ_u is equal to δ_x from Table 6-18, which is 0.95 in. at the top of the wall, and δ_u / h_w = 0.002 < 0.007 (use

0.007). Therefore, special boundary elements are required if c is greater than or equal to $240/(600 \times 0.007) = 57.1$ in.

The distance c to be used in Eq. 21-8 is the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with the design displacement δ_u. From a strain compatibility analysis, the largest c is equal to 38.5 in. corresponding to a factored axial load of 1,089 kips and nominal moment strength of 12,379 ft-kips, which is less than 57.1 in. Therefore, special boundary elements are not required.

Where special boundary elements are not required according to ACI 21.6.6.2, the provisions of ACI 21.6.5.5 must be satisfied. These provisions require boundary transverse reinforcement for walls with moderate amounts of boundary longitudinal reinforcement to help prevent buckling of the longitudinal bars.

Boundary transverse reinforcement in accordance with ACI 21.4.4.1(c), 21.4.4.3, and 21.6.6.4(a) must be provided at the ends of walls where the longitudinal reinforcement ratio at the wall boundary is greater than $400/f_y$. In this example, the reinforcement layout is similar to that shown in the lower portion of ACI Figure R21.6.6.5, i.e., uniformly distributed bars of the same size. The longitudinal reinforcement ratio ρ at the wall boundary is:

$$\rho = \frac{2 \times 0.2}{10 \times 18} = 0.0022 < \frac{400}{60{,}000} = 0.0067$$

Thus, no special transverse reinforcement is required in the wall. Also, according to ACI 21.6.6.5(b), the horizontal reinforcement terminating at the edges of the wall need not have a standard hook engaging the edge reinforcement since V_u = 143 kips < $A_{cv}\sqrt{f'_c}$ = 170 kips. However, a standard hook is used at the ends of the horizontal reinforcement to develop these bars.

The vertical reinforcement is also adequate to resist the required out-of-plane seismic force prescribed in IBC 1620.1.7.

Splice length of reinforcement.

Class B lap splices for the No. 4 vertical bars in the wall are determined in the same way as for the No. 4 vertical bars in Section 6.2.4.2 of this publication.

Use a 1 ft-4 in. splice length for the No. 4 bars.

Reinforcement details for this wall are the same as those shown in Figure 6-3.

6.5 REFERENCES

6.1 International Conference of Building Officials, *Code Central – Earthquake Spectral Acceleration Maps*, prepared in conjunction with U.S. Geological Survey; Building Seismic Safety Council; Federal Emergency Management Agency; and E.V. Leyendecker, A.D. Frankel, and K.S. Rukstales, Whittier, CA (CD-ROM).

6.2 *BRDLAT*, an internal Basic program developed by Blue Ridge Design, Inc., Winchester, VA

6.3 American Society of Civil Engineers, *ASCE Standard Minimum Design Loads for Buildings and Other Structures*, ASCE 7-98, Reston, VA, 2000.

6.4 Precast/Prestressed Institute, *Standard Connections for Precast and Prestressed Concrete Construction*, Chicago, IL 2003.

6.5 International Code Council, *2002 Accumulative Supplement to the International Codes*, Falls Chuch, VA, 2002.