NIST GCR 11-917-10

NEHRP Seismic Design Technical Brief No. 5

Seismic Design of Composite Steel Deck and Concrete-filled Diaphragms

A Guide for Practicing Engineers

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National Institute of Standards and Technology
U.S. Department of Commerce

NEHRP Seismic Design Technical Briefs

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This NIST-funded publication is one of the products of the work of the NEHRP Consultants Joint Venture carried out under Contract SB134107CQ0019, Task Order 10253. The partners in the NEHRP Consultants Joint Venture are the Applied Technology Council (ATC) and the Consortium of Universities for Research in Earthquake Engineering (CUREE). The members of the Joint Venture Management Committee are James R. Harris, Robert Reitherman, Christopher Rojahn, and Andrew Whittaker, and the Program Manager is Jon A. Heintz.

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NIST GCR 11-917-10

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Prepared for
U.S. Department of Commerce
Engineering Laboratory
National Institute of Standards and Technology
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August 2011

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Disclaimers

This Technical Brief was prepared for the Engineering Laboratory of the National Institute of Standards and Technology (NIST) under the National Earthquake Hazards Reduction Program (NEHRP) Earthquake Structural and Engineering Research Contract SB134107CQ0019, Task Order 10253. The statements and conclusions contained herein are those of the authors and do not necessarily reflect the views and policies of NIST or the U.S. Government.

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The policy of NIST is to use the International System of Units (metric units) in all of its publications. However, in North America in the construction and building materials industry, certain non-SI units are so widely used instead of SI units that it is more practical and less confusing to include measurement values for customary units only in this publication.

Cover photo – Steel deck prior to reinforced concrete placement.

How to Cite This Publication

Sabelli, Rafael, Sabol, Thomas A., and Easterling, Samuel W. (2011). "Seismic design of composite steel deck and concrete-filled diaphragms: A guide for practicing engineers," *NEHRP Seismic Design Technical Brief No. 5*, produced by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, MD, NIST GCR 11-917-10.

1. Introduction

Building structures are typically composed of horizontal spanning elements, such as beams and floor and roof decks; vertical elements, such as columns and walls; and foundation elements. Together these elements comprise an integral system that resists both vertical and lateral loads. Seismic design of building systems entails controlling the building displacements, typically by providing resistance to the inertial forces generated by the acceleration of the building mass. Often the great majority of the load is derived from the mass of the roof and floor systems themselves, and resistance is composed of a continuous lateral load path from these spanning elements to vertical elements that have lateral resistance (e.g., walls, braced frames, moment frames), which in turn deliver the forces to the foundation.

The first segment of this load path is composed of the diaphragm system. This system is typically conceived of as spanning horizontally between the vertical elements of the lateral load-resisting system. Without this element of the load path there would be no resistance to the movement of the distributed building mass, and thus large movements, and perhaps collapse, would result. Thus, diaphragms are a critical component of seismic design and must be properly designed to ensure adequate performance. Additionally, diaphragms serve a number of other functions in providing structural stability and resistance to lateral loads, as discussed in Section 2.

This Guide addresses the design of diaphragms composed of steel beams and steel deck with concrete fill. In passing, the Guide addresses some issues related to the design of diaphragms with non-composite (bare) steel deck, but a future Technical Brief devoted entirely to bare steel-deck diaphragms is anticipated.

The National Earthquake Hazards Reduction Program (NEHRP) Seismic Design Technical Brief No.3, *Seismic Design of Cast-in-Place Concrete Diaphragms, Chords, and Collectors*, includes a great deal of useful information on diaphragm design in general. In order to maximize the utility of this Technical Brief as a stand-alone design reference work, some material is duplicated here, however, this material is integrated with a treatment of conditions beyond the scope of the reinforced concrete diaphragm Technical Brief, such as semirigid and flexible diaphragms.

This Guide covers seismic design issues pertaining to Seismic Design Category B up through Seismic Design Category F. As Seismic Design Category A is exempt from seismic design, it is not specifically addressed, although many of the diaphragm analysis and design methods described herein are applicable to the design of diaphragms to resist wind forces and provide structural integrity in Seismic Design Category A buildings.

Sidebars in the Guide

Sidebars are used in this Guide to illustrate key points and to provide additional guidance on good practices and open issues in analysis, design, and construction.

Items not covered in this document

A number of important issues related to diaphragm design are not addressed in this document; these include:

- Formed concrete diaphragms on steel members (these are addressed in *Seismic Design Technical Brief No.3: Seismic Design of Cast-in-Place Concrete Diaphragms, Chords, and Collectors*);
- Out-of-plane wall support and design of sub-diaphragms;
- Design of open-web joists as chords or collectors;
- Ramp issues in parking garages;
- Saw-tooth roofs and similar discontinuities;
- Detailed treatment of steel-deck only systems;
- Strut-and-tie analysis methods; and
- Expansion joints and seismic separation issues.

The design forces and analysis requirements for diaphragms are contained in ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* (ASCE 2010, herein referred to as ACSE 7). ASCE 7-10 is the latest published version of that standard, though in a particular case at the time a reader may consult this Guide, a jurisdiction may reference the previous (2005) edition in its code regulations. The forward-looking approach here in this Guide will facilitate its use over the next several years, because ASCE 7-10 has been adopted into the 2012 edition of the *International Building Code* (IBC 2012, herein referred to as IBC), which establishes general regulations for buildings. The 2012 IBC adoption of ASCE 7-10 has no modifications relevant to composite or concrete-filled steel deck diaphragm design.

Component strengths are determined using ANSI/AISC 360 *Specification for Structural Steel Buildings* (AISC 2010, referred to here as AISC 360) for steel and composite members. ANSI/AISC 341 *Seismic Design Provisions for Structural Steel Buildings* (AISC 2010b, herein referred to as AISC 341) contains additional requirements, including limitations and quality requirements. The IBC adopts both of these standards.

The design in-plane shear strength of concrete-filled or unfilled steel deck can be determined by calculation, or it may be done by testing and subsequent development of an evaluation report. Historically, two approaches have commonly been used to calculate the in-plane shear strength. These approaches are described in the Steel Deck Institute *Diaphragm Design Manual* (SDI 2004, referred to here as SDI DDM, with SDI DDM03 citing the third edition) and the *Seismic Design of Buildings - TI 809-04* (USACE 1998.) Neither is a design code, however IBC recognizes the SDI DDM. Note that TI 809-04 often called the Tri-Services Manual, was superseded by UFC 3-310-04, *Seismic Design for Buildings* in 2007 and updated in 2010 (UFC 2010.) The specific design information that appears in TI 809-04 for diaphragms does not appear in UFC 3-310-04. A consensus standard for steel deck diaphragms that is predominately based on the SDI DDM03 is under development by the American Iron and Steel Institute. In cases where the designer wishes to ignore the presence of steel deck in concrete-filled systems, the in-plane strength of the concrete above the top flange of the deck is evaluated using Building Code Requirements for Structural Concrete and Commentary (ACI 2008, herein referred to as ACI 318). The attachment of the slab to the steel framing would then need to be addressed using one of the other documents, as ACI 318 does not explicitly address this condition. (References to the building code in this Guide refer to the editions cited above.)

Together these documents comprise the building code requirements applicable to composite deck and steel deck diaphragms. While each of these documents has been developed or revised over numerous cycles to work with the others, there nevertheless exist ambiguities, and engineering judgment is required in their consistent application. This Guide is intended to address these ambiguities and to provide guidance on the appropriate design of composite deck and steel deck diaphragms. While numerous respected practitioners, researchers, and other authorities have been consulted, this Guide represents only the opinion of the authors on matters not explicitly defined by building codes, design standards, or design manuals, and other interpretations may be reasonable.

This Guide was written for practicing structural engineers and is intended to provide guidance in the application of code requirements for the design of diaphragms in steel systems. This Guide will also be useful to others wishing to apply building code provisions correctly, such as building officials, and to those interested in understanding the basis of such code provisions and of common design methods, such as educators and students.

This Guide begins by generally discussing the role of diaphragms (Section 2), identifying the components of diaphragms (Section 3), and proceeding to the behavior of diaphragms (Section 4). Next the Guide describes the building analysis necessary to obtain appropriate diaphragm design forces (Section 5), and the analysis of the diaphragm itself (Section 6). The Guide proceeds to detailed guidance on the design of diaphragm components (Section 7). Additional requirements are given in Section 8, and constructability concerns are discussed in Section 9. References are listed in Section 10. Section 11 contains a list of notations, abbreviations, and a glossary. Section 12 provides credits for figures contained within this document.

2. The Roles of Diaphragms

2.1 Typical Conditions

Diaphragms serve multiple roles to resist gravity and lateral forces in buildings. **Figure 2-1** illustrates several of these roles for a building with a podium level at grade and with below-grade levels. The main roles include:

- *Transfer lateral inertial forces to vertical elements of the seismic force-resisting system* – The floor system commonly comprises most of the mass of the building. Consequently, significant inertial forces can develop in the plane of the diaphragm. One of the primary roles of the diaphragm in an earthquake is to transfer these lateral inertial forces, including those due to tributary portions of walls and columns, to the vertical elements of the seismic force-resisting system.

- *Resist vertical loads* – Most diaphragms are part of the floor and roof framing and therefore support gravity loads. They also assist in distributing inertial loads due to vertical response during earthquakes.

- *Provide lateral support to vertical elements* – Diaphragms connect to vertical elements of the seismic force-resisting system at each floor level, thereby providing lateral support to resist buckling as well as second-order forces associated with axial forces acting through lateral displacements. Furthermore, by tying together the vertical elements of the lateral force-resisting system, the diaphragms complete the three-dimensional framework to resist lateral loads.

- *Resist out-of-plane forces* – Exterior walls and cladding develop out-of-plane lateral inertial forces as a building responds to an earthquake. Out-of-plane forces also develop due to wind pressure acting on exposed wall surfaces. The diaphragm-to-wall connections provide resistance to these out-of-plane forces.

- *Transfer forces through the diaphragm* – As a building responds to earthquake loading, lateral shears often must be transferred from one vertical element of the seismic force-resisting system to another. The largest transfers commonly occur at discontinuities in the vertical elements, including in-plane and out-of-plane offsets in these elements. **Figure 2-1** illustrates a common discontinuity at a podium slab. The tendency is for a majority of the shear in the vertical elements above grade to transfer out of those elements, through the podium slab, and to the basement walls. Large diaphragm transfer forces can occur in this case.

- *Support soil loads below grade* – For buildings with subterranean levels, soil pressure bears against the basement walls out-of-plane. The basement walls span between diaphragms or between a diaphragm and the foundations, producing compressive reaction forces at the edges of the diaphragms.

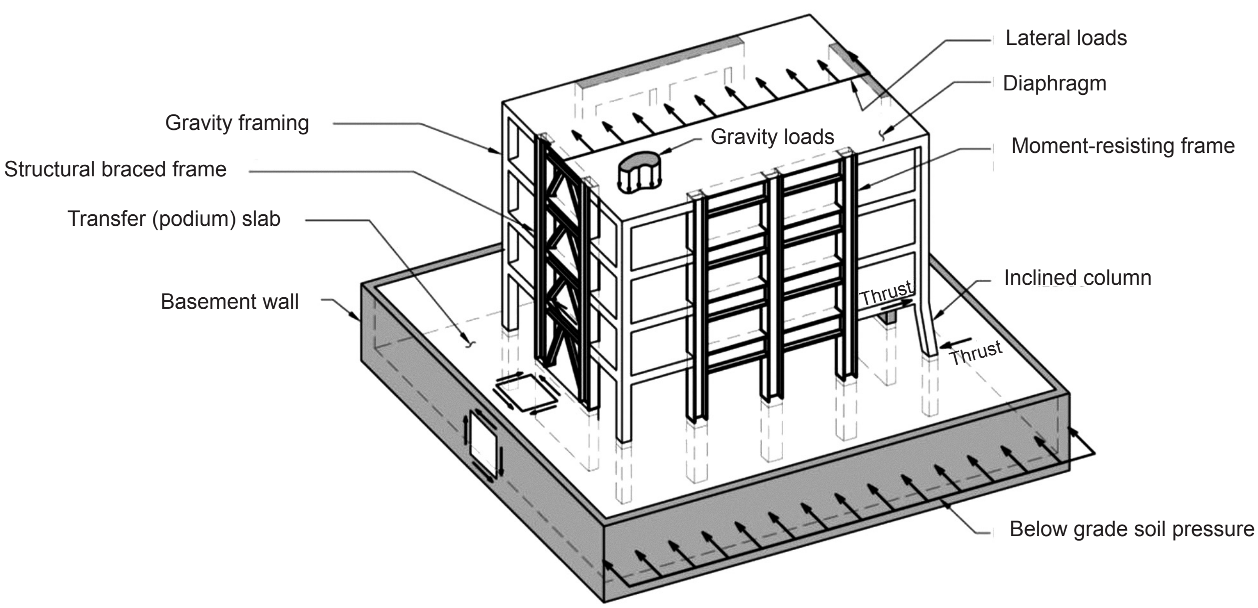


Figure 2-1 – Roles of diaphragms.

2.2 Additional Functions

Diaphragms also serve a number of specialized functions, which include:

- *Redistribution of loads around openings* – For buildings with stairway openings, mechanical shafts, elevator shafts, and other large openings such as atria, the diaphragm assists in redistributing lateral forces around the openings and to the lateral force-resisting elements.

- *Redistribution of forces due to torsion* – Some architectural configurations result in torsional response due to the application of lateral forces. Diaphragms with sufficient strength and stiffness are capable of distributing forces to the lateral force-resisting elements. Relatively flexible diaphragms generally do not facilitate the distribution of lateral forces due to torsion.

- *Resist thrust from inclined and offset columns* – Architectural configurations sometimes require inclined or offset columns, which can result in large horizontal thrusts acting within the plane of the diaphragms, due to gravity and overturning actions. Additionally, vertical columns become somewhat inclined when the building undergoes significant drift. The thrusts can act either in tension or compression, depending on orientation of the column and whether it is in compression or tension. The diaphragm or components within it need to be designed to resist these thrusts.

Chapter 12 of ASCE 7 classifies a number of horizontal structural irregularities, such as reentrant corners, diaphragm discontinuities, and torsional irregularities that may impact the response of a diaphragm and must be considered by the designer. While designers often attempt to evenly distribute vertical elements of the seismic force-resisting system throughout the footprint of the diaphragm, portions of the diaphragm without vertical seismic elements may sometimes exist and extend a considerable distance from the main body of the diaphragm. These diaphragms cantilever horizontally from the bulk of the diaphragm and need to be carefully evaluated by the designer. Generally speaking, aspect ratios associated with flexural behavior, e.g., aspect ratios greater than 1.5 to 2, may require diaphragm chords to develop the tension component of flexural demand. Although not a code requirement, the importance of maintaining an integral load path suggests that the magnitude of the chord force assumed in design should be sufficient to maintain elastic behavior under all but the largest earthquakes. Use of building code seismic demands amplified by Ω_0 , the system overstrength factor, is one approach to accomplish this goal. The development of these chord members may extend a considerable distance into the main body of the diaphragm. In addition, the cantilevered diaphragm's aspect ratio may result in significant horizontal displacements at the extreme edges that are not accurately captured by analytical models that assume essentially rigid body response.

Another common condition that demands the attention of the designer is where a chord or collector is laterally unbraced over a significant distance, such as at openings in the diaphragm around its perimeter, as shown in **Figure 2-2**, or as a part of a bridge connection between adjacent segments of diaphragm. In these conditions, the effect of the unbraced length on the available compression strength must be considered.

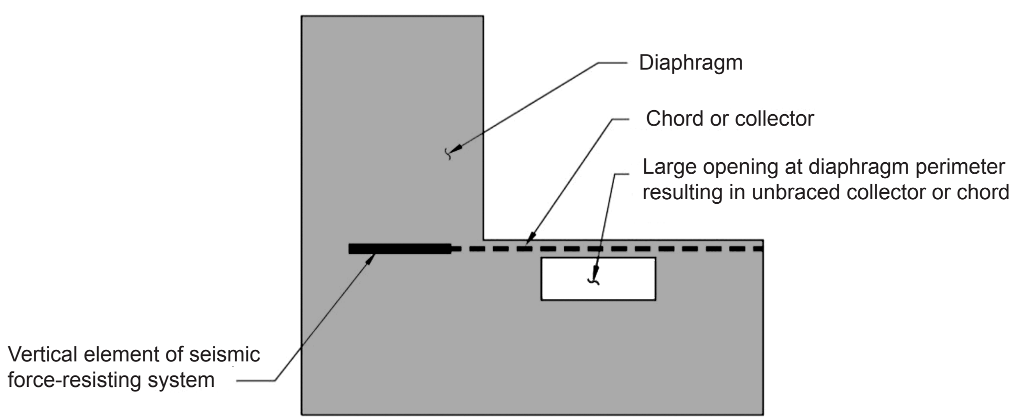


Figure 2-2 – Unbraced collector or chord.

3. Diaphragm Components

Diaphragms consist of several components, each of which must be considered as part of the strength determination and response. These components consist of the deck (bare steel deck or composite slab), chords, collectors (also known as drag struts) and fasteners used to attach the deck to the perimeter framing members. The Glossary in Section 11 defines the specific meanings of these and other terms used in this Guide. The diaphragm is commonly idealized as a beam spanning horizontally as shown in **Figure 3.1**. The supports for the beam are the vertical elements of the lateral load-resisting system, such as braced frames, moment frames, or walls. The top and bottom flanges of the beam, referred to as chords, are made up of the steel framing at the perimeter of the floor. The web of the beam is the deck, which provides the shear resistance. The perimeter fasteners are required to tie the deck to the chords and to the vertical elements of the lateral load-resisting system. In cases where the vertical elements of the lateral load-resisting system are not the full depth of the diaphragm, the framing members along the frame line function to "collect" the diaphragm shears and deliver these forces to the frame.

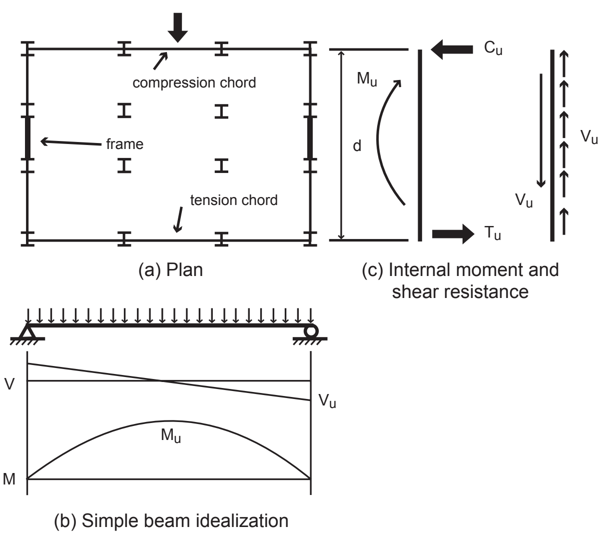


Figure 3-1 – Diaphragm component idealization.

The chord components must be designed to resist the tension or compression generated from beam behavior. These members may be considered non-composite, or, depending on the means by which they are attached to the deck, the designer may wish to consider their composite strength. Given that the diaphragm typically involves multiple bays, the chord members must be tied together through the connections to columns. This results in an axial component of force through the connection that must be considered.

Collectors, or drag struts, occur where the deck forces are transferred to a frame line over a partial length, that is, where the beams that are part of the braced or moment frame do not extend the full depth of the diaphragm. This is illustrated in **Figure 3.1** at the outer frame lines. The remaining spandrel members in **Figure 3.1** are attached to the deck through fasteners collecting inertial forces from the deck and in turn delivering those forces to the frame members. These collector members must transfer the forces to each other across their connections to the columns. Collector forces are illustrated in **Figure 3.2**.

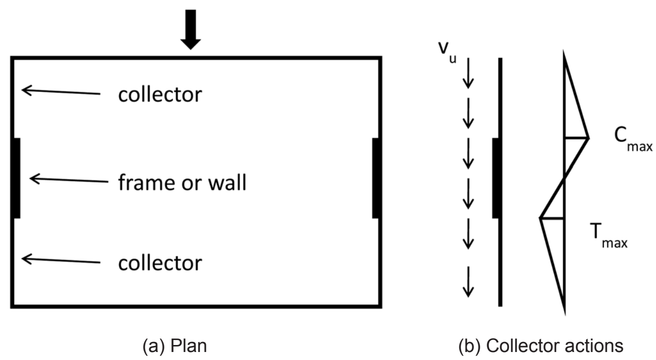


Figure 3-2 – Diaphragm collectors.

The diaphragm deck may consist of either an unfilled steel deck (typical for roofs) or a filled steel deck, or composite steel deck and concrete-filled diaphragm. In all cases, the steel deck consists of individual deck sheets, seam (or stitch) fasteners at the edges of sheets, and structural fasteners at locations of deck support.

The seam fasteners are important to the shear behavior of the unfilled steel deck. Their role in the filled deck diaphragm is less important. They are critical at the construction stage in filled diaphragms, but after the concrete has cured, they are not the mechanism for load transfer, which is achieved through bond of the steel deck to the concrete over essentially the entire diaphragm area. More details and design approaches are given in Chapter 7.

4. Diaphragm Behavior and Design Principles

4.1 Dynamic Response of Buildings and Diaphragms

From fundamental studies of structural dynamics (e.g., Chopra 2005) we know that the dynamic response acceleration of an oscillator subjected to earthquake ground motion varies with time and that the peak value will be a function of the vibration period of the structure as compared to the frequency content of the input motion. The smooth design response spectrum of ASCE 7 (2010) (**Figure 4-1**) represents this period dependency.

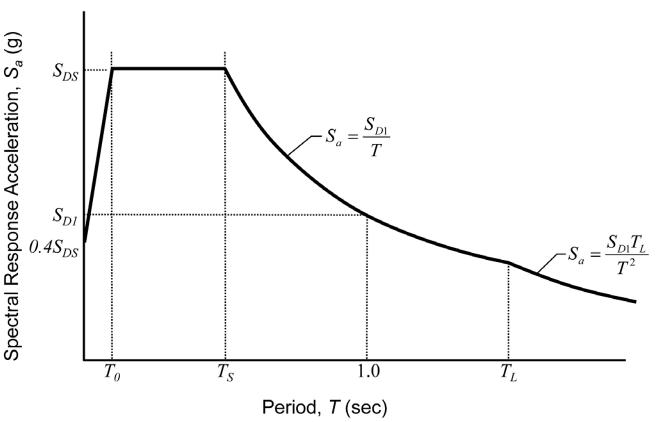


Figure 4-1 – ASCE 7 design response spectrum showing spectral response acceleration as a function of vibration period.

In **Figure 4-1**, the term S_{DS} represents the design spectral acceleration for short-period structures. The peak ground acceleration, which is the spectral acceleration at $T = 0$, has a building code-prescribed value of $0.4S_{DS}$. The ratio of the peak response acceleration to the peak ground acceleration is called the response acceleration magnification. Its value for short-period structures is 2.5 in this design spectrum.

The behavior of multi-story buildings is similar. Studies of building responses (e.g., Shakal et al. 1995; Rodriguez et al. 2007) show response acceleration magnification also is around 2.5 for buildings responding essentially elastically. For buildings responding inelastically, a lower response acceleration magnification generally is obtained.

One important observation about multi-story buildings is that, because of higher-mode response, the different floors trace out different acceleration histories. Each floor should be designed to resist the inertial force corresponding to the peak response acceleration for that floor, but it would be overly conservative to design the vertical elements of the seismic force-resisting system for the sum of all the individual peaks, because each floor reaches its peak response at a different time during the dynamic response. Thus, two different sets of design forces commonly are specified for design:

- One set of design forces, F_x, is applied to the design of the vertical elements of the seismic force-resisting system.
- A second set of design forces, F_{px}, is applied to the design of the diaphragms.

Figure 4-2 shows these sets of loads.

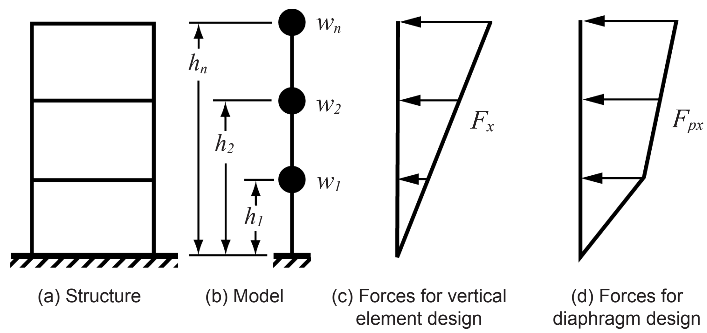


Figure 4-2 – Design forces for vertical elements and diaphragms.

In addition to resisting inertial forces (tributary mass times floor acceleration), diaphragms also must be able to transfer forces between different vertical elements of the seismic force-resisting system. For example, frames and walls acting independently have different displacement profiles under lateral loads; if interconnected by a diaphragm, the diaphragm develops internal forces as it imposes displacement compatibility (**Figure 4-3**). Almost all buildings have force transfers of this type that should be investigated and considered in design. In general, considering only diaphragm actions due to F_{px} is not sufficient.

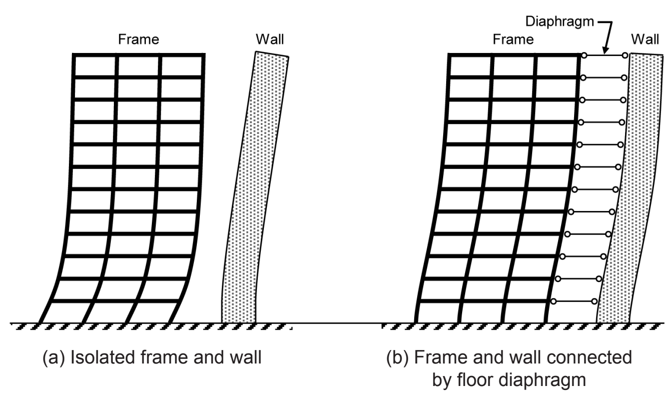


Figure 4-3 – Diaphragms develop transfer forces by imposing displacement compatibility between different vertical elements of the seismic force-resisting system.

Sometimes the largest diaphragm transfer forces are at offsets or discontinuities of the vertical elements of the seismic force-resisting system. **Figure 4-4** shows a common example involving vertical discontinuities at (a) a setback in the building profile and (b) a podium level at grade. If the diaphragm is modeled as a rigid element in a computer analysis of the building, unrealistically large transfer forces might be calculated at the levels of the discontinuities. At such locations, and sometimes for one or several floors below the discontinuity, modeling diaphragm flexibility can produce more realistic estimates of design forces in the diaphragms and the vertical elements. See the sidebar on nonlinear dynamic analysis for additional guidance.

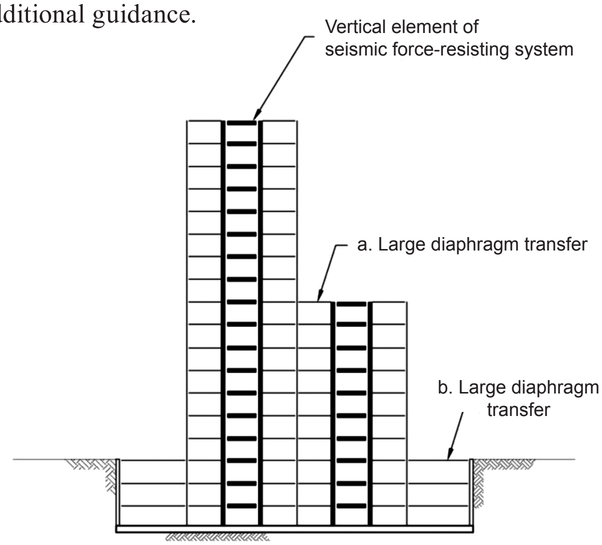


Figure 4-4 – Diaphragm transfer forces at irregularities in the vertical elements of the seismic force-resisting system.

Some building configurations result in longitudinal splits in the diaphragm (e.g., in a parking structure), which, when combined with significant separation of the vertical elements of the seismic force-resisting system, result in relatively long and narrow diaphragm segments. These segments tend to respond dynamically somewhat independently of the vertical system, and the lateral deformations in these flexible diaphragms can result in diaphragm displacements significantly exceeding displacements of the vertical elements (Fleischman et al. 2002). Design of gravity columns needs to accommodate the increased displacements. In addition, the inclined ramps can act as unintended diagonal braces, causing significant axial load in the diaphragm. Seismic joints can relieve this action if provided at every level. The situation is further complicated when a single column supports sloping diaphragms from two different levels such that the relative stiffness of the vertical elements changes, which results in concentrations of lateral force in the stiffer (i.e., shorter) elements that are not always considered by designers. See SEAOC (2009).

Nonlinear Dynamic Analysis Guidance

Nonlinear response history analysis is sometimes used to determine forces in collectors and their connections as an alternative to using Ω_0-amplified forces F_x and F_{px}. This approach can be acceptable if the analysis and design approach are established to achieve the intent of the code that the collector not be the weak link in the load path. Collector demands should be determined using an appropriate estimate of the materials properties, for example, expected materials properties. They should consider the variability in demands produced by different earthquake ground motions. Likewise, the collector design strengths should be determined using a conservative estimate, such as the design strength using nominal material properties and the code strength reduction factor. The model should also reflect a realistic proportioning of the relative stiffness of the column and the diaphragm. By appropriate selection of the design demands and strengths, an acceptably low probability of failure can be achieved.

See NEHRP Seismic Design Technical Brief No. 4, *Nonlinear Structural Analysis for Seismic Design* (Deierlein et al. 2010).

4.2 Intended Diaphragm Performance

Diaphragms are not intended to be the main source of inelastic deformation in a structure. Significant inelastic response in the seismic force-resisting system, if it occurs at all, should be restricted to the vertical elements. Thus, one of the principles of earthquake-resistant design is to maintain a relatively stiff and damage-free diaphragm that is capable of tying together the vertical elements of the seismic force-resisting system. This goal is implied in the design approach contained in ASCE 7, but is not an explicit requirement; it is possible that fully code-compliant designs may not meet this proportioning goal. To achieve this goal, seismic design of a diaphragm should clearly identify the load paths to the vertical elements, and should aim to provide diaphragm strength along that load path at least equal to the maximum force that can be developed by the vertical elements. Design approaches for steel deck diaphragms appear to have been relatively effective in limiting diaphragm damage, with few cases of observed damage following earthquakes.

4.3 Diaphragm Classification

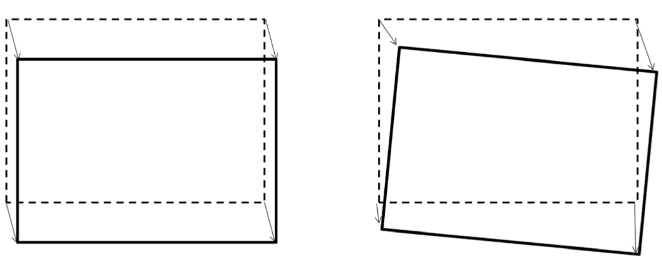
An appropriate analysis of the lateral system requires the correct assessment of the relative stiffness of the diaphragm compared to the vertical elements of the lateral force-resisting system. Traditionally, diaphragms have been idealized as either "flexible," "rigid," or "semirigid." These idealized designations affect the manner in which the designer distributes the design lateral force to be resisted by various vertical elements in the lateral force-resisting system as well as whether the diaphragm is capable of distributing load via torsion. A three-dimensional building analysis is necessary to determine the horizontal distribution of forces when diaphragms are rigid or semirigid.

The definitions of flexible and rigid diaphragms are given in ASCE 7 § 12.3.1. It is possible for a diaphragm to be idealized in different ways depending upon the direction under consideration. It is also possible that the prescriptive definition of a rigid diaphragm may not be applicable in situations where relatively large seismic demands need to be transferred through the diaphragm.

In a structure with a rigid diaphragm, the distribution of seismic demand to the walls and frames at a given level generally depends upon the relative rigidity of these elements. In essence, the diaphragm is considered to be significantly more rigid than the vertical elements. In a structure with a flexible diaphragm, the distribution of seismic demand to the vertical elements in the lateral force-resisting system generally depends on the tributary area of the diaphragm supported by the vertical element. The tributary area is considered from a lateral load perspective rather than a vertical load perspective. In the flexible case, the vertical elements are considered to be significantly more rigid than the diaphragm.

A rigid diaphragm sustains little in-plane deformation (relative to the walls and frames) due to its dynamic response, and all points in it experience essentially the same displacement about a given axis (see **Figure 4-5a**). Torsional response can exaggerate the displacement of some edges of a rigid diaphragm (see **Figure 4-5b**). A diaphragm is considered rigid for the purpose of distributing story shear and torsional moment to lateral force-resisting elements when the lateral deformation of the diaphragm is less than or equal to two times the average story drift. (This method does not specifically address variations in drift due to sloping diaphragms or due to the maximum drift being at a location other than diaphragm midspan.) ASCE 7 permits concrete-filled steel deck with span-to-depth ratios of three or less to be idealized as rigid.

On the other hand, a diaphragm is idealized as flexible for the purpose of distributing story shear when the computed maximum in-plane deflection of the diaphragm under lateral load is more than two times the average story drift. An example of a flexible diaphragms is a bare steel deck spanning between braced frames or shear walls.



(a) Translation (b) Translation with rotation (torsion)

Figure 4-5 – Plan view of rigid diaphragm.

In some cases, the diaphragm in-plane stiffness relative to the vertical elements of the lateral force-resisting system does not permit it to be idealized as either a rigid or a flexible diaphragm. In these cases, the structural analysis must explicitly include consideration of the in-plane stiffness of the diaphragm. Examples of semirigid diaphragms include steel deck diaphragms spanning between moment frames.

In real buildings, diaphragms often have multiple spans, and may have very different proportions in orthogonal directions. They may also have different lateral force-resisting systems. A diaphragm that is flexible in one direction will not be effective in sharing torsion between the orthogonal systems, even if the diaphragm is rigid or semirigid in the orthogonal span. Conversely, for a diaphragm to be idealized as rigid it must meet the criteria for both directions; otherwise semirigid modeling is necessary. Diaphragms are always permitted to be treated as semirigid. Where some, but not all, spans meet the criteria for flexible diaphragms, semirigid modeling of those spans is necessary.

4.4 Building Code Provisions Pertaining to Diaphragms

Seismic design of diaphragms is required for all buildings in Seismic Design Category B through F. ASCE 7 § 12.10 contains the main provisions for diaphragm design. The design must consider the lateral seismic forces F_x, the diaphragm design forces F_{px}, and any transfer forces associated with response under the design seismic loading.

The lateral seismic forces F_x are determined in the analysis of the vertical elements of the seismic force-resisting system (**Figure 4-6a**). These forces typically are determined from the Equivalent Lateral Force Procedure (ASCE 7 § 12.8), although the Seismic Response History Procedure of ASCE 7 Chapter 16 also can be used. These lateral seismic forces represent the overall building design lateral force distribution, the sum of which results in the design base shear, V.

As discussed in Section 4.1, the lateral seismic forces, F_x, do not necessarily reflect the estimated maximum force induced at a particular diaphragm level. Thus, ASCE 7 § 12.10.1.1 also requires the diaphragm to be designed for the diaphragm design force, F_{px} (**Figure 4-6b**). This force is defined by three equations in ASCE 7. The first (Equation 12.10-1) is constructed from the story forces, F_x, from the Equivalent Lateral Force vertical distribution (Equation 12.8-12) and thus relates to the design base shear. The second two equations (Equations 12.10-2 and 12.10-3) are minima and maxima and relate to the response-spectrum parameter S_{DS}, and they are not dependant on the system design coefficients R and Ω_0.

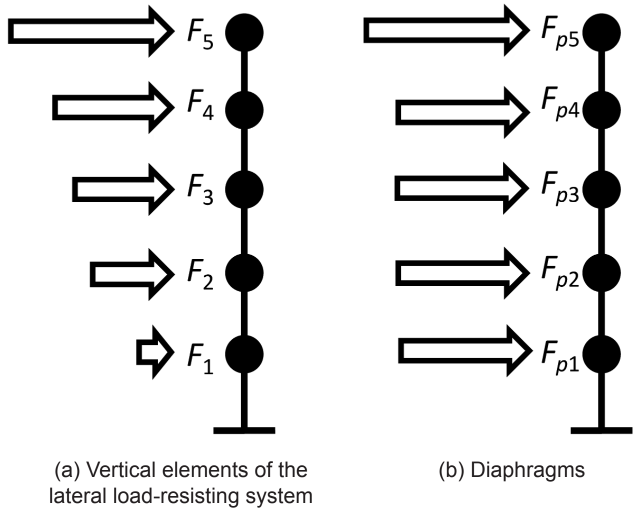


Figure 4-6 – Design forces.

Associated design requirements typically are evaluated by applying F_{px} to one floor at a time rather than all floors simultaneously, using either simplified models (see Section 6.1 of this Guide) or the overall building model. Approaches to diaphragm analysis that consider the overall building model are discussed by Sabelli et al. (2009).

Diaphragms must also be designed to resist the transfer forces that develop due to framing interaction among different vertical elements, including horizontal offsets or changes in mass and stiffness of the vertical seismic force-resisting system.

Failure of some connections between concrete diaphragms and concrete shear walls in the 1994 Northridge earthquake triggered code changes for collectors that apply to all diaphragms. According to ASCE 7 § 12.10.2, collectors must be capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces. For structures assigned to Seismic Design Categories C through F, collectors, including splices and connections to resisting elements, are required to resist the load combinations, including overstrength factor Ω_0. The lateral seismic load effect is either $\Omega_0 F_x$ or $\Omega_0 F_{px}$, whichever produces the larger effect. Note that if F_{px} is governed by ASCE 7 Equation § 12.10-2 or § 12.10-3, those forces need not be amplified by Ω_0, as the forces are not derived from the system response coefficient, R. In such cases, the force from Equation 12.10-2 or 12.10-3 should be compared to Ω_0 times the force from ASCE 7 Equation § 12.10-1 to determine the governing loading for the component.

Transfer forces are added to those calculated using § 12.10.2 and are subject to either the overstrength factor or the redundancy factor, depending upon the specific condition being evaluated. See Section 5.1 of this Guide.

Once the forces are determined using the ASCE 7 provisions, the diaphragm and its components must be designed to resist all shears, moments, and axial forces, including effects of openings and other discontinuities. For buildings assigned to Seismic Design Categories D through F, which are those structures subject to the more stringent seismic design requirements in ASCE 7 due to their occupancy and design earthquake ground motion, the provisions of ACI 318 § 21.11 apply for the design of the concrete portion of the diaphragm, and AISC 341 Chapters A, B, and D apply to the steel portions. For buildings assigned to Seismic Design Categories B or C, the general requirements in ACI 318 Chapters 1 through 18 and in AISC 360 apply.

Sections 5 through 9 of this Guide provide guidance on how to analyze and design the diaphragm and different diaphragm related components.

4.5 Alternative Approaches

There are alternative approaches to determine design forces in diaphragms and collectors. In performance-based seismic design, a nonlinear response history analysis typically is used. Ground motions sometimes are selected and scaled with a focus on the fundamental period of vibration. However, because peak diaphragm accelerations and design forces may be determined by higher vibration modes, the selection and scaling procedure needs to properly address those vibration modes. Different ground motions will result in differing degrees of response in a given structure, and thus multiple ground motions are typically used to analyze the response of structures. Diaphragm accelerations and the resulting forces can be determined directly from the analysis. If diaphragms are modeled as finite elements, section cuts can be used to track diaphragm forces at each time step. As with any computer model, the engineer should exercise good judgment when using the results of a nonlinear response history analysis.

Capacity-based design is another way to determine diaphragm design forces. This approach uses the maximum force that can be delivered to a diaphragm by the framing system as the design force, and the reliable resistance as the design strength. The approach may be suitable for roof diaphragms, especially those governed by the minimum diaphragm force of ASCE 7 Equation 12.10-2, and for levels with significant transfers, such as podium slabs, but it is overly conservative for other levels. Where capacity-based design is used, engineers should consider expected material properties, multiple failure mechanisms, multiple load patterns, and appropriate strength calculation procedures so that the resulting demands and capacities safely cover the range of combinations that can be reasonably expected.

5. Building Analysis and Diaphragm Forces

The seismic forces developed in a diaphragm are dependent on the overall response of the building to the ground motion. The building period, the seismic load-resisting system employed, the relative stiffness of the vertical elements of the lateral load-resisting system, discontinuities, the vertical position of the diaphragm in the building, and the building's torsional response all play a role in determining appropriate diaphragm design forces. In most circumstances, the diaphragm cannot be designed until there is at least some preliminary analysis of the overall building structure.

Additionally, the design of the diaphragm should be done consistently with the design approach for the building as a whole. This typically entails design with the expectation of inelastic demands in the vertical elements of the lateral load-resisting system.

5.1 Consistency of Internal Diaphragm Design Forces and Design Forces for the Vertical Elements

In limited cases, diaphragms may be determinate with respect to the horizontal distribution of lateral loading. Determinate cases are those in which diaphragms are truly flexible, and those in which there are only three lines of lateral resistance (corresponding to the three degrees of freedom in the diaphragm plane; see **Figure 5-1**). In the far more common cases, the condition is indeterminate, and there exist multiple load paths for seismic forces from their point of origin to the foundation. In such cases, no single load path is absolutely correct. Although elastic analysis can be used to determine the relative stiffness of each load path, and to assign the force accordingly, it should be understood that yielding of the vertical elements of the lateral load-resisting system will have a significant effect on the relative stiffness of these load paths. During an earthquake the load path may change significantly.

Nevertheless, the elastic analysis of the building serves several important purposes, among which is demonstration that there is sufficient strength and stiffness in the structure with respect to design seismic loading. Failure to provide this strength and stiffness is a serious deficiency with respect to fundamental building code requirements. The diaphragm, being a critical part of the load path, must provide sufficient capacity consistent with the load path and force distribution to demonstrate compliance with these strength and stiffness requirements. Thus, the internal forces used in the design of diaphragm components should be consistent with the forces in the vertical elements to demonstrate code compliance.

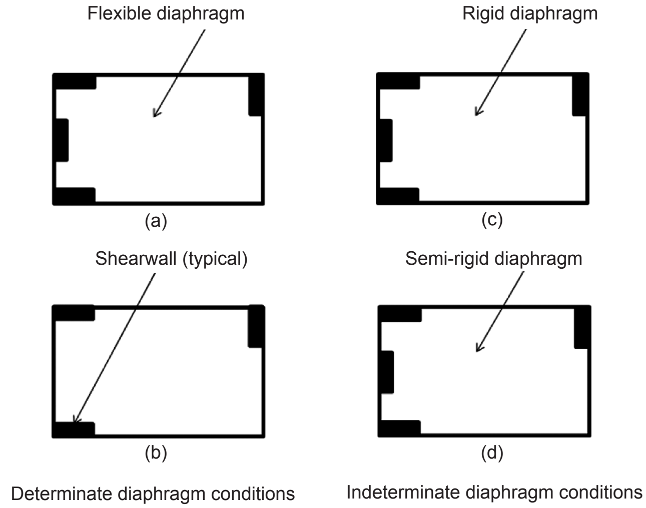


Figure 5-1 – Determinate cases are those in which diaphragms are truly flexible (Figure 5-1a), and those in which there are only three lines of lateral resistance (corresponding to the three degrees of freedom in the diaphragm plane; see Figure 5-1b). In the far more common cases, the condition is indeterminate (Figure 5-1 c and d), and there exist multiple load paths for seismic forces from their point of origin to the foundation.

5.2 Diaphragm Classification

As mentioned above, in certain circumstances the diaphragm is determinate with respect to the horizontal distribution of lateral loading. In such cases the design procedure begins with analyzing the diaphragm and applying the reaction forces to the vertical elements. Note that the forces used in the design of diaphragms other than the roof are typically larger than those used for the design of the vertical elements as explained in Section 4. **Figure 5-1** shows a number of determinate and indeterminate diaphragm conditions.

In cases in which the diaphragm support condition is indeterminate, the design sequence is reversed. It begins with a three-dimensional analysis of the building. From the results of that analysis, the corresponding forces transferred from the diaphragm to each wall or frame at each level can be obtained for each loading condition. These forces can be conceived of as the reactions of the diaphragm on its flexible supports. The procedure for obtaining diaphragm forces from a Modal Response Spectrum Analysis requires additional steps; see sidebar.

As explained in Section 4, the code-prescribed diaphragm design forces are typically larger than the corresponding story force from an Equivalent Lateral Force analysis. In indeterminate cases, diaphragms also resist transfer forces as a result of discontinuities in frames, discontinuities in frame stiffness, interaction between frames, or other dynamic characteristics of the building. Determining the appropriate

Transfer Forces and Building Analysis: The "Backstay Effect"

A common example of a building analysis that depends on transfer forces is a shearwall connection to a ground floor slab above a basement (see **Figure 5-2**). This is sometimes referred to as the "backstay effect." The analysis might show a shear diagram such as is shown in the figure, which indicates a reaction at the ground floor diaphragm greater than the total shear force in the wall (PEER 2011). This is often a difficult force to accommodate, and designers are inclined to reduce it by modeling flexibility in that slab or detailing a gap to permit some relative movement of slab and wall. However, the resulting structure may be significantly more flexible, as the shearwall might be effectively several stories taller. In this case, the building must be analyzed again.

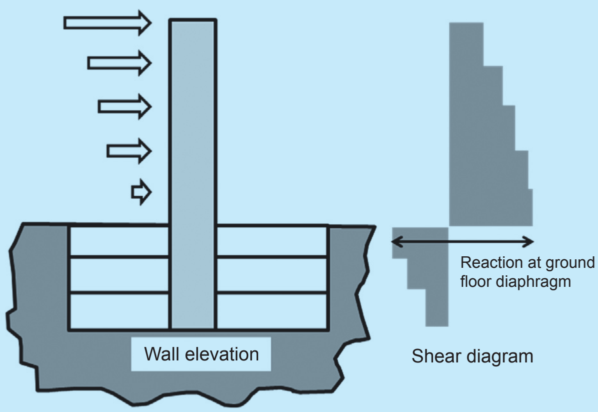


Figure 5-2 – Shearwall in building with basement.

diaphragm forces in such cases is significantly more complicated than it is for flexible diaphragms.

In some cases, transfer forces in the absence of distinct discontinuities may be considered an artifact of the analysis and not inherent in the building's response to ground motions. However, if the design of the horizontal distribution of strength and stiffness among the vertically oriented resisting elements is performed based on such an analysis, it is possible that without the ability to transfer forces, the building will have inadequate strength in some vertical elements, larger displacements than anticipated, or concentrated ductility demands in the diaphragm.

Failure to provide adequate strength and stiffness in the diaphragm to deliver those transfer forces may invalidate use of that analysis for design. Alternatively, a building analysis that does not rely on such transfer forces may be performed. In such cases, the slab will transfer shear consistent with its strength, and some shear ductility demands would be expected. With current commercially available software, this can be achieved through modeling of diaphragm flexibility.

5.3 Determination of Design Lateral Forces

The typical diaphragm design procedure presupposes that an Equivalent Lateral Force analysis has been performed, even if a Modal Response Spectrum Analysis is ultimately used to design the vertical elements. As discussed in Chapter 4, such a distribution produces appropriate design shears and overturning moments for the walls and frames, but may underestimate diaphragm inertial forces, especially at the lower levels of the building. Forces determined using ASCE 7 Equation 12.10-1 (F_{px}) reflect the acceleration of a particular diaphragm within the building. These forces are higher than the Equivalent Lateral Force story forces (F_x) at all levels below the roof. This equation is applied to the story forces obtained from an Equivalent Lateral Force analysis and thus includes the Response Reduction Coefficient, R.

This equation is given upper and lower bounds in ASCE 7 Equations 12.10-2 and 12.10-3. The lower bound corrects the potential underestimation of forces at diaphragms low in the building due to higher-mode effects. This is especially important for systems with a high Response Modification Coefficient, R, as the reduction in response is more effective in the first mode than in other modes. The upper bound often governs in systems with a low R.

Equivalent Lateral Force building analysis is performed with forces corresponding to the design base shear, and diaphragm design must be performed at a higher force level. Therefore, it is often convenient to amplify the analysis forces for the purposes of diaphragm design by a simple ratio of F_{px} to F_x. This is always appropriate for determinate structures. However, application of Equation 12.10-1 to all diaphragms within a single analysis would overestimate the shear and overturning in the walls and frames, because the higher F_{px} forces are not to be considered as simultaneous forces. Additionally, this procedure would overestimate shears in diaphragms that resist transfer forces. Such amplification of transfer forces often underestimates the forces in some components and overestimates the forces in others (Sabelli et al. 2009).

The degree to which such forces are incorrect depends on the relative magnitude of diaphragm inertial forces and transfer forces. Transfer forces are nearly always present in analyses of multi-story buildings with indeterminate diaphragms. Transfer forces are necessarily large when there are discontinuities in the system, such as discontinuous walls or frames or abrupt

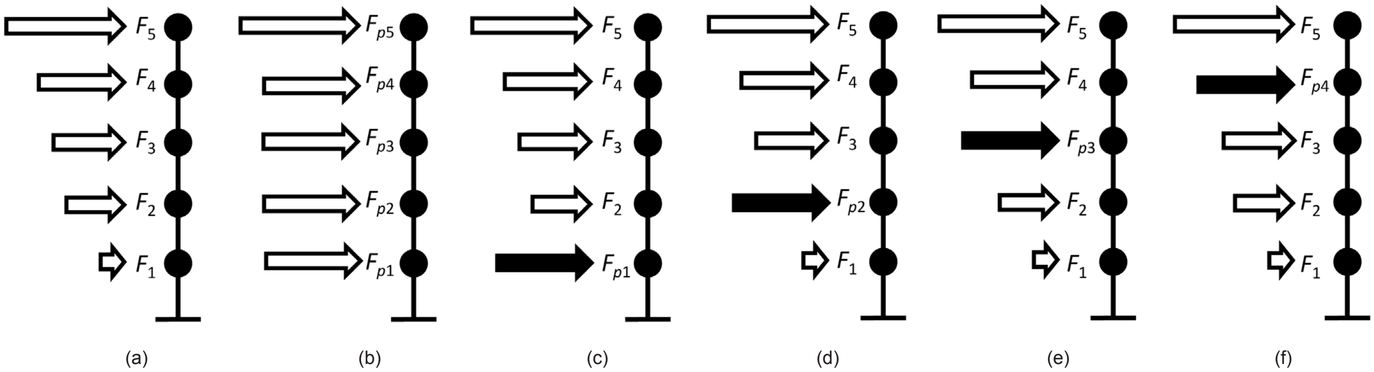


Figure 5-3 – Vertical force distribution with diaphragm force.

changes in stiffness. They also can be large in taller buildings and buildings that combine different lateral systems, such as walls and moment frames. In practice, it is difficult to determine whether the transfer forces are large enough to have a significant effect without performing an appropriate analysis or examining the reactions as described below.

One approach to analyzing the combination of transfer and inertial forces is to perform a separate building analysis for each diaphragm, substituting the diaphragm force F_{px} for the story force F_x at the level of interest. Such a force distribution is shown in **Figure 5-3**, where (a) is the Equivalent Lateral Force distribution (F_x), (b) is the set of diaphragm forces (F_{px}), and (c), (d), (e), and (f) are the combinations of story forces and diaphragm forces appropriate for evaluating the diaphragms at levels 1, 2, 3, and 4 respectively. This approach explicitly addresses the appropriate combination of transfer and diaphragm inertial forces for each diaphragm. Forces applied below the diaphragm of interest typically have little or no effect on the forces within the diaphragm.

An alternative to this was proposed by Sabelli et al. (2009) in which the transfer forces from the building analysis are not amplified but the diaphragm forces are. Use of Modal Response Spectrum Analysis has also been proposed for obtaining diaphragm reactions.

Modal Response Spectrum Analysis

Modal Response Spectrum Analysis is often required for the building analysis. In cases where it is not required, it nevertheless provides significant economy due to the reduced base shear and thus is often used. Designers may consider using the results of a Modal Response Spectrum Analysis to obtain diaphragm design forces (although such an approach is not formally addressed in ASCE 7). Story forces, as used in ASCE 7 Equation 12.10-1, do not exist in Modal Response Spectrum Analysis. Thus designers must adopt other procedures to obtain the story forces necessary for Equation 12.10-1. Additionally, the results of Modal Response Spectrum Analysis do not distinguish forces resulting from the acceleration of a diaphragm from transfer forces affecting the diaphragm.

One procedure commonly used is to design the vertical elements of the lateral load-resisting system for Modal Response Spectrum Analysis forces and perform a separate static analysis only for the purposes of obtaining diaphragm design forces. In theory, there could be significant differences in the load paths determined in the two procedures, but this is not a concern for most regular buildings.

Alternatively, designers have used Modal Response Spectrum Analysis to represent the anticipated diaphragm accelerations, without applying ASCE 7 Equations 12.10-1, 12.10-2, and 12.10-3. Such an approach would not be in strict compliance with the code.

As discussed above, diaphragm reaction forces can be obtained for each loading case being considered. For Modal Response Spectrum Analysis, reactions need to be computed for each mode and combined using an appropriate modal combination procedure. Reactions cannot be meaningfully computed using the difference between Modal Response Spectrum Analysis wall or frame shear at one level and the next. Current commercially available analysis software is typically able to calculate diaphragm reactions for each mode and provide a modal combination. It should be noted that a set of such reactions for a particular diaphragm will not be statically consistent, and a consistent set of force directions will not be provided, making certain mechanisms difficult to discern in some cases. Designers must consider whether such ambiguities significantly affect the internal diaphragm forces used in design. An alternative approach is to use the Modal Response Spectrum Analysis directly to determine design forces for diaphragm components of interest. This would entail modeling these elements so that modal forces, and combinations thereof, are determined in the analysis.

Once these reactions are obtained, an analysis of the diaphragm may be performed to obtain internal diaphragm forces, namely the shear in the deck, and the chord and collector or distributor forces, consistent with the building analysis. Such an analysis includes the reactions, the diaphragm inertial forces, the additional moment due to accidental eccentricity, and the effects of orthogonal frames in resisting torsion on the diaphragm.

Such internal diaphragm forces are consistent with the building analysis for the loading case considered. Changes in the loading case, such as application of accidental torsion in the opposite direction, will have some effect on the internal diaphragm forces.

5.4 Diaphragms and Discontinuities in the Vertical System

There are several types of discontinuities in the walls and frames that require large force transfers through the diaphragm. Cases include:

- Walls and frames that are supported by columns;
- Walls and frames that terminate at a level below the roof;
- Walls and frames that have a significant change in shear or overturning stiffness from one level to the next;
- Diaphragms that have a significant offset in the center of mass form one level to the next.

In such cases, transfer forces are typically large, and special components such as distributors may be required to resist forces being delivered into the diaphragm by the walls and frames.

5.5 Deformation Compatibility of Gravity System with Flexible and Semirigid Diaphragms

Components of the structural system and nonstructural systems are evaluated considering the expected story drifts of the building to ensure that the deformations imposed on these elements do not cause a loss of their ability to support their gravity loads. In some cases, the flexibility of the diaphragm contributes significant additional displacement which should be considered in evaluating these systems in accordance with the deformation compatibility requirements of ASCE 7 § 12.12.5.

Building Modeling Issues

In many cases explicit modeling of the diaphragm as part of a three-dimensional building structure model is advantageous. It may even be required, as in the case of semirigid diaphragms, as discussed in Section 4. Such modeling should reasonably reflect the expected behavior and also permit the determination of appropriate design forces. If such modeling is used to determine chord and collector forces, it may be appropriate to model these elements with the axial stiffness of the beams and tributary area of the deck, while simultaneously reducing the principal membrane stiffness of the deck.

Where shear in the deck is large, it is reasonable to include a factor to represent moderate cracking in the slab and other softening mechanisms of the diaphragm. This is especially advantageous in the design of ground floor diaphragms above basements and diaphragms at the top of podium levels. Research supporting the selection of this factor is lacking; designers have typically used a modification factor between 0.15 and 0.5 (Moehle et al. 2010). Such a reduction in diaphragm stiffness and the resulting change in horizontal force distribution may have a significant effect on the building response, and sensitivity studies or design envelopes may be appropriate for the design of walls and frames.

6. Diaphragm Analysis and Internal Component Forces

Once the building is appropriately modeled and diaphragm inertial and transfer forces are determined, an analysis of the internal forces within the diaphragm must be done in order to determine the design forces for the diaphragm components. Several analytical methods have been developed, and there is little guidance on the selection of a suitable one. At a minimum, the diaphragm analysis method should use diaphragm inertial and transfer forces that are consistent with those in the building analysis. This Guide provides some guidance in the selection and application of analytical methods.

6.1 Beam Analogy

Because diaphragms can be thought of as spanning between the vertical elements of the lateral load-resisting system that act as lateral supports, simple-span diaphragms are often analyzed as simple beams. **Figure 6-1** shows a simple-span diaphragm and beam model. Shear and moment diagrams are used to compute maximum diaphragm shear and chord forces:

$$V = wL / 2 \qquad \text{(Equation 6-1)}$$

$$C = wL^2 / 8 / d \qquad \text{(Equation 6-2)}$$

where C is the chord force, d is the distance between chords, L is the diaphragm span between vertical elements of the lateral load-resisting system, V is the total diaphragm shear adjacent to the line of support at the vertical elements of the lateral load-resisting system, and w is distributed, calculated as F_p/L for rectangular diaphragms with uniform mass.

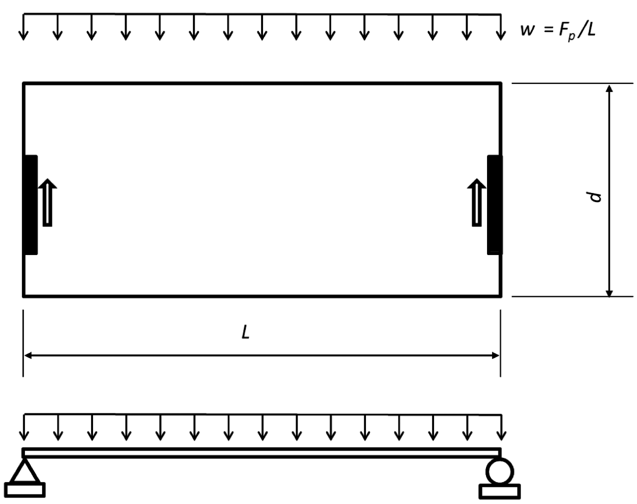


Figure 6-1 – Beam analogy for simple diaphragm.

Where the entire depth of the diaphragm is utilized to resist the shear the unit shear may be computed as:

$$v = V / d \qquad \text{(Equation 6-3)}$$

where v is the average unit shear along the depth of the diaphragm.

In some cases the diaphragm is continuous with interior walls or frames. In such cases flexible diaphragms may be analyzed similarly, with both positive moments in the spans and negative moments at the interior supports. Cantilever portions of diaphragms, being a simpler case, can be analyzed as cantilever beams regardless of their rigidity with respect to the supporting frame.

6.2 Equivalent Beam Corrected for Moment Equilibrium

As discussed in Section 5, for indeterminate diaphragm conditions a building analysis is performed to determine the diaphragm reactions (forces transferred from or to the vertical elements of the lateral load-resisting system). Thus for a given loading case, the diaphragm loading is in equilibrium with the complete set of reactions at that level, including those of the orthogonal lateral load-resisting system that resists some of the torsion. In such cases, the moment diagram constructed using the methods described above does not close due to the torsional moment resisted by the orthogonal lateral load-resisting system. In other words, the moment diagram so constructed would not come to zero at one of the diaphragm ends.

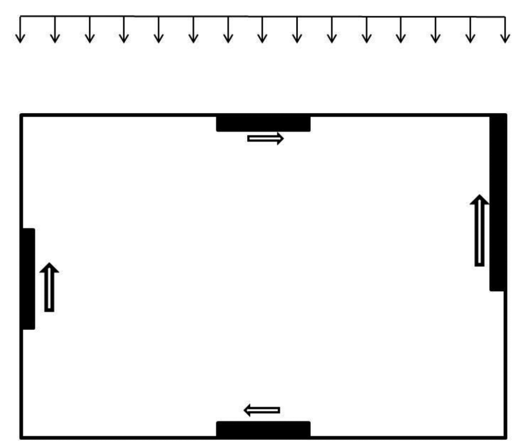


Figure 6-2 – Diaphragm reactions for an indeterminate case.

In such cases the analysis can be used with a simple adjustment: the moment imparted by the orthogonal lateral load-resisting system should be applied to the moment diagram, which in turn is used to calculate the chord forces. **Figure 6-2** shows diaphragm reactions for an indeterminate case, where the reactions of the two walls parallel to the loading are not equal due to the unequal wall stiffness. **Figure 6-3** shows the shear diagram constructed from these unequal reactions. **Figure 6-4** shows three moment diagrams. The first is the moment diagram constructed from the shear diagram without consideration of the effect of the perpendicular walls; this diagram is labeled "Moment." The second corresponds to the effect that the orthogonal frames impose ("Moment Correction"). Note that if the orthogonal walls did not align, the two chord force diagrams would differ as the "Moment Correction" would occur at different points along the chord. The third diagram ("Corrected Moment") combines the other two and shows zero moment at the diaphragm ends.

Figure 6-4 presupposes orthogonal walls on the chord lines. In such cases the chord forces are often significantly smaller than the collector forces for the orthogonal direction of loading, and the moment correction described above may have no effect on the design. If orthogonal walls are not on these chord lines, they may deliver their forces to the chords in a concentrated length, which would have an effect similar to that illustrated in **Figure 6-4** , or they may deliver their force in a more distributed fashion, as is illustrated in **Figure 6-5**. Other functions may also be used to represent the pattern of the contribution of orthogonal frames.

Where the orthogonal frames are not on the chord lines, there will be some additional shear in the diaphragm, both between chord line and orthogonal collector lines, and between the orthogonal collector lines. Refer to **Figure 6-6**. In this case the force R_A and R_B in the orthogonal walls (that is, the walls perpendicular to the direction of the inertial force considered) will be larger than the moment correction force calculated at the diaphragm chord by a factor equal to the ratio of moment arms (d/d'). This creates additional shear between these collector lines and the chord lines; the value of this shear is equal to the chord correction force. In this case:

$$M_c = F_p * L / 2 - R_2 * L \quad \text{(Equation 6-4)}$$

$$R_B = M_c / d' \quad \text{(Equation 6-5)}$$

$$C' = M_c / d \quad \text{(Equation 6-6)}$$

$$v_1' = C' / L = M_c / dL \quad \text{(Equation 6-7)}$$

$$v_2' = [R_B - C'] / L = [1 / d' - 1 / d] \, M_c / L \quad \text{(Equation 6-8)}$$

where C' is the chord correction force required to close the moment diagram, F_p is the inertial force on the diaphragm, L is the length of the diaphragm, M_c is the eccentric moment required to close the moment diagram, R_2 is the reaction at one of the shear walls parallel to the force F_p in **Figure 6-6**, R_B is the reaction at one of the shear walls perpendicular to the force F_p in **Figure 6-6**, d is the diaphragm depth, d' is the depth between orthogonal collectors resisting the eccentric moment M_c, v_1' is the additional unit shear between the orthogonal collector and the chord, and v_2' is the additional unit shear between the orthogonal collectors.

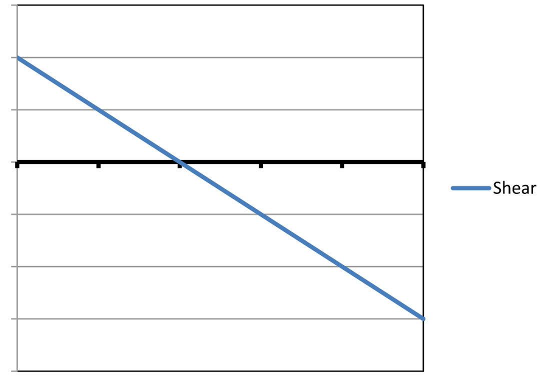


Figure 6-3 – Diaphragm shear diagram.

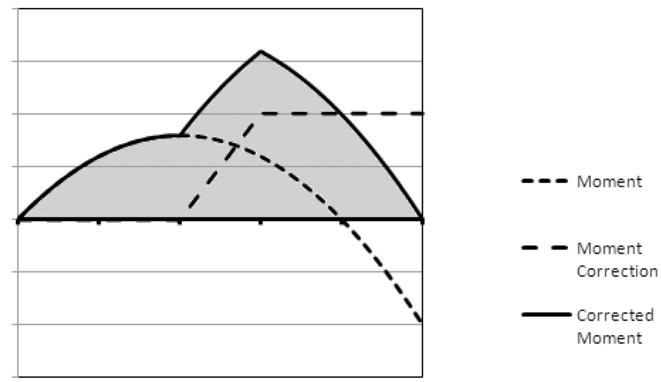


Figure 6-4 – Moment diagrams for diaphragm (concentrated force).

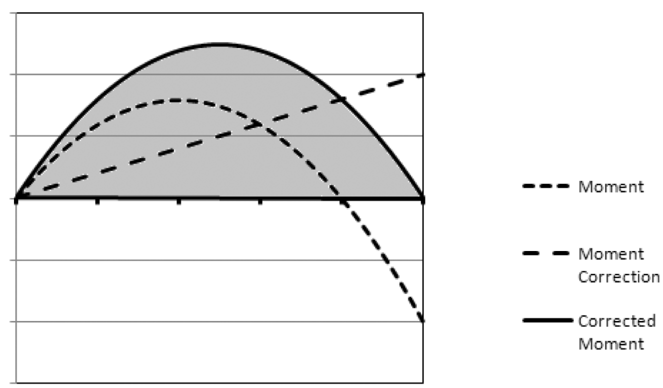


Figure 6-5 – Moment diagrams for diaphragm (distributed force).

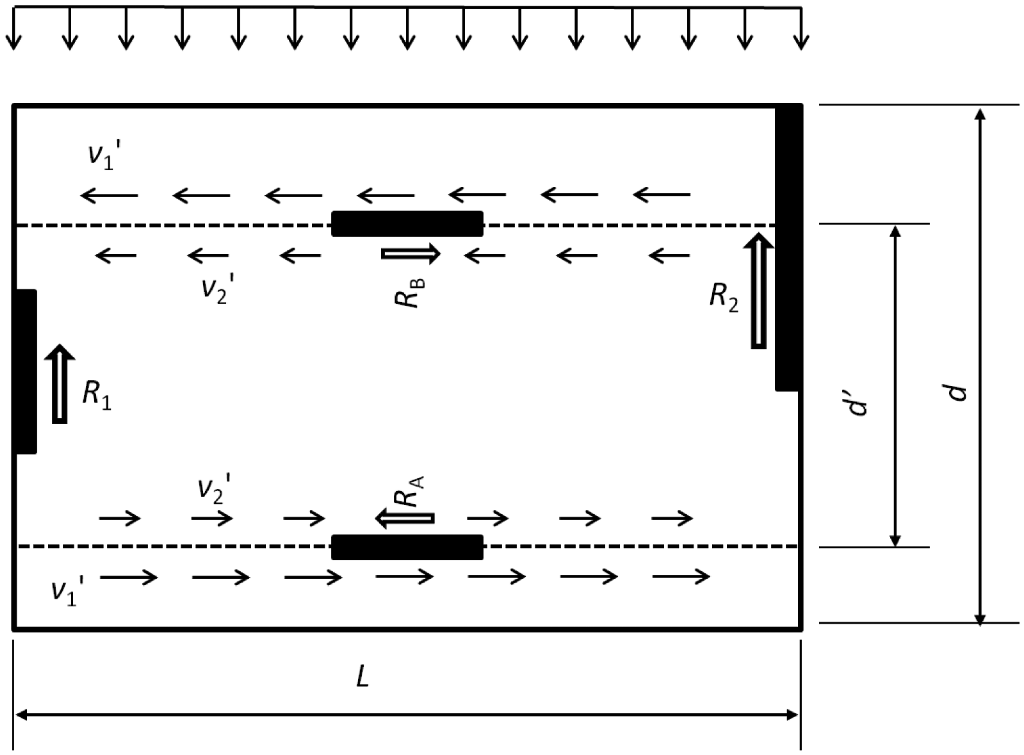


Figure 6-6 – Orthogonal collectors eccentric from chord lines.

Note that the additional unit shears computed using Equation 6-7 and Equation 6-8 must be combined with the shears computed using the shear from Equation 6-3. In some areas, the two shears will be additive and in others subtractive. In many cases, it is convenient to utilize the orthogonal collectors as chords even when they are not located at the diaphragm boundaries.

6.3 Internal Load Paths

To complete the diaphragm analysis, forces on individual components must be determined. The unit shear in the deck and chord, and the collector forces, must be calculated so that those components may be designed. The deck shear may be uniform or non-uniform; chord and collector forces may be considered to be concentrated or distributed. There is relatively little guidance in design standards and other publications for the determination or selection of appropriate distributions of shear forces along chords and collectors. At a minimum, the forces calculated in the chords and collectors should be consistent with the assumed shear distribution, as discussed below. In the absence of a rigorous analysis that includes both the nonlinear diaphragm properties and the nonlinear behavior of the system, (as well as the full range of possible ground-motion characteristics), the design in effect relies on some limited ductility in the diaphragm to permit redistribution of forces to account for the simplifications in the assumed distribution.

The shear may be considered to be uniformly distributed along the depth of the diaphragm, or concentrated near the vertical elements of the lateral load-resisting system. Uniform shear distribution along the diaphragm depth typically requires a linear accumulation of the force in a collector to deliver it to a wall or frame. **Figure 6-7** shows such a uniform shear distribution and its corresponding linear distributed axial force on a collector. For very long collector lengths, such an assumption may require significant shear ductility in the deck to accommodate the deformations consistent with axial deformations in the collector.

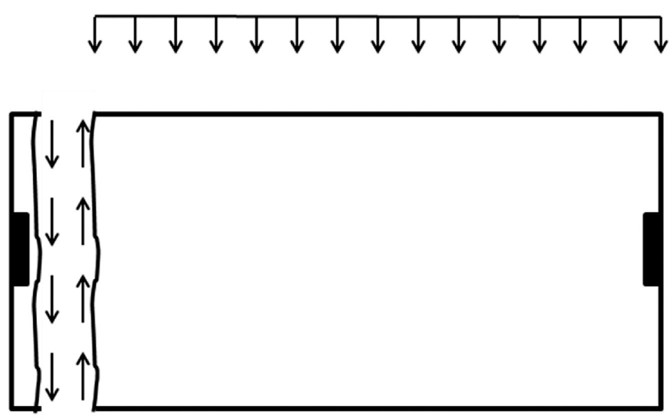


Figure 6-7 – Uniform shear model.

Alternatively, the shear may be assumed to be concentrated near the vertical elements. Such a non-uniform shear distribution corresponds to lower collector forces. For short diaphragm spans and diaphragms with relatively low shear stiffness (e.g., non-composite steel deck diaphragms) such an assumption may require significant collector axial ductility to accommodate deformations consistent with the shear deformations. Non-uniform shear is also more consistent with a distributed chord force than with a concentrated chord force at the diaphragm boundary. **Figure 6-8** shows a non-uniform shear force distribution and the corresponding distributed chord force. The shape of the shear force distribution may vary along the span of the diaphragm, resulting in other chord force distributions. For example, a secondary collector and local chords may be used to convert concentrated shear forces near a wall or frame to uniformly distributed shear in the body of the diaphragm, as illustrated in **Figure 6-9** (Moehle et al 2010). Such a shear distribution is more consistent with concentrated chords at the diaphragm boundaries.

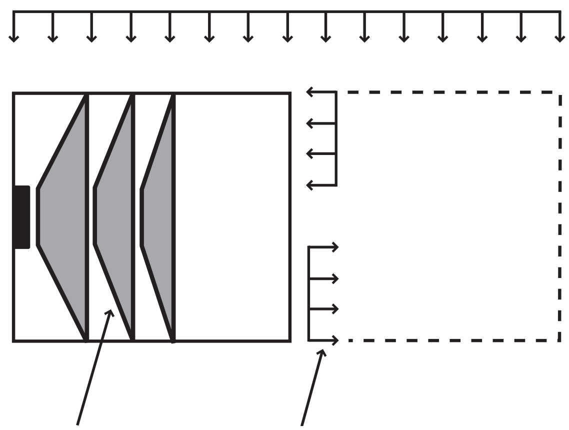


Figure 6-8 – Non-uniform shear and distributed chord forces.

As neither assumption, uniform or concentrated shear in the deck, can be verified in the absence of a rigorous nonlinear building analysis for known loads, it is necessary to provide some ductility in the components in which the demands may be underestimated. That is, when uniform shear is assumed, the deck or its connections should be detailed to provide ductility. Conversely, when concentrated shear is assumed, the collector load path should be detailed for ductility. Little guidance is available on the potential magnitude of these ductility demands. In current practice, engineers typically rely on the basic ductility measures outlined in ACI 318 and AISC 341, as discussed in Section 7.

An alternative approach is to design components for the larger forces resulting from each assumption. This is not common practice, and there is no clear evidence that such an approach is necessary.

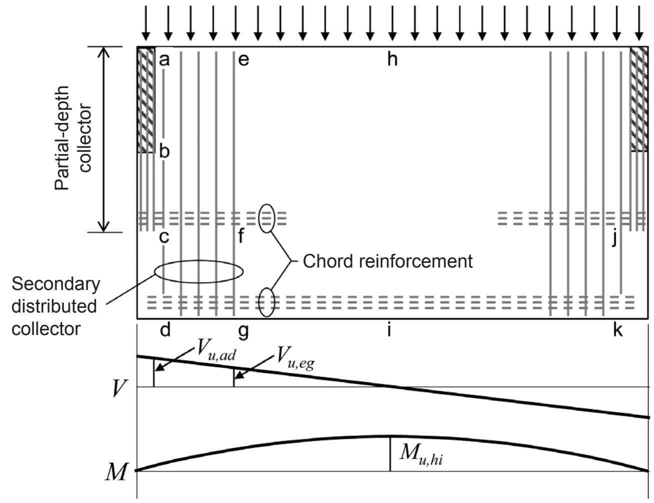


Figure 6-9 – Partial depth collector and secondary collector.

For non-composite steel deck diaphragms, flexure in the diaphragm is assumed to be resisted in a force couple consisting of axial forces in the beams at the diaphragm boundary. As mentioned above, this chord force distribution is consistent with uniform shear in the diaphragm. For composite deck diaphragms, the chord forces may also be assumed to be resisted in such beams. Alternatively, chord forces may be accommodated in areas of the deck near the diaphragm boundaries. Such a mechanism would result in a somewhat smaller effective depth of the section resisting diaphragm forces and some deviation from uniform diaphragm shear. As discussed in Section 7, the width of deck used as the chord is typically selected based on limiting the compressive stress in the deck.

Similarly, collector forces, whether they correspond to a uniform shear distribution or not, may be concentrated in beams or distributed in a composite deck. For collectors consisting of reinforcement distributed in an area of the composite deck, there is an eccentricity from the wall or frame, typically approximately half the collector width. Local reinforcement is required to resolve this eccentricity, as discussed in NEHRP Seismic Design Technical Brief No. 3 (Moehle et al. 2010).

Diaphragm Modeling Issues

As discussed in the Sidebar in Section 5, semirigid diaphragm modeling can be used to determine diaphragm component forces, either within a three-dimensional building analysis, using the appropriate diaphragm loading from ASCE 7 § 12.10 combined with the equivalent lateral forces from § 12.8, or as a separate analysis of the diaphragm itself with applied inertial and transfer forces and appropriate reactions from the building model as discussed in Section 5.

Where semirigid modeling is used, non-uniform shear stress is likely to be reported. For design purposes this stress may be integrated over a limited area; five to ten feet is often used. Larger areas of integration may require local shear ductility. This permits concentrating reinforcement in areas of expected higher demand.

Stresses corresponding to distributed chord and collector forces in composite decks may also be reported in a finite element analysis. These stresses may similarly be integrated over moderate widths to permit design of a portion of the deck as a collector.

If beams are used as the chords or collectors, deck principal membrane stiffness should be modeled as very low to permit determination of beam axial forces. It may also be necessary to reduce the axial stiffness of nearby parallel beams, unless the chord or collector force is intended to be shared on those lines.

In composite decks, diagonal compression and tension may be reported in a finite element analysis. The compression on the diagonal should be treated similarly to compression in the principal axes of the deck, with the same maximum compressive stress permitted. Tension and combined shear and tension are more conveniently evaluated in the principal axes of the deck.

While such modeling of the diaphragm is more accurate than the simpler beam analogy models discussed above, it should be noted that it shares some of the same assumptions and limitations, and results should be treated with similar caution. Generally, only one simplified loading pattern is considered, and neither nonlinear behavior in the diaphragm nor in the vertical elements of the lateral load-resisting system is directly addressed. Thus, providing moderate ductility in the diaphragm is warranted.

6.4 Local Effects at Discontinuities

Discontinuities in the diaphragm, such as openings, steps, and reentrant corners, require consideration by the designer.

Openings in a diaphragm should be located to preserve as much of the overall diaphragm as possible about any axis. Designers should avoid locating openings in such a way that narrow sections of diaphragm are used to connect different parts of the diaphragm, because of the large forces that must be transferred through the small section of remaining diaphragm. For similar reasons, clusters of diaphragm openings at reentrant corners or along a single edge of the diaphragm should be avoided. See **Figure 6-10**. It is generally better to locate diaphragm openings so that they are surrounded by substantial portions of the diaphragm or are separated as much as possible. See **Figure 6-11**. Openings in the diaphragm may be idealized similarly to openings in the web of a steel beam. The impact of the reduced area on the portions of the diaphragm that remain must be taken into account as well as the need to transfer forces around the opening. In some cases, additional walls or frames may be required in order to prevent isolating one section of the diaphragm from another.

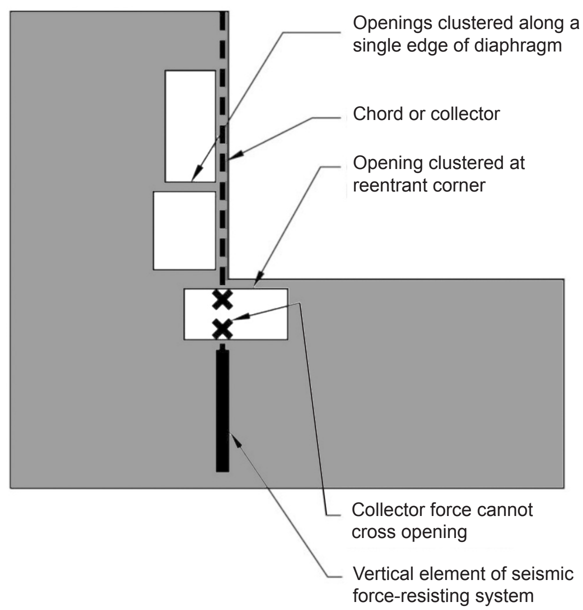


Figure 6-10 – Undesirable diaphragm openings clustered along a single edge of the diaphragm or at a reentrant corner.

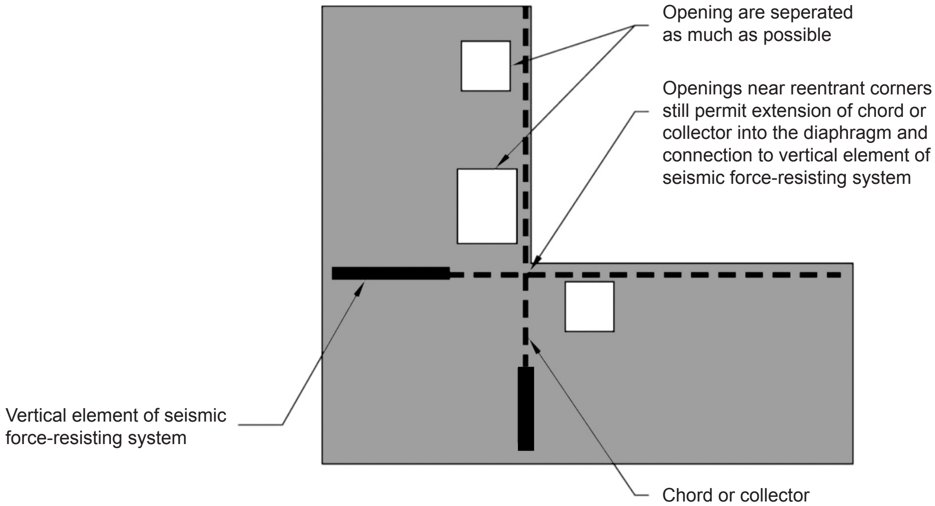


Figure 6-11 – Diaphragm openings located to minimize local diaphragm discontinuities.

When the slab elevation from one section of the diaphragm to another changes abruptly, a step in the diaphragm is created. Unless a wall or frame is located at the step, the designer must ensure that an adequate load path is available to transfer both overturning and chord forces. Depending upon the magnitude of the vertical offset, the steel beam at the step or a downturned concrete slab can be used to transfer the in-plane forces. As shown in **Figure 6.12**, when the vertical offset is significant, and walls and frames are remote from the offset, the stiffness of the out-of-plane load path should be considered in evaluating the effectiveness of transferring forces through the step.

Reentrant corners often require extending boundary members into the adjacent section of the diaphragm to adequately transfer loads through the reentrant corner. In this case, the extended portion of the diaphragm chord functions as a collector, and the magnitude of the force that needs to be transferred should at least be equal to the demand allocated to the vertical elements aligned with the collector. The collector needs to extend at least far enough across the diaphragm to develop this demand or, preferably, across the entire diaphragm.

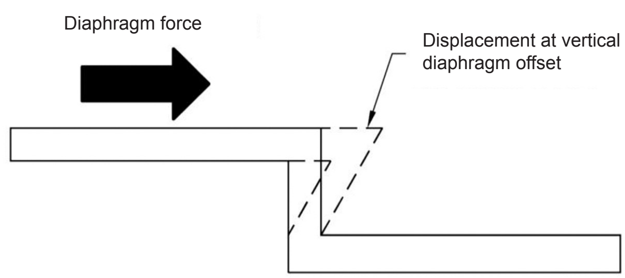


Figure 6-12 – Elevation view of the displacement at vertical diaphragm offset.

7. Component Design

The individual components described in Section 3 constitute a system, but it is traditional and more convenient to design the individual components based on their behavior and the roles they play in the diaphragm system.

Typical composite slab construction consists of embossed composite decks attached to steel framing and filled with either normalweight or structural lightweight concrete. The embossed steel deck serves as both the stay-in-place formwork and the tensile reinforcement for gravity load resistance. Secondary reinforcement, used to constrain cracks in the concrete caused by shrinkage and temperature effects, is also provided. The secondary reinforcement may be in the form of deformed bars, welded-wire reinforcement, steel fibers, synthetic macro-fibers, or a blend of steel and synthetic macro-fibers.

Both beams and girders are typically designed as composite members, following the provisions of AISC 360. The steel headed-stud anchors are detailed to provide the desired composite action between the slab and structural steel member for gravity load resistance. Although AISC 360 stipulates that composite diaphragms and collectors shall be designed, specific guidance is not provided. The commentary to § I7 of AISC 360 provides general guidance that is presented later in this section.

The procedure for the design of the members that are part of a moment or braced frame at the edge of the diaphragm is not entirely clear, nor is it described in detail in design specifications. This is because the combined load effects from gravity and lateral loads on members that have both negative and positive moments (and thus may be treated as both composite and non-composite) have not been effectively studied. In design of steel decks for gravity loads, it is common practice to not design negative moment regions in beams to act compositely. This is due to the requirement of having to transfer large shear forces between the steel shape and the slab, thus requiring large numbers of steel headed-stud anchors over a very small length of the steel shape, i.e., the length of the negative moment region. Additionally, significant steel reinforcement must be placed in the slab to resist the tension components of the composite beam moment in the slab. Both the significant numbers of steel anchors and additional reinforcement result in additional cost, and the behavior is not fully known. Thus, these members may be designed as non-composite beams over their entire length, or compositely in the positive moment region and non-compositely in the negative moment region, or some other approach may be selected by the designer.

The detailing of steel anchors is discussed in Section 7.3.

7.1 Composite Deck

Steel-framed buildings frequently utilize steel deck composite slabs for the floors. The vast majority, estimated to be as high as 90 %, of composite deck slabs are attached to framing members using steel headed-stud anchors. The composite deck can be evaluated for in-plane, or diaphragm, shear using multiple approaches. These include 1) calculation-based methods; 2) deck manufacturer evaluation reports from agencies such as ICC Evaluation services; and 3) results of full-scale in-plane diaphragm tests. Calculation-based methods will be presented here.

There is not a diaphragm design specification presently in place in the United States. At the time of this writing, the American Iron and Steel Institute (AISI) Committee on Specifications is developing and balloting a diaphragm design specification. The AISI document is based extensively on the methodology used in the Steel Deck Institute Diaphragm Design Manual (SDI DDM03.) SDI member companies publish diaphragm load tables that are based on calculation methods, test results, and combinations of those two approaches. Calculation procedures for composite diaphragms were also presented by Easterling and Porter (1994a; 1994b).

For diaphragms designed with steel headed-stud anchors, there is one primary limit state that must be considered for the composite deck, and that is the shear strength limit state in the concrete. Easterling and Porter (1994a; 1994b) reported this strength using normalweight concrete as

$$V_n = 3.2\, t_e\, b \sqrt{f'_c} \qquad \text{(Equation 7-1)}$$

Where V_n is the shear strength of the diaphragm, t_e is the effective thickness of the composite slab including a contribution from the steel deck using a transformed section approach, b is the depth of the diaphragm (inches) and f'_c is the concrete compressive strength (psi). The coefficient 3.2 was based on research findings. Diaphragm tests conducted as part of the research utilized concrete thickness that varied between 4 and 7.5 in. No primary or secondary reinforcement was used in the slabs. The agreement between the calculated and experimental strength of diaphragms in which the experimental strength was controlled by the diagonal shear strength was very good. Of the 16 diaphragms that exhibited this limit state, the mean experimental-to-calculated strength was 1.1 with a range of 0.84 – 1.29, and only 2 of the 16 tests fell below 1.0. Rewriting the equation for V_n in the form used in SDI DDM03 results in the following:

$$S_n = 0.0032\, t_e\, b \sqrt{f'_c} \qquad \text{(Equation 7-2)}$$

where S_n is the nominal shear strength for diaphragms with structural concrete fill (kips/foot), b is unit width of the deck (12 in).

The SDI DDM03 uses a methodology that incorporates a combination of the shear strength of the concrete cover thickness and a contribution of the deck-to-steel framing fasteners within the field (i.e., away from the perimeter) of the diaphragm. This approach is consistent with the non-composite steel deck diaphragm calculation model used in SDI DDM03 that will be briefly reviewed later in this section. The shear stress utilized is $3\sqrt{f'_c}$ for normal weight concrete and equations are presented that utilize a concrete thickness of 2.5 in and a concrete compressive strength of 3000 psi. Design load tables appear in the SDI DDM03 that use these limits. Once the strength is determined within the field of the diaphragm, then the edge fasteners are detailed to resist the required shear.

The basic equation used in SDI DDM03, and the current draft of the AISI diaphragm specification, is

$S_n = BQ_f / L + kbd_c\sqrt{f'_c}$ (SDI DDM03 Eq. 5.1-1) Equation 7-3

Where B is the contribution of fasteners that attach deck to steel support members (see definitions for Eq. 2.2-4 in SDI DDM03), Q_f is the structural fastener strength, L is the deck panel length, k is a coefficient that depends on the unit weight of concrete (see definitions for Eq. 5.3-1 in SDI DDM03) and equals 0.003 for concrete with a unit weight of 145 pcf, and d_c is concrete cover depth.

While the two approaches do not take the same form, both use the shear strength of the concrete as the primary mechanism of resistance. The work by Easterling and Porter used a contribution of the steel deck through a transformed section approach, and the SDI DDM03 approach utilizes the concrete cover plus a contribution of fasteners that connect the deck to the steel support members. Both methods give acceptable results, but given that the SDI DDM03 approach will likely be included in the AISI Diaphragm Standard, the recommendation is to use the SDI approach. A key point to reiterate is that the strength calculations reviewed pertain to the strength of the field of the diaphragm. Edge fasteners must be detailed to transfer the strength to the lateral load-resisting frame if that strength is to be utilized.

Both approaches discussed rely on the steel deck to serve as minimum reinforcement within the field of the diaphragm. As mentioned, none of the test diaphragms reported by Easterling and Porter contained additional reinforcement (no flexural or secondary reinforcement was used.) The welded wire fabric prescribed in the SDI DDM03 serves to mitigate the effects of shrinkage and temperature-induced cracking. If, when evaluating the diaphragm demand, the composite slab properties do not provide adequate resistance, additional shear reinforcement may be added to the slab to increase the strength. In a case such as this, the provisions of ACI 318 should be used to determine the strength within the field of the diaphragm. NIST Tech Brief No. 3 (Moehle et al. 2010) provides guidance.

Alternatives to steel headed-stud anchors for transfer of the composite deck diaphragm forces to the framing members include arc-spot welds, screws, and powder-actuated fasteners. The behavior of the composite diaphragm near the frame members is very different in these cases. The interface between the steel deck and the concrete must be utilized to transfer the forces into the framing. Near the edge of the diaphragm within approximately 36 inches, warping of the deck may result in separation of the deck and concrete, and the behavior of the composite deck near the edge of the framing becomes similar to that of a bare steel deck diaphragm. Therefore, the edge fastening along the collectors and chords must be detailed to satisfy the shear transfer demand based on the strength of the field of the diaphragm.

7.2 Steel Deck

Unfilled steel deck diaphragms are often used at roofs, but seldom at floors. Steel deck diaphragm strengths are generally determined by one of three methods: 1) calculation-based methods; 2) deck manufacturer evaluation reports from agencies such as the ICC Evaluation Service; and 3) results of full-scale in-plane diaphragm tests. The most detailed of the calculation methods, and one that is covered well in the literature, is the approach described in SDI DDM03. As mentioned previously, this method forms the basis for the AISI standard that is under development. A full description of the method is not presented here due to space limitations and the fact that the non-composite diaphragm strength is beyond the scope of this document. However, a brief description of the approach follows.

The strength and stiffness of steel deck diaphragms is based on an elastic model that considers the deck, fasteners of the deck to the structural members, and fasteners of deck sheets at their edges or seams. The controlling strength is based on the minimum calculated strength based on edge fasteners, interior panel fasteners, and corner fasteners. These provisions are described in section 2.1 of SDI DDM03, and extensive design load tables are given in Appendix V.

Steel deck manufacturers have conducted tests and analyses to develop design load tables. These load tables are often based on evaluation service reports that were developed by combining test results with various calculation methods. The tests and analyses often utilize proprietary fastening methods that may not be represented in the more general methods described in SDI DDM03. That said, the methods in SDI DDM03 are generally applicable if basic fastener strength and stiffness characteristics are available.

The strength of steel deck diaphragms is primarily a function of the panel-to-panel fasteners, as well as the interior deck-to-structure fasteners. These fasteners are less important to composite diaphragms. The panel-to-panel fasteners have little impact on composite diaphragm strength because the bond between the deck and the concrete is a more complete way to provide panel-to-panel connections, and the concrete transfers shear between adjacent panels rather than the shear being transferred through the connection. The deck-to-structural fasteners do have an influence on the composite diaphragm strength, as previously discussed and represented in the SDI DDM03 equation for filled diaphragms.

Diagonal Bracing

In some cases it may not be possible to utilize composite deck or steel deck diaphragms to transfer lateral loads around large openings. At other times, the demands in the diaphragm exceed its capacity, and additional reinforcement, additional concrete thickness, or heavier-gage steel deck are not practical methods to increase the capacity of the diaphragm. Diagonal bracing within the plane of the diaphragm may be a practical way of solving these problems. The seismic provisions in AISC 341 do not contain explicit requirements for dealing with this type of diaphragm. However, the underlying design intent for diaphragm behavior, i.e., essentially elastic behavior, as well as the design philosophy for concentrically braced frames in AISC 341, may provide some guidance to the designer. The members in this diagonally braced diaphragm should be capable of resisting the amplified seismic demand, i.e., using Ω_0, because out-of-plane bracing of the nodes cannot be provided, and the connections should be capable of developing the tensile strength of the brace member.

7.3 Shear Transfer

The strength of the diaphragm deck determined for the field of the diaphragm must be adequately transferred to the perimeter framing members if that strength is to be utilized. A variety of fasteners can be utilized to accomplish this load transfer. These include arc-spot (or puddle) welds, self-tapping/self drilling screws, powder-actuated fasteners, and steel headed-stud anchors. Additionally, the side seam fastening can be accomplished using welds, screws, or crimping, either traditional "button-punching" or proprietary seaming. As has already been mentioned, the side seam fastening has little influence on composite deck diaphragm strength.

The selection of the type and number of fasteners depends on the level of force that needs to be transferred and the relative economy of the fastener options. The use of steel headed-stud anchors is the most commonly used form of fastening deck to steel support members, given the prevalent use of composite design for flexural members. Arc-spot welds are commonly used to fasten the deck in place when it is first laid out on the frame. The welding of the steel headed-stud anchors follows. The SDI requires that the floor deck be attached on 12 inch centers and, either arc-spot welds, steel headed-stud anchors, or a combination of both are typically used. The use of screws or powder-actuated fasteners to attach deck to structural members in composite floor diaphragms is much less common.

Regardless of the choice of fasteners, the strength of welds, powder-actuated fasteners, and screws can be found in AISI S100, manufacturers test reports, and in the AISI diaphragm standard that is under development. The strength of steel headed-stud anchors can be determined using AISC 360.

In the past few years, a number of proprietary deck seam attachment methods have been developed. These are typically deck crimping mechanisms that have been evaluated with both elemental experimental tests and full-size steel diaphragm tests. The detailed results are not generally available in their basic form, but evaluation reports developed for the manufacturers reflect the diaphragm strength based on these seam fasteners. Design load tables based on the evaluation reports are available for use by the engineering community. As previously noted, the seam fastening is not particularly important to composite diaphragm behavior and strength.

The detailing of steel headed-stud anchors for combined gravity and in-plane (diaphragm) forces has been the subject of much discussion. In the typical design scenario, steel headed-stud anchors have been detailed as part of the gravity composite beams. The question then arises as to how many additional anchors are required to resist the in-plane forces. The commentary of AISC 360 provides guidance on this issue for the first time. The recommendation is to apply ASCE 7 load combinations that recognize reduced demand from live loads if in-plane loads are at a maximum, and vice-versa. The direction of shear flow is not uniformly additive for gravity and in-plane loads. Typically, half the beam receiving the in-plane forces will experience additive forces and the other half will see forces in opposite directions. The behavior of the steel headed-stud anchors is known to be ductile.

Therefore the approach described in the AISC Commentary, and as illustrated in **Figure 7-1**, is deemed to be appropriate.

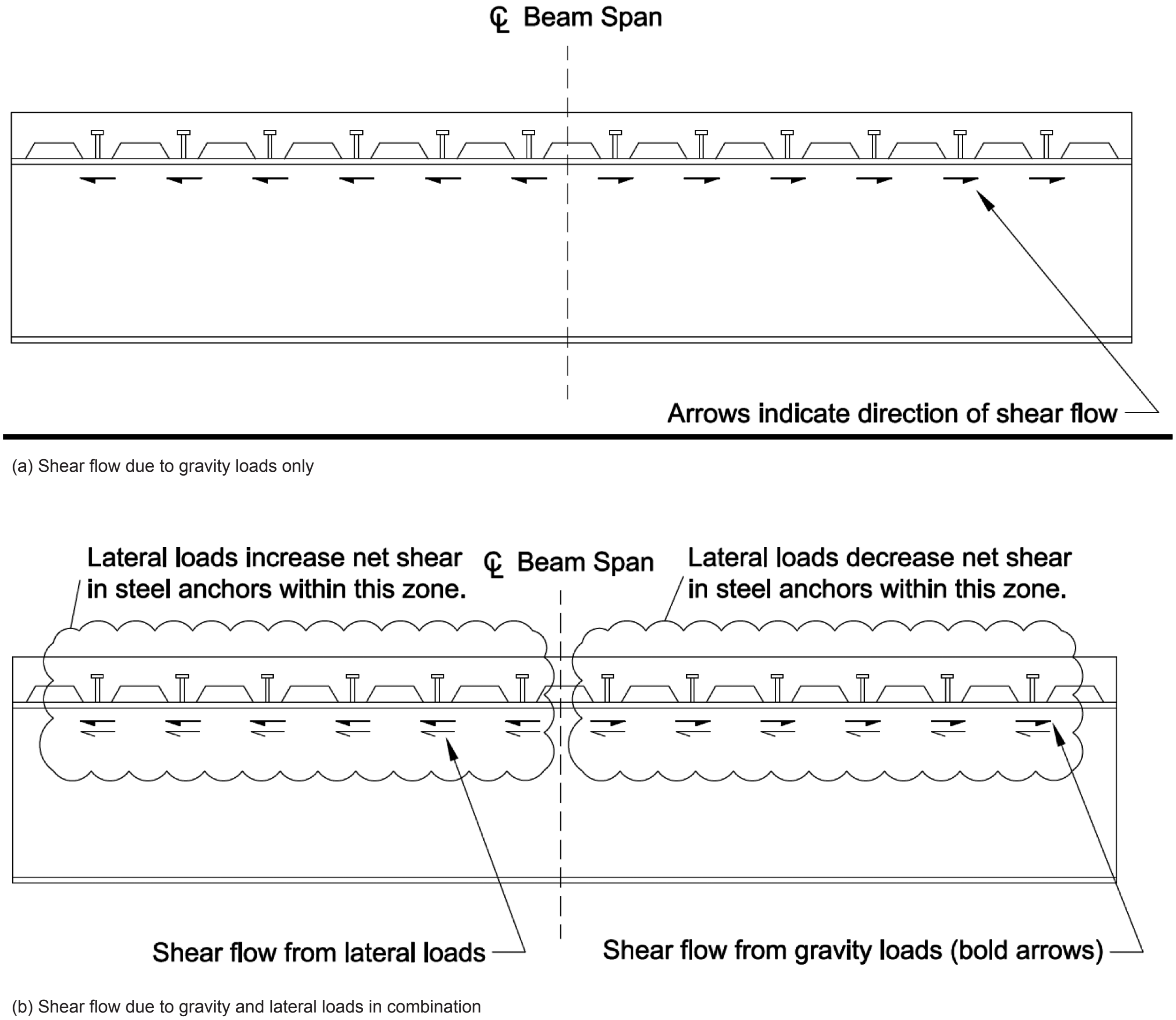


Figure 7-1 – Shear flow at collector beams (AISC 360)

AISC 341 requires that the steel headed-stud anchor strength be reduced from the values given in AISC 360 in cases in which composite moment frames, composite braced frames or composite shear walls are used as part of intermediate or special seismic force-resisting systems. A "user note" (guidance similar to commentary placed within the provisions) indicates that the 25 % reduction is not necessary for gravity and collector components in structures with intermediate or special seismic force-resisting systems designed for the amplified seismic load.

7.4 Chords and Collectors

The AISC 360 Commentary provides guidance for the first time for chord and collector elements. The guidance is not always definitive, because the behavior and performance of these elements have not been studied extensively. The best judgment of the members of the task committee responsible for AISC Chapter I is provided in the AISC commentary. That information is presented here with the added input of the authors of this Guide.

Diaphragm chords and collectors function as the means by which deck forces are transferred from fasteners to the lateral frame elements. Chord members accumulate the tension and compression forces delivering the moment of the "deep beam" model. Collectors transfer the deck forces along the edge of the diaphragm when the edge members are not directly part of the lateral frame. Both of these members can be designed assuming that they behave non-compositely or compositely.

Clearly, when steel headed-stud anchors are present, the member will respond as a composite beam-column with an eccentric axial load delivered from the plane of the deck. The behavior of these members is complex and has not been the subject of significant research. The AISC 360 Commentary recommends a simplified approach until such time as research results become available to improve the understanding of these members. This approach recommends designing the collectors and chords for the axial forces, assuming non-composite action. There is no clear guidance on how to incorporate the eccentricity.

The flexural strength of the collector and chord members can be evaluated assuming either non-composite or composite action. If non-composite beam action is chosen as the design method, the Commentary recommends providing steel anchors at a minimum level of 25 % composite action if the in-plane forces require fewer steel headed-stud anchors. This is deemed to be good practice because of concerns that the ductility demand may cause failure of the steel anchors if less than 25 % composite action is used. The beam will behave compositely in the presence of the steel anchors, even if the designer chooses to ignore the flexural composite action.

The majority of testing conducted in the U.S. on steel headed-stud anchors has not included reinforcement within the shear cone breakout area, thus the ductility achieved in these tests has relied only on the steel anchor behavior. Reinforcement is not required by AISC 360 adjacent to the steel headed-stud anchors that are detailed as part of composite beams.

Combined axial and flexural interaction can be evaluated using the provisions of Chapter H in the AISC Specification. It is recommended that the simplifying assumption of non-composite axial strength and composite flexural strength be made for the composite beam-column.

AISC 360 addresses the application of stability bracing for beam-columns, stipulating that the requirement may be addressed by adding the stability bracing required for the axial force (column bracing) to the stability bracing required for the flexural forces (beam bracing). This applies to both the required strength and stiffness of the stability bracing. This is deemed to be conservative and is appropriate given the lack of more definitive guidance.

Beam-to-column connections along the chord or collector must be designed for the combined shear, moment and axial effects delivered to the connection. The *Steel Construction Manual* (AISC 2005) and generally available technical literature have addressed a variety of connections that must transfer loads from shear, axial force, and flexure in varying relative magnitudes. Refer to parts 10, 12 and 13 in the *Steel Construction Manual* (AISC 2005.)

Ductility of Shear Transfer Devices

The ductility of diaphragms is an important design consideration. In general, composite diaphragms exhibit ductile behavior. However the fastener configuration used to transfer the diaphragm shears to collector elements can influence the ductility. In unfilled steel deck diaphragms, ductility is primarily achieved through bearing type deformations in the sheet steel around fasteners (welds, screws, powder-actuated fasteners), and through warping deformations of the deck profile. Research has shown that while welds possess the highest strength of typical fasteners, they exhibit the least ductile behavior. This may be attributable to the less than uniform attachment around the circumference of the weld. The low relative ductility of welded diaphragms, when compared to diaphragms constructed with other fastener types, has been documented in the work of Rogers, Tremblay and their colleagues (Rogers and Tremblay 2003; Essa et al. 2003.) In cases in which concrete-filled steel deck diaphragms are used and fastened by means other than steel headed-stud anchors, the behavior may be expected to be similar to unfilled steel deck diaphragms. The designer should consider the conclusions of Rogers and Tremblay that screws and powder-actuated fasteners exhibit ductility superior to that of welds without washers. Composite diaphragms typically rely on steel headed-stud anchors for the load transfer from the composite deck to the collector elements. Steel headed-stud anchors possess significant ductility and allow load sharing and redistribution along the length of attachment. This ductility is principally achieved from deformations that occur in the mild steel material used to manufacture steel headed-stud anchors.

8. Additional Requirements

8.1 Inspection

Composite steel deck diaphragms and their chords and collectors are a part of the seismic force-resisting system. Proper construction of diaphragms and their elements is paramount to ensure that the structure will perform as intended during a major earthquake.

In an effort to ensure proper construction, inspections are required for most structural steel buildings. Chapter 17 of the IBC requires that the design professional for a building prepare a statement of special inspections identifying the required inspections for construction of the building. The statement is to include inspection requirements for seismic force-resisting systems in structures assigned to Seismic Design Categories C, D, E, or F. Diaphragms and their elements provide resistance to prescribed seismic forces; therefore, diaphragms are part of the seismic force-resisting system and should be identified on the statement of special inspections. Refer to the IBC for current requirements. In addition to the requirements in the IBC, AISC 360 Chapter N and AISC 341 Chapter J describe inspection tasks for structural steel seismic systems.

IBC Tables 1704.3 and 1704.4, AISC 360 Table N6-1, and AISC 341 Tables J6, J7, J8 and J9 list the specific structural steel and reinforced concrete components that require inspection. Examples of components that require inspection include structural steel members, connections, stud shear connectors, concrete fill, and steel reinforcing.

The IBC acquires material verification of structural steel components, welding filler materials, and high-strength bolts. Inspection of steel frame joint details for compliance with approved construction documents is also required. In addition, the IBC requires continuous special inspection of structural steel welding, with the exception of single-pass fillet welds not exceeding 5/16 inch in size and floor and roof deck welding. AISC 360 requires the inspection prior to concrete placement and installation of steel deck and prior to the placement and installation of stud shear connectors.

For reinforced concrete components, the IBC requires that the size and placement of reinforcing steel be verified with periodic inspections. Periodic inspection is intended to include inspection of all completed reinforcing steel placement, including diaphragm steel. Concrete for diaphragms also requires special inspections. These special inspections often include the following, from IBC Table 1704.4:

- Verifying use of required design mixture;
- Sampling fresh concrete for strength test specimens, performing slump and air content tests, and determining concrete temperature at time of placement;
- Concrete placement;
- Maintenance of specified curing temperature and techniques;
- Grouting of bonded prestressing tendons that are part of the seismic force-resisting system.

8.2 Quality Assurance

According to IBC § 17.10.2, structural observations by a registered design professional are required for all structures assigned to Seismic Design Category D, E, or F whose height is greater than 75 ft. Shorter structures of high occupancy categories or Seismic Design Category E also require structural observations. Specific required observations for seismic force-resisting systems are not specified, but observing diaphragm components is recommended.

8.3 Bracing of Columns into Diaphragms

Columns spanning more than one floor through openings in the diaphragm are sometimes laterally braced at intermediate levels back to the adjacent diaphragm to reduce the unbraced length of the column. AISC 360 Chapter C and Appendix 6 contain design requirements for stability bracing of columns. Connections of elements bracing the columns to the adjacent diaphragm must be capable of developing these loads and providing the required level of stiffness.

9. Detailing and Constructability Issues

9.1 Detailing of Connections at Chords and Collectors

The load path along the length of a collector or chord must be maintained, even when it is otherwise interrupted by intervening girders or columns. The magnitude of the forces along the load path can be significant, and connections using only the web of the chord and collector are often not sufficient. It is not uncommon to find these members connected at their flanges and webs to intervening girders and columns. Although a bolted connection at the web has significant stiffness in the direction of the applied load, connections at column flanges using angles may be flexible enough that deformation in the connection is significant. As a result, welded connections of the chord or collector flanges across the flanges of intervening girders and columns are often used. Some engineers use added reinforcement in the slab to resist chord or collector forces, although the ultimate transfer of these forces to the vertical elements of the seismic force-resisting system may result in magnitudes that exceed the capacity of the diaphragm in the vicinity of the vertical element. It is for this reason that a direct load path to the vertical elements using structural steel framing is typically employed.

9.2 Penetrations

Isolated penetrations, such as those for conduits, pipes, and electrical junction boxes, are generally not of significance with respect to the seismic performance of the diaphragm. If a significant number of penetrations are localized in one area, the diaphragm should be analyzed and reinforced as if an opening in the diaphragm existed.

9.3 Embedded Items in Composite Deck

Conduits or other items embedded in composite deck have the potential to introduce an area of reduced structural strength as well as compromise the fire rating of the composite deck system. Most manufacturers of steel deck have obtained structural and fire rating approvals through organizations such as ICC-ES and their evaluation reports, and these evaluation reports are generally silent with respect to conduits within the composite deck and their impact on the performance of the composite deck as a diaphragm or as a fire-rated system. For this reason, it is recommended that conduit not be permitted within the composite deck system and that, instead, the conduit run beneath the steel deck.

If conduit within the composite deck cannot be avoided, § 6.3 of ACI 318 addresses items embedded in concrete from the perspective of the strength of the concrete fill. This section places limits on embedded items with an outside dimension larger than one third of the overall thickness of the slab. It also requires that they be spaced no closer than three diameters on center. Lastly, the conduit shall not significantly impair the strength of the construction. These requirements do not address the potential impact on the fire rating of the floor system, and fireproofing of the steel deck may be required to establish a reliable fire rating.

9.4 Protected Zones

Testing conducted subsequent to the 1994 Northridge earthquake showed that discontinuities within the protected zone, such as those caused by welding of stud shear connectors, have the potential to encourage premature fracture of steel subjected to significant inelastic deformation. The installation of welded stud shear connectors within the protected zone of a moment frame beam or the link in an eccentrically braced frame is not permitted by AISC 341 § I2.1. Arc spot welds as required to secure the decking are permitted because it is believed that the penetration of the weld into the base metal is sufficiently small.

9.5 Location of Construction Joints

Construction joints can create weakened planes within the diaphragm. They can also impact development and splices of reinforcement. Shear-friction reinforcement can be provided across construction joints if necessary to maintain continuity of the diaphragm in shear. The impacts to the continuity and development of chord and collector reinforcement at construction joints should also be understood. While construction joints are often detailed carefully on reinforced concrete projects, construction joints in composite deck diaphragms are often overlooked by the designer and located haphazardly by the contractor. For that reason, typical details, limitations, and instructions should be clearly detailed in the Contract Documents. Contract Documents should also require that contractors provide detailed construction joint layout drawings well in advance of concrete placement.

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11. Notations, Abbreviations, and Glossary

b	in-plane depth of diaphragm considered in the calculation of shear strength (unit width of 12 inches or entire diaphragm depth, as appropriate)
b_{eff}	effective width of collector
B	contribution of fasteners that attach deck to steel support
C	chord force
C_u	factored compressive force at section
C_{max}	maximum compression force in a collector element
C'	chord correction force required to close the moment diagram
d	diaphragm depth (distance between chords)
d_c	concrete cover depth
d'	depth between orthogonal collectors resisting the eccentric moment M_c
D	effect of dead load
e	eccentricity created by diaphragm step or depression
e_x	eccentricity of diaphragm design lateral force relative to center of rigidity
E	effect of horizontal seismic (earthquake-induced) forces
E_v	effect of vertical seismic input
f'_c	specified compressive strength of concrete
f_y	specified yield strength of reinforcement
f_l	live load factor, taken as 0.5 except taken as 1.0 for garages, areas occupied as places of public assembly, and all areas where L is greater than 100 psf
F_p	inertial force on the diaphragm
F_{px}	diaphragm design force
F_{px}	diaphragm force from ASCE 7 Equations 12.10-1, 12.10-2, and 12.10-3
$F_{px,max}$	upper limit to the diaphragm design force
$F_{px,min}$	lower limit to the diaphragm design force
F_x	story force from ASCE 7 Equations 12.8-11 and 12.8-12.
h_x	height above the base to Level x

H	effects of soil, water in soil, or other materials
I	importance factor
K	coefficient that depends on the unit weight of concrete (see definitions for Eq. 5.3-1 in SDI DDM03) and equals 0.003 for concrete with a unit weight of 145 pcf
k	distribution exponent for design seismic forces
k	coefficient that depends on the unit weight of concrete (see definitions for Eq. 5.3-1 in SDI DDM03) and equals 0.003 for concrete with a unit weight of 145 pcf
k_i	stiffness of vertical element i
L	span of diaphragm or diaphragm segment
L	diaphragm span between vertical elements of the lateral load-resisting system
L	length of the diaphragm
L	the effect of live load
L	deck panel length
M_c	eccentric moment required to close the moment diagram
M_u	factored moment
Q_f	structural fastener strength
R	response modification coefficient
R_i	reaction force in slab at vertical element i
R_A, R_B, R_1, R_2	forces in shear walls
S	effect of snow load
S_a	spectral response pseudo-acceleration, g
S_{DS}	design, 5 percent damped, spectral response acceleration parameter at short periods
S_{D1}	design, 5 percent damped, spectral response acceleration parameter at a period of 1 second
S_{DS}	design, 5 percent damped, spectral response acceleration parameter at short periods
S_m	elastic section modulus
S_n	nominal shear strength for diaphragms with structural concrete fill (k/ft)
t_e	effective thickness of the composite slab including a contribution from the steel deck using a transformed section approach
T	fundamental period of the building

T_L	long-period transition period
T_{max}	maximum tension force in a collector element
T_s	$= S_{D1} / S_{DS}$
T_0	$= 0.2\ S_{D1} / S_{DS}$
v	average unit shear along the depth of the diaphragm
v_u	factored shear stress
v_1'	additional unit shear between the orthogonal collector and the chord
v_2'	additional unit shear between the orthogonal collectors
V	design base shear
V	total diaphragm shear adjacent to the line of support at the vertical elements of the lateral load-resisting system
V_u	factored shear force
W	distributed force, calculated as F_p /L for rectangular diaphragms with uniform mass
w_{px}	the weight tributary to the diaphragm at Level x
w_x	portion of effective seismic weight of the building that is located at, or assigned to, Level x
ϕ	strength reduction factor
ρ	a redundancy factor based on the extent of structural redundancy present in
Ω_0	system overstrength factor

Abbreviations

ACI	American Concrete Institute
AISC	American Institute of Steel Construction
AISI	American Iron and Steel Institute
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
DDM (SDI DDM 03)	Steel Deck Institute Diaphragm Design Manual, Third Edition
IBC	International Building Code
ICC	International Code Council
SDI	Steel Deck Institute
SEAOC	Structural Engineers Association of California
SEI	Structural Engineering Institute

Glossary

The terms used to refer to describe elements of diaphragms and aspects of diaphragm design have not been used consistently in the past. To avoid confusion, these terms are presented below with the meaning that they are given in this Guide.

Chord	Boundary elements in the diaphragm that resist in-plane flexural forces. Chords may be distinct members (beams), or they may be designated regions of a composite deck with appropriate reinforcement.
Collector	A collector is a member or system of members that resists a horizontal force (axial force in the case of a beam), transferring it between the deck and the walls and frame. This element is considered to collect the force as a distributed shear from the deck and deliver it via axial force or shear to the walls and frames. Collectors may be distinct members (beams), or they may be designated regions of a composite deck with appropriate reinforcement.
Composite deck	Steel deck with concrete fill with bond such that the steel deck acts as reinforcement.
Concrete-filled deck	Steel deck with concrete fill without significant bond.
Deck	The element in the diaphragm systems that resists the in-plane shear necessary to deliver diaphragm forces to the collectors or walls and frames (steel deck or composite deck). Horizontal bracing is treated as equivalent to deck in this Guide.
Design forces	Forces derived from loads prescribed by ASCE 7 for the design of members and connections.
Diaphragm	The complete system necessary to deliver diaphragm forces to the walls and frames. This includes chords, collectors, deck, and distributors.
Diaphragm forces	The combination of inertial forces and transfer forces.

Diaphragm reaction	The force transferred between a diaphragm and a wall or frame.
Distributor	A distributor is a type of a collector. The difference is that in the conceptualization of the load path, the distributor takes force away from a wall or frame and delivers it as a distributed force to the deck.
Drag strut	The term "drag strut" has been used synonymously with "collector." In some circumstances, it has been used to denote members transferring lateral forces between discontinuous walls or frames. The term "collector" is preferred in this Guide.
Inertial forces	Diaphragm forces resulting from the acceleration of the mass tributary to the diaphragm.
Load path	Following the common convention for the conceptualization of seismic design, the load path is treated as beginning at the inertial mass of the building and ending with the delivery of the forces to the supporting soil.
Protected zone	Area of a steel member expected to be subject to significant inelastic strain demand during an earthquake and for which the consequence of failure is high. Plastic hinge regions of beams in moment frames and links in eccentrically braced frames are protected zones. See AISC 341 for more information.
Redistribution forces	A type of transfer force not due to the presence of a discontinuous wall or frame. Redistribution forces act on the diaphragm as a result of discontinuities in frame stiffness, interaction between frames, or other dynamic characteristics of the building.
Steel stud anchor	Element providing shear transfer between steel and concrete in a composite member (commonly referred to as a "steel stud").
Steel deck	Steel deck without concrete fill (referred to as "metal deck" in ASCE 7).
Torsion	In-plane rotation of the diaphragm due to lateral load.
Transfer forces	Diaphragm forces resulting from the acceleration of mass from levels above. These forces act on the diaphragm as a result of discontinuities in frames, frame stiffness, interaction between frames, or other dynamic characteristics of the building.
Vertical elements	Vertical elements of the lateral load-resisting system. These include shearwalls, braced frames, and moment frames. While beams and diagonal braces are not oriented vertically, they form part of a frame that is addressed by the term "vertical element."

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