# DESIGNING WITH STRUCTURAL

# STEEL

## A GUIDE FOR ARCHITECTS

### SECOND EDITION

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# PART I

# BASIC STRUCTURAL ENGINEERING

## UNDERSTANDING LOAD FLOW

All structures are subjected to forces that are imposed by gravity, wind and seismic events (see Figure 1). The combination and anticipated severity of these forces will determine the maximum design force the member can sustain. The structural engineer will then select a member that meets all of the strength as well as serviceability issues such as deflection and/or vibration criteria for any specific project. The following is a brief discussion on each of the types of loads and how these loads are transferred to the other structural components.

### Gravity Loads

Gravity loads include all forces that are acting in the vertical plane (see Figure 2). These types of forces are commonly broken down into dead loads and live loads in a uniform pounds per square foot loading nomenclature. Dead loads account for the anticipated weight of objects that are expected to remain in place permanently. Dead loads include roofing materials, mechanical equipment, ceilings, floor finishes, metal decking, floor slabs, structural materials, cladding, facades and parapets. Live loads are those loads that are anticipated to be mobile or transient in nature. Live loads include occupancy loading, office equipment and furnishings.

The support of gravity loads starts with beams and purlins. Purlins generally refer to the roof while beams generally refer to floor members. Beams and purlins support no other structural members directly. That is to say, these elements carry vertical loads that are uniform over an area and transfer the uniform loads into end reactions carried by girders.

Girders generally support other members, typically beams and/or purlins, and span column to column or are supported by other primary structural members. Girders may support a series of beams or purlins or they may support other girders. Forces imposed on girders from beams, purlins, or other girders are most often transferred to the structural columns. The structural column carries the vertical loads from all floors and roof areas above to the foundation elements.

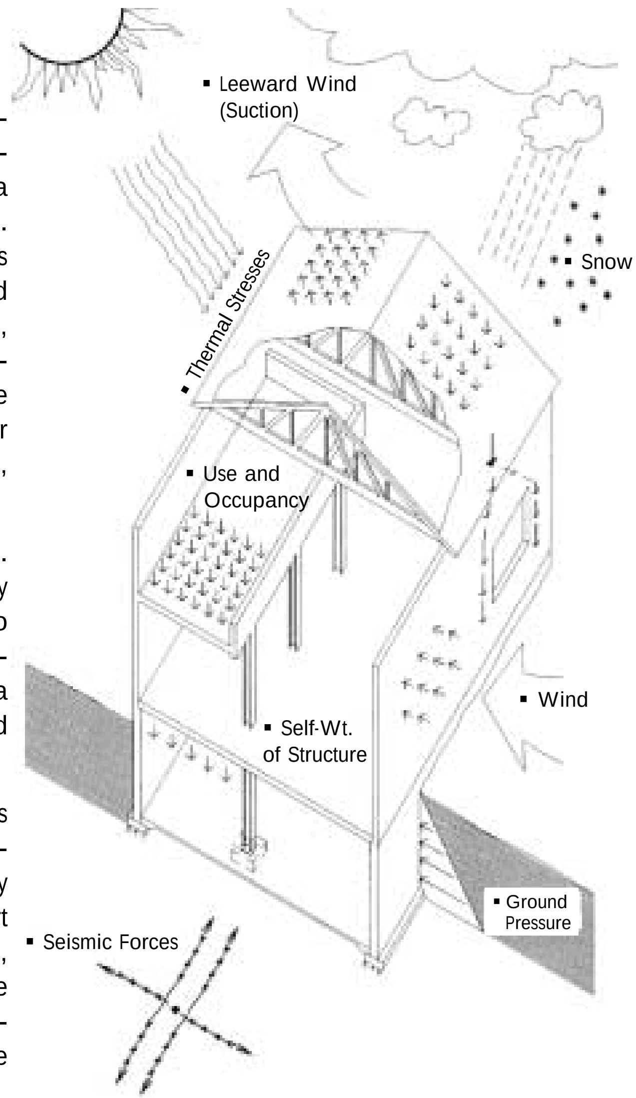


**Figure 1.** Forces experienced by structures

## Horizontal Loads

Forces created by wind or seismic activity are considered to act in the horizontal plane. While seismic activity is capable of including vertical forces, this discussion will be based only on horizontal forces. The majority of this section will address wind forces and how they are transferred to the primary structural systems of the building (see Figure 3).

Wind pressures act on the building's vertical surfaces and create varying forces across the surface of the façade. The exterior façade elements, as well as the primary lateral load resisting system, are subjected to the calculated wind pressures stipulated by code requirements. This variation accounts for façade elements being exposed to isolated concentrations of wind pressures that may be redistributed throughout the structural system. Design wind pressures can be calculated using a documented and statistical history of wind speeds and pressure in conjunction with the building type and shape. Calculated wind pressures act as a pushing force on the windward side of a building. On the leeward (trailing) side of the building, the wind pressures act as a pulling or suction force. As a result, the exterior façade of the entire building must be capable of resisting both inward and outward pressures.

Roof structures made up of very light material may be subjected to net upward or suction pressures from wind as well. Roofs typically constructed of metal decking, thin insulation and a membrane roof material without ballast have the potential to encounter net upward forces. Roof shape may also determine the net uplift pressures caused by wind. Curved roofs will actually exhibit a combination of downward pressures on the top portion of the curve and upward pressure on the lower portion of the curve. This distribution of downward and upward pressures caused by the curve is similar to the principles of air pressure and lift acting on an airplane wing.

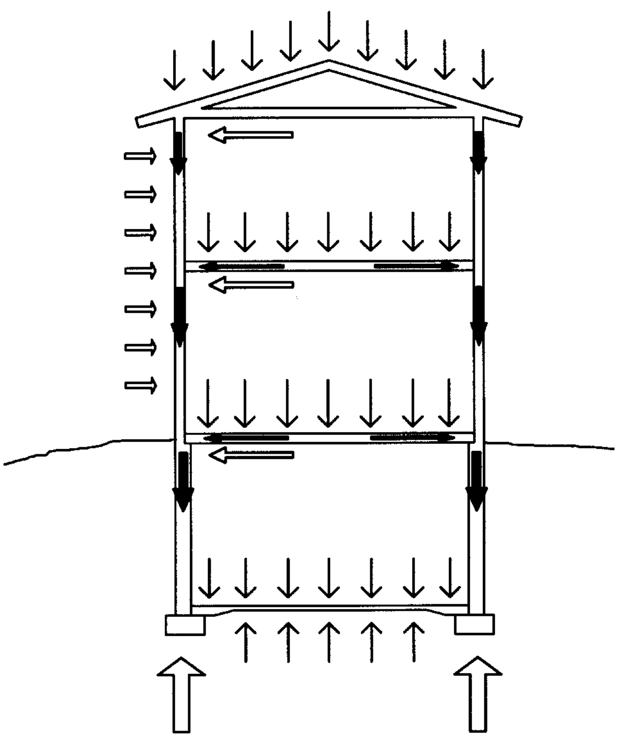

**Figure 2.** Gravity and wind loads

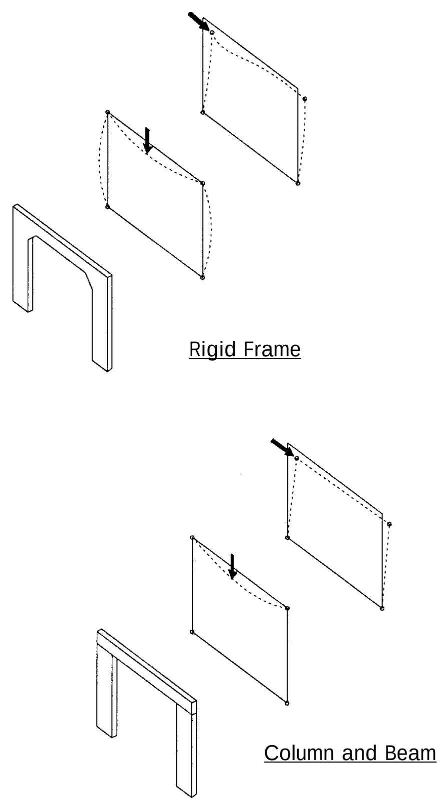


**Figure 3.** Loads on columns and beams

As the wind pressures are applied to the exterior of the building, the façade (actually a structural element to some degree), transfers the horizontal pressures to the adjacent floor or roof. As these pressures are transferred, the floor and roof systems must have a means to distribute the forces to the lateral load resisting systems. Floors and roofs that are generally solid or without large openings or discontinuities may behave as a diaphragm. A diaphragm is a structural element that acts as a single plane with the connecting beams and columns. When experiencing a force, this single plane causes the beams and columns to displace horizontally the same amount as the diaphragm. This can be exemplified by a sheet of paper or cardboard that is supported by a series of columns. Should the paper, a flexible diaphragm, be pushed horizontally, all points in contact with the paper will move laterally by the same amount. The metal roof decking on most projects will behave as a flexible diaphragm. Substituting a piece of cardboard for paper in our example, the paper will behave more like a rigid diaphragm. A typical floor decking and composite structural slab are examples of a rigid diaphragm.

Horizontal diaphragms are an efficient means to transfer the horizontal loads at each level of a building to the lateral load resisting systems (see Figure 4). Should large openings, such as atriums, skylights, raised floors or other discontinuities exist to interrupt the diaphragm, the lateral or horizontal loads may not flow easily to the lateral load resisting systems. As a result, the structural engineer will investigate the need for a horizontal truss system utilizing the floor beams and/or girders as chord members. Secondary web members will be added to complete the truss concept. This is particularly common in roof areas where there may be very long continuous skylights on a relatively narrow or long roof area.

### Seismic

Seismic activity induces horizontal forces, and at times, vertical loads. The discussions in this publication will focus on horizontal forces imposed during seismic activity. Forces created during a seismic event are directly related to weight or mass of the various levels on a specific building. During seismic activity horizontal diaphragms behave like wind load transfers with respect to the primary lateral load resisting systems. However, the induced forces are much more sensitive to the shape of the building and the positioning of the lateral load resisting systems. It is advantageous to consider a very regular building plan in areas of the country with significant seismic activity.

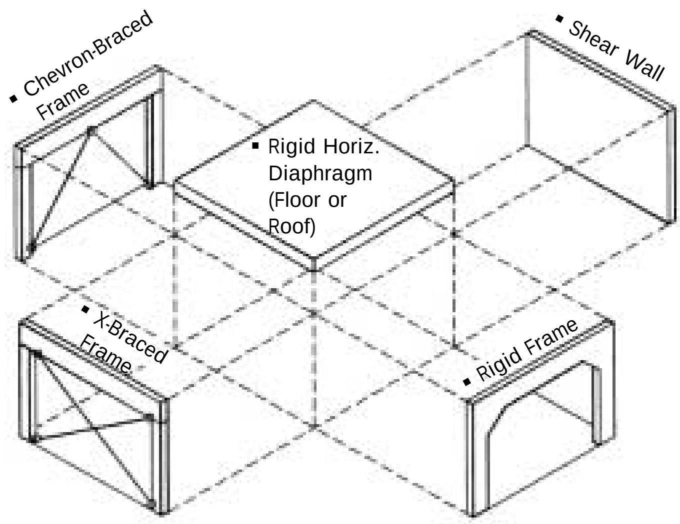


**Figure 4.** Horizontal diaphragm/lateral load resisting interface

## TYPES OF BASIC LATERAL SYSTEMS

During the initial planning stage of any project, consideration should be made for the type of lateral load resisting system(s) to be used in the building. Three basic types of lateral resisting systems are commonly used: braced frames, rigid frames, and shear walls. The structural engineer should be consulted early in the project to establish the type of system best suited for the specific building footprint, height and available locations. Careful consideration should be given to meet the lateral resistance requirements of the structure as well as the architectural needs of the building. In order to meet these needs the engineer may select one or more types of lateral systems. Each system has its own specific limitations and potential architectural implications.

## Braced Frames — General

Three types of braces used in braced frames typically seen in buildings today include the cross brace, Chevron (or inverted V) and eccentric brace. Cross bracing is often analyzed by the structural engineer as having tension-only members. Chevron bracing is used in a building that requires access through the bracing line. Eccentrically braced frames allow for doorways, arches, corridors and rooms and are commonly used in seismic regions to help dissipate the earthquake energy through the beam/girder between workpoints of the bracing/beam interface. Braced frames are generally more cost-effective when compared to rigid frame systems.

## Braced Frames — Cross Bracing

Perhaps the most common type of braced frame is the cross-braced frame. A typical representation of a cross-braced frame is shown in Figures 5 and 6. Figure 5 shows a typical floor framing plan with cross bracing denoted by the dashed-line drawn between the two center columns. The solid lines indicate the floor beams and girders. A typical multi-floor building elevation with cross-braced bays beginning at the foundation level is shown in Figure 6. While only one bay is indicated in Figure 6 as having cross bracing, it must be understood that many bays along a given column line may be necessary to resist the lateral loads imposed on a specific structure. One or more column lines having one or more bays of cross bracing may be necessary as well. It is important to establish early on in the development of any project the location of braced bays. These considerations are typical to all of the braced frames discussed in this publication.

Connections for this type of bracing are concentrated at the beam to column joints. Figure 7 illustrates a typical beam to column joint for a cross-braced frame. For taller buildings, usually over two or three stories, these connections could become large enough to minimize the available space directly adjacent to the column and below the beam. This restricted space may have an effect on the mechanical and plumbing distribution as well as any architectural soffit details. The structural engineer needs to be able to provide this type of information to the architect to avoid potentially costly field revisions during construction.

Bracing members are typically designed as tension only members. With this design approach only half of the members area active when the lateral loads area applied. The adjacent member within the same panel is considered to contribute no compressive strength. Utilizing tension only members makes very efficient use of the structural steel shape and will result in using the smallest members. Without full consideration of a specific bay size and amount and location of the bracing, a generalized range of sizes cannot be determined.

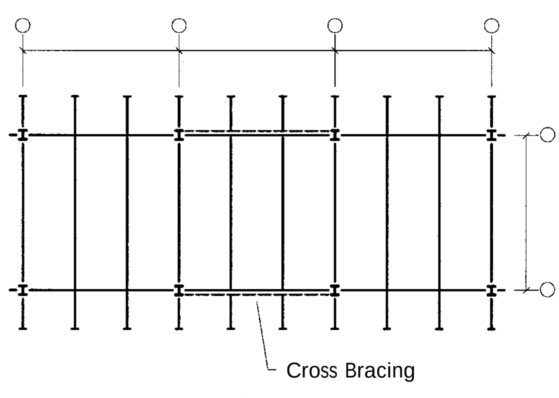


**Figure 5.** Typical floor plan with cross bracing

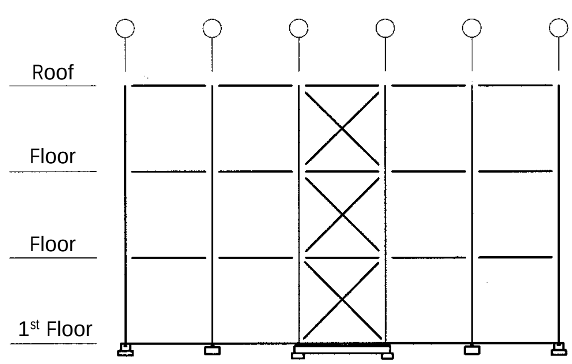


**Figure 6.** Cross-braced building elevation

Cross-braced frames are composed of single span, simply connected beams and girders. Columns that are not engaged by the braced frame can be designed as gravity load only column. Tables prepared for this publication in the Materials chapter may be used to select preliminary member sizes.

## Braced Frames — Chevron Bracing

Chevron bracing (inverted V bracing) is a modified form of a braced frame which allows for access ways to pass through a braced bay line. Figure 8 shows a typical floor framing plan with the bays using Chevron bracing denoted by the dashed-line drawn from between the two center columns. The solid lines indicate the floor beams and girders. Figure 9 shows a typical multi-floor building elevation using Chevron bracing. This system allows the architect to consider placing doorways and corridors through the bracing lines on a building.

There are two types of connections required for bracing elements. At the floor line the connection will be very similar to that required for cross-braced frames. This type of connection is illustrated in Figure 7. The connection at the floor above requires a gusset plate and field welded or bolted connection between the bracing members and the gusset plate. The depth of the gusset plate connection must be considered in the layout and coordination of mechanical ductwork and utility piping above the doorways and corridors.

As a consequence of the bracing configuration, the bracing members are subjected to gravity compressive loads. Each of the bracing members is considered active in the analysis of the system when lateral loads are applied. As a result, the bracing elements are subjected to both tension and compressive forces.

Beams and girders used in the Chevron-braced frame are designed as two span continuous members. This will almost always result in shallower and lighter members when compared to a simple span member of equal column-to-column length.

## Eccentrically Braced Frames

Eccentrically braced frames are very similar to frames with Chevron bracing. In both systems the general configuration is an inverted V shape with a connection between the brace and the column and a connection at the beam/girder at the next level up. However, unlike the Chevron-braced frame which has the brace member workpoints intersecting at the same point on the beam/girder for the brace elements. The condition is shown in Figure 10.

This type of bracing is commonly used in seismic regions requiring a significant amount of ductility or energy absorption characteristics within the structure. The beam/girder element between the workpoints of the bracing member shown is designed to link elements and assists the system in resisting lateral loads caused by seismic activity.

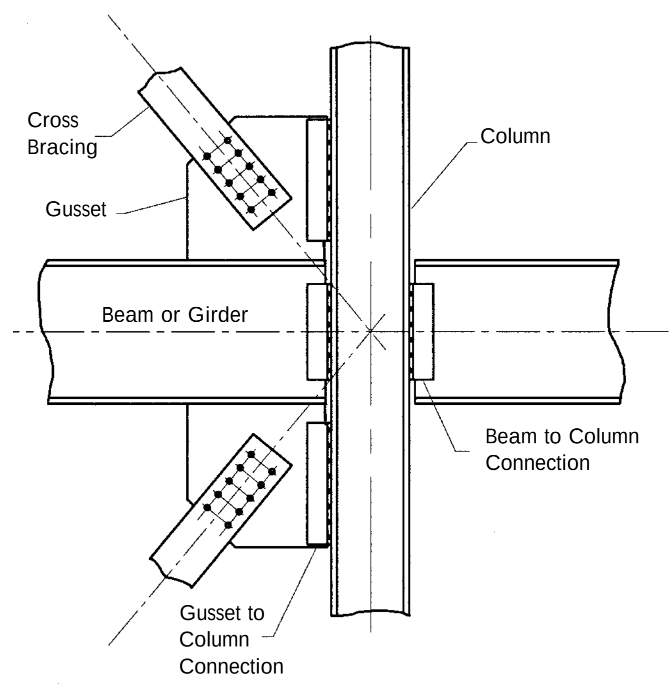


**Figure 7.** Typical beam to column brace connections

## Rigid Frames

Rigid frames are used when the architectural design will not allow a braced frame to be used. This type of lateral resisting system generally does not have the initial cost savings as a braced frame system but may be better suited for specific types of buildings.

Figures 11 and 12 show a floor plan and building line elevation of a rigid frame system. Figure 11 indicates the solid triangle designation typically used to show rigid connections between beam and column as well as girder and column. The building elevation shown in Figure 12 indicates the same solid triangular symbols at the floor line beam to column joints.

Connections between the beam/girder and column typically consist of a shear connection for the gravity loads on the member in combination with a field welded flange to column flange connection. Column stiffener plates may be required based on the forces transferred and column size. This type of joint is illustrated in Figure 13. It must be noted that this type of joint requires all vertical utility ductwork and piping to be free and clear of the column and beam/girder flanges. Coping of the beam/girder flanges to allow passage of piping or other utilities is usually not acceptable and must be brought to the attention of the structural engineer as soon as possible.

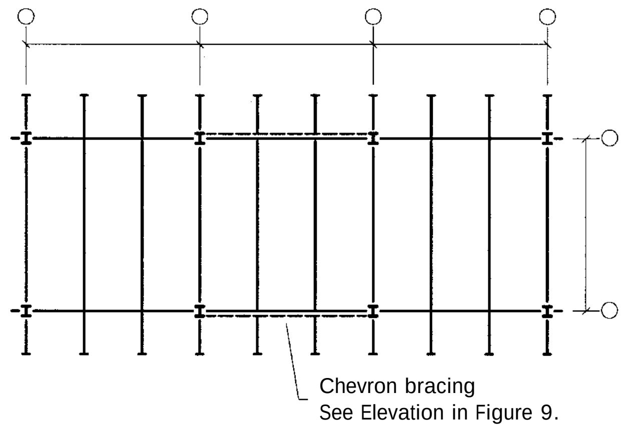


**Figure 8.** Typical floor plan with Chevron bracing

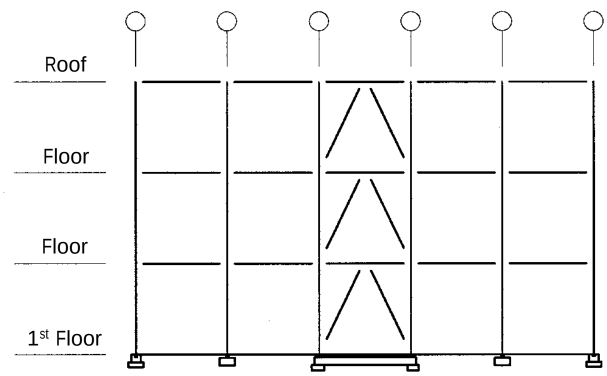


**Figure 9.** Elevation with Chevron bracing

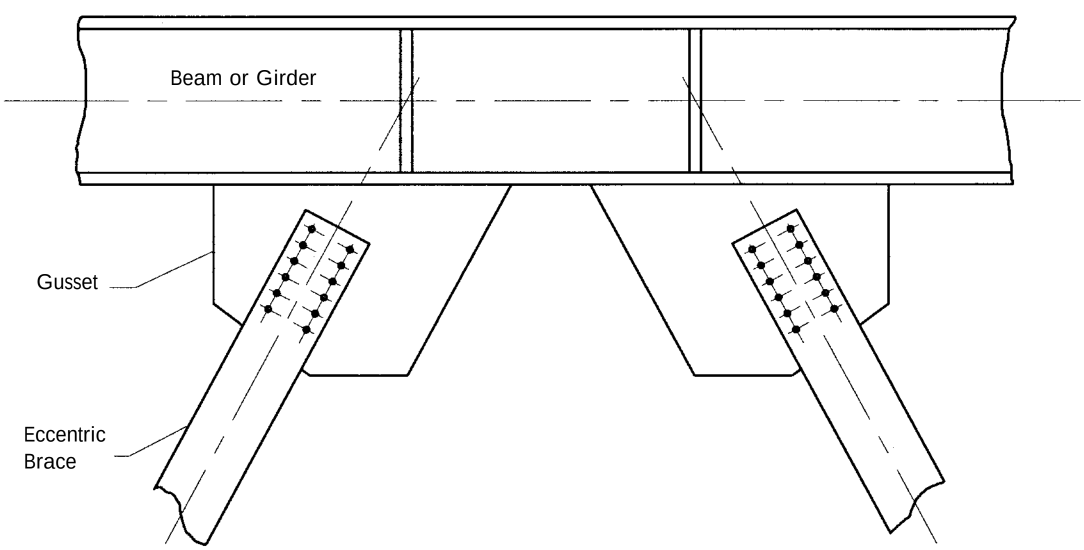


**Figure 10.** Eccentric brace with typical brace to beam connection

### Shear Walls

This type of lateral load resisting system engages a vertical element of the building, usually concrete or masonry, to transfer the horizontal forces to the ground by a primary shear behavior. Shear walls are usually longer than they are high and are inherently stiff elements. Careful attention to detailing the joint between the shear wall and floor or roof diaphragm elements may be required. Code-specific spacing of masonry shear walls may also impact the interior layout of the building.

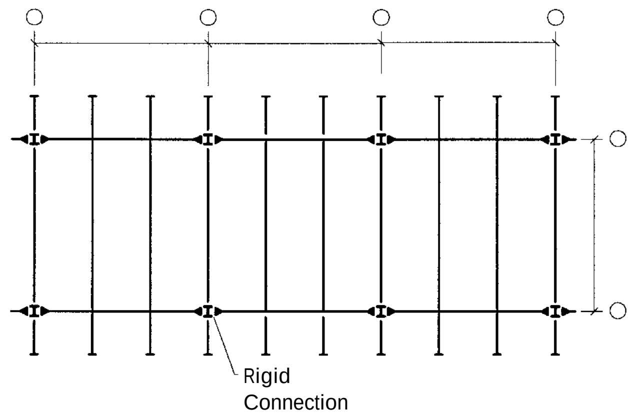


**Figure 11.** Typical floor plan with rigid frames

## BEAM WEB PENETRATIONS

Beam web penetrations are a way of allowing mechanical ductwork and plumbing lines to pass through structural beams and girders while maintaining a shallow ceiling sandwich and minimum floor-to-floor height. Beams and girders in buildings have, by natural consequence, regions of reserve capacity. The length of the member offers areas that can tolerate the placement of a round, square or rectangular penetration, either concentrically or eccentrically placed (see Figure 14). Concentrically placed penetrations have the centerline of the penetration matching the member depth centerline. Eccentric holes have their centerline either above or below the member depth centerline. Depending on the size, location and beam or girder, loading will determine whether the penetration should be reinforced or unreinforced. In some cases, beam

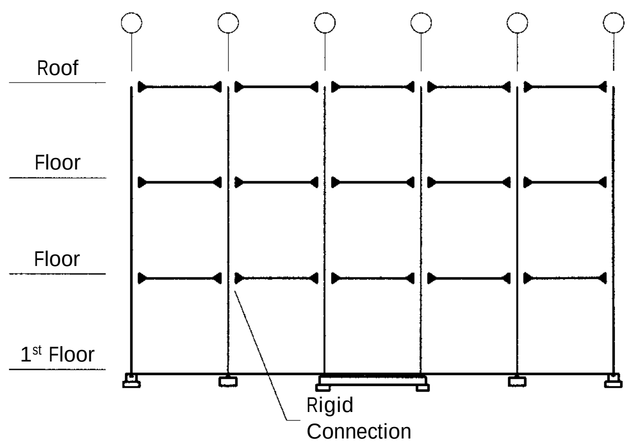


**Figure 12.** Rigid frame building elevation

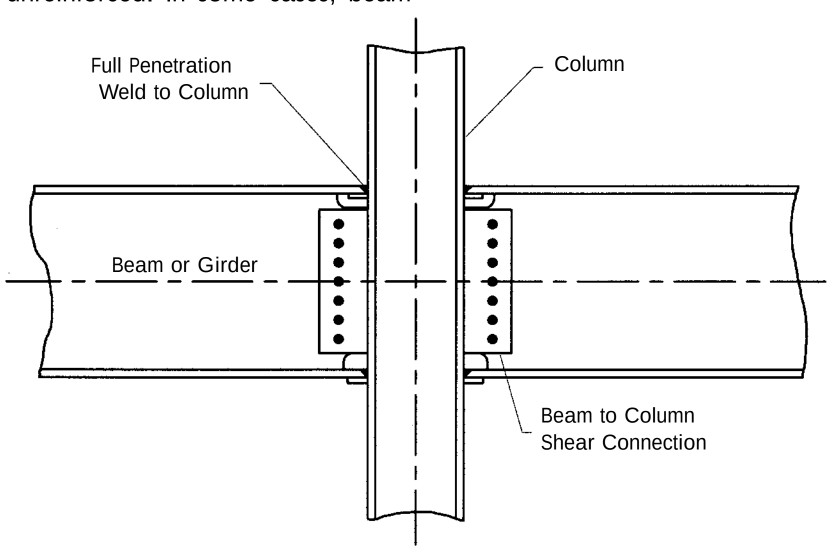


**Figure 13.** Typical rigid (moment) connection

and girder penetrations may not be structurally feasible. It is important to fully discuss the size and location of all intended web penetrations early in the project with a qualified structural engineer so that the structural design may proceed and costly field installed penetrations may be avoided.

Unreinforced web penetrations are holes cut in the web of the beam or girder with no other material added to strengthen the member, as the member carries the shear and moment forces in the beam satisfactorily. These type of penetrations are the least expensive to provide. Reinforced web penetrations are required in critical structural beams and girders that are heavily loaded and/or have very large penetrations that will compromise the integrity of the member. The material taken away by the penetration may be so significant that the member shears and moments cannot be accommodated by the remaining beam or girder material alone. As a result, reinforcing material must be added.

Hole reinforcing may consist of horizontal plates, a combination of horizontal and vertical plates or pipe sections for round penetration. This reinforcing is placed on one or both sides of the web. The specific structural member loading, member size, size of penetration and location of penetration will all play a role in determining the amount of reinforcing required.

As an aid to the architect in coordinating beam and girder web penetrations with the building ductwork and piping services, the following guidelines are suggested:

- Penetrations through members that have a depth-to-web thickness, $d/t_w$ > 75 should be avoided. Domestically available rolled shapes generally fall outside this criterion.
- The ratio of hole length to depth should be limited to 2.5.
- The hole depth must be limited to a maximum of 70 percent of the member depth.
- A minimum 15 percent of the member depth must remain from the edge of the hole to the outside face of the flange.
- Corners of penetrations must be made with a radius of approximately one inch. This must be considered in determining the size of penetration to accommodate ductwork and piping services.
- Concentrated loads from beams and column transfers must not be made within the length of the hole.
- Multiple holes should have a minimum two times the hole length between hole edges.
- Beams are to be laterally supported by the floor/roof construction.
- Penetrations in members that are at or near deflection limits or that have sensitive vibrations should be avoided.
- All penetrations must be investigated by a qualified structural engineer to insure the structural integrity of the member.

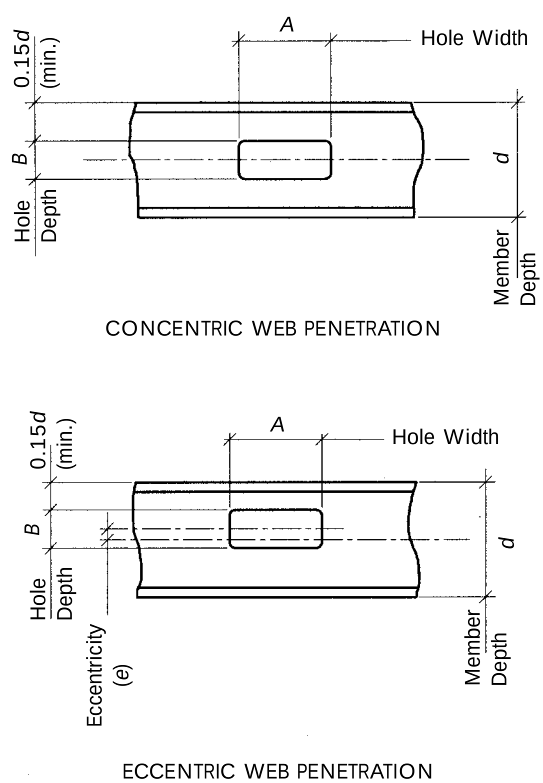


**Figure 14.** Concentric and eccentric web penetrations

## THERMAL MOVEMENT OF STRUCTURAL STEEL

One of the most difficult things to evaluate throughout the life of a building, and particularly during the construction period, is the amount of horizontal movement, expansion and contraction. It is difficult to design for movement since the designer cannot control some of the parameters. Expansion or contraction requirements for a structure under construction will be determined by the greatest change in temperature that the structure is exposed to prior to being enclosed and conditioned. Thermal movement is a concept that is not unique to exposed structural steel. In fact, it is not unique to steel as a building material. Movement applies to all building materials and must be accounted for in all types of construction. However, for these purposes discussion will be limited to movement of structural steel resulting from changes in temperature.

For example, it is reasonable for a steel building that is under construction in the Midwest to be erected in summer where the temperature of the steel exposed to the sun can exceed 100° Fahrenheit. The same building may not be enclosed by January, when the night temperatures can dip well below zero. The building would see a temperature change of more than 100° Fahrenheit from summer to winter.

The type of temperature differential might not appear to be significant. The integrity of the steel structure would not be affected by the thermal changes. However, the movement and stresses in the steel structure associated with a 100° change in temperature could be substantial.

The movement and changes in stress of steel are related to the steel's coefficient of linear expansion. The coefficient of linear expansion (or contraction) for any material is defined as the change in length (per unit of length) for a one degree change in temperature. The coefficient of linear expansion for steel is 0.0000065 for each degree Fahrenheit.

To determine how much a piece of steel will expand or contract throughout a change in temperature, the following equation is used:

$$\text{Change in steel length} = (0.0000065) \times (\text{Length of steel}) \times (\text{Temperature differential})$$

If a building with a large rectangular floor plan is exposed to a temperature differential of 60° Fahrenheit, and has expansion joints at every 200 ft in the long direction (see Figure 15), the horizontal movement in that direction will be as follows:

$$\begin{aligned}\text{Change in steel length} &= (0.0000065) \times (200 \text{ ft}) \times (60° \text{ Fahrenheit})\\ &= 0.08 \text{ ft}\\ &= 0.94 \text{ in.}\end{aligned}$$

It should be noted that this is the total horizontal expansion or contraction that would be expected within that temperature range. If the building were constructed during the coldest temperature of the 60° temperature range, each 200-ft segment between expansion joints would expand approximately 0.94 in. Conversely, if the building were constructed during the warmest temperature season, each 200-ft segment between building expansion joints would contract by approximately 0.94 in.

Realistically, each expansion joint in this example should be at least one-inch wide if not more. Remember, building construction tolerances must be considered, and a one-inch joint may not be sufficient. The separate sides of the expansion joint should never come in contact with each other even when the building has fully expanded. It should also be noted that the floor, wall, and ceiling finish materials that are selected to cover the expansion joints should be able to accommodate the one inch movement. This would also be true of any mechanical, electrical or plumbing components that span across the expansion joints.

The previous example is a simplified explanation of building movement. There are, however, other factors that contribute to the "real world" thermal movement of buildings. One of those factors is the fixity of the column bases. If the column bases are "fixed", the thermal movements will be less than with "pinned" base connections. The stress in the members, however, would increase substantially. Other factors, such as whether or not the building is heated and cooled in its designed environment will have an impact on the building's movement.

An excellent reference on the topic of thermal expansion and contraction is the Federal Construction Council's Technical Report No. 65, *Expansion Joints in Buildings*. A structural engineer should be consulted before determining expansion joint locations, sizes and spacings.

Once expansion joint locations and sizes have been determined, accommodations must be made for the movement. Basically, there are two ways to accommodate movement. One way is to provide support members such as columns on both sides of the expansion joint as shown in Figure 16. In essence, the structure on each side of the expansion joint is treated as a separate structure, free to move independently of the other side. The other approach is to make provisions for movement by allowing some of the structure to slide relative to the other while still supported on a common support. This is typically accomplished by creating a seated slide-bearing detail that is supported directly on either a column or a beam as shown in Figure 17. This alternate type of expansion joint is generally used when double columns cannot be accommodated, or where double columns in an exposed position of the building would be undesirable.

Regardless of what type expansion/contraction system is used, it cannot be overemphasized that freedom of movement must be incorporated throughout all of the building systems. Again provisions must be made for all components that cross the expansion joint.

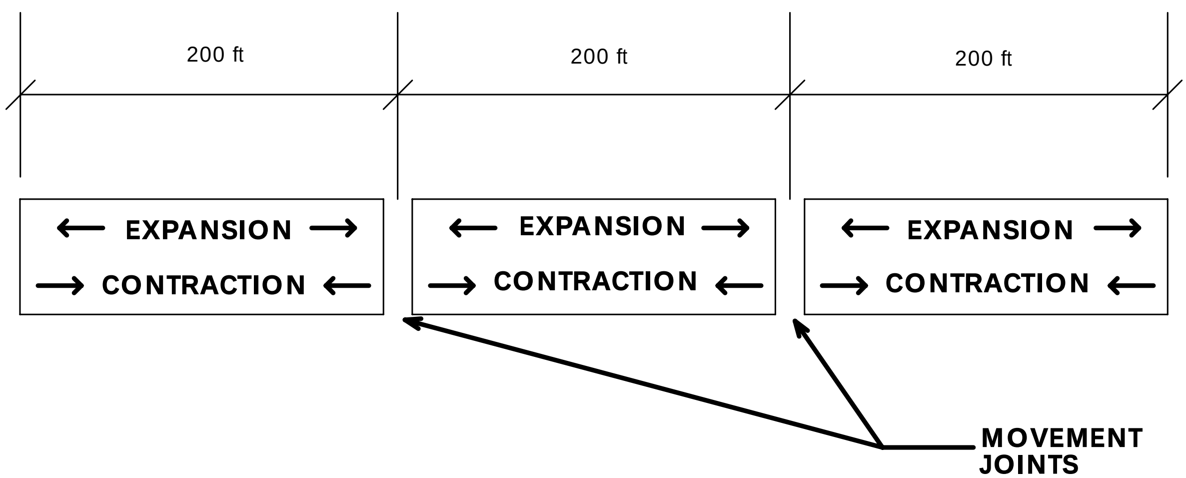


**Figure 15.** Diagram of building expansion example

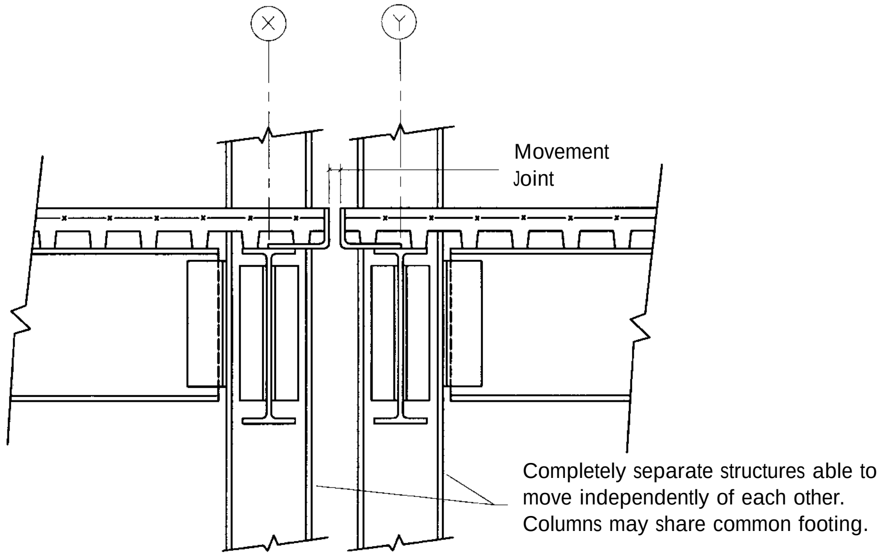


**Figure 16.** Double-column movement connection

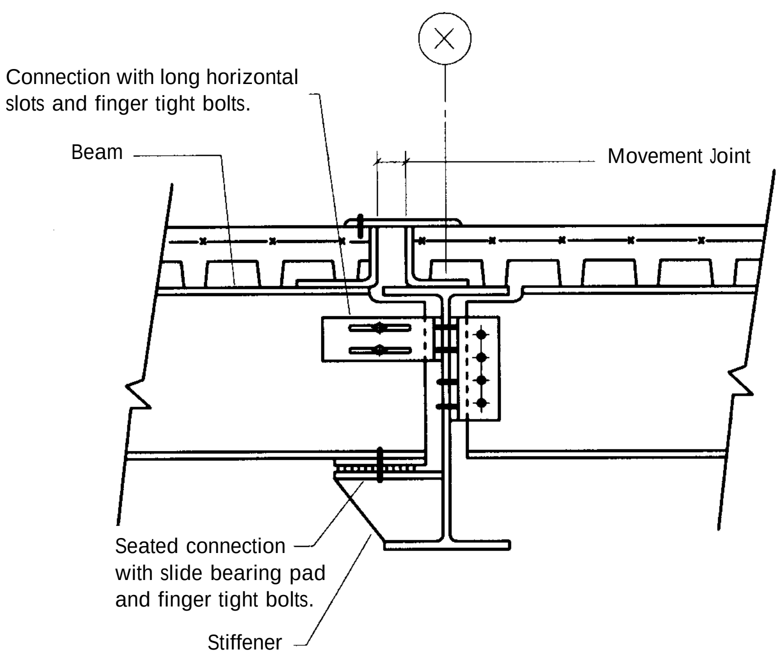


**Figure 17.** Seated slide-bearing connection

## FLOOR VIBRATION

Movement of floors caused by occupant activities can present a serious serviceability problem if not properly considered and prevented by the design of the structural system. Humans are very sensitive vibration sensors - vertical floor movement of as little as forty thousandths of an inch can be very annoying. Post-construction repairs of floors that vibrate are always very expensive, and sometimes cannot be done because of occupancy limitations. This reinforces the necessity of addressing potential vibration problems in the original design.

The response of individuals to floor motion depends on the environment, occupant age, and location. People are more sensitive in quiet environments, such as a residence or quiet office, as compared to a busy shopping mall. The elderly are more sensitive than young adults, and sensitivity appears to increase when sitting as compared to standing or reclining.

Stiffness and resonance are dominant considerations in the vibration serviceability design of steel floor structures and footbridges. The first known stiffness criterion appeared nearly 170 years ago. In 1828, an English carpenter named Tregold published a book on carpentry writing that girders over long spans should be "made deep to avoid the inconvenience of not being able to move on the floor without shaking everything in the room." The traditional stiffness criterion for steel floors limits the live load deflection of beams or girders supporting plastered ceilings to span/360. This limitation, along with restricting span-to-depth ratios of members to 24 or less, have been widely applied to steel-framed floor systems in an attempt to control vibrations, but with limited success.

Traditionally, soldiers "break step" when marching across bridges to avoid large, potentially dangerous, resonant vibrations. Until recently, resonance had been ignored in the design of floors and footbridges. Approximately 30 years ago problems arose with the vibrations induced by walking on steel-joist supported floors that had satisfied traditional stiffness criteria. Since that time much has been learned about the loading function due to walking and the potential for resonance. More recently, new rhythmic activities, such as aerobics and high impact dancing, have caused serious floor vibrations due to resonance.

A number of analytical procedures have been developed which allow a structural designer to assess the floor structure for occupant comfort for a specific activity and for suitability for sensitive equipment. Generally, the analytical tools require the calculation of the first natural frequency of the floor system and the maximum amplitude of acceleration, velocity, or displacement for a reference activity or excitation. An estimate of the damping in the floor is also generally required. A human comfort scale or sensitive equipment criterion is then used to determine whether the floor system meets serviceability requirements. Some of the analytical tools incorporate limits on acceleration into a single design formula whose parameters are estimated by the designer.

Before presenting a technical explanation of floor design principles, basic terminology is listed and explained. A review of this terminology will greatly assist in the understanding of the structural design principles that follow.

### Basic Vibration Terminology

***Dynamic Loadings***. Dynamic loadings can be classified as harmonic, periodic, transient and impulsive as shown in Figure 18. Harmonic or sinusoidal loads are usually associated with rotating machinery. Periodic loads are caused by rhythmic human activities such as dancing and aerobics, and by impactive equipment. Transient loads occur from movement of people and include walking and running. Single jumps and heel-drop impacts are examples of impulsive loads.

***Period and Frequency.*** Period is the time, usually in seconds, between successive peak excursions in repeating events. Period is associated with harmonic (or sinusoidal) and repetitive time functions as shown in Figures 18a and 18b. Frequency is the reciprocal of period and is usually expressed in Hz (Hertz or cycles per second).

***Steady State and Transient Motion.*** If a structural system is subjected to a continuous harmonic driving force (see Figure 18a), the resulting motion will have a constant frequency and constant maximum amplitude and is referred to as steady state motion. If a real structural system is subjected to a single impulse, damping in the system will cause the motion to subside as illustrated in Figure 19. This is one type of transient motion.

***Natural Frequency and Free Vibration.*** Natural frequency is the frequency at which a body or structure will vibrate when displaced and then quickly released. This state of vibration is referred to as free vibration. All structures have a large number of natural frequencies; the lowest or "fundamental" natural frequency is of most concern.

***Damping and Critical Damping.*** Damping refers to the loss of mechanical energy in a vibrating system. Damping is usually expressed as the percent of critical damping or as the ratio of actual damping to critical damping. Critical damping is the smallest amount of viscous damping for which a free vibrating system that is displaced from equilibrium and released comes to rest without oscillation.

***Resonance.*** If a frequency component of an exciting force is equal to a natural frequency of the structure, resonance will occur. At resonance, the amplitude of the motion can become very large as shown in Figure 20.

***Step Frequency.*** Step frequency is the frequency of application of a foot or feet to the floor, e.g., walking, dancing or aerobics.

***Harmonic.*** A harmonic multiple is an integer multiple of the frequency of application of a repetitive force (e.g., multiple of step frequency for human activities or multiple of rotational frequency of reciprocating machinery). Harmonics can also refer to natural frequencies, e.g., of strings or pipes.

***Mode Shape.*** When a floor structure vibrates freely in a particular mode, it moves up and down with a certain configuration or mode shape. Each natural frequency has a mode shape associated with it. Figure 21 shows typical mode shapes for a simple beam and for a slab/beam/girder floor system.

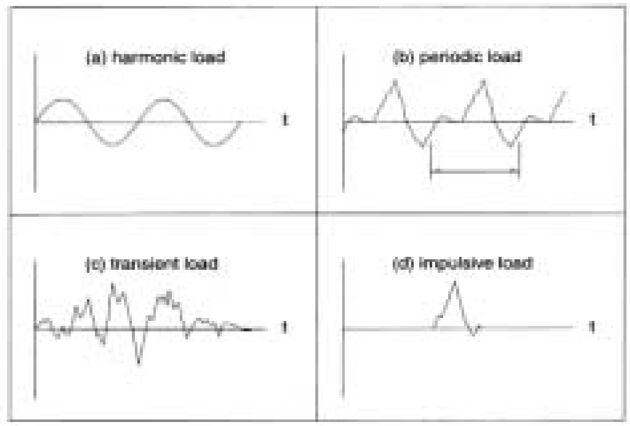


**Figure 18.** Types of dynamic loading

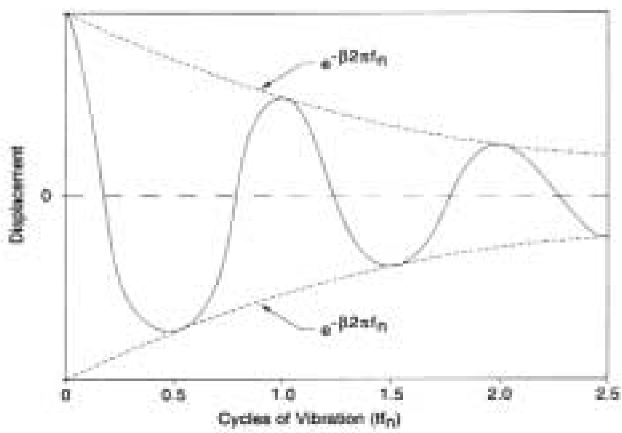


**Figure 19.** Decaying vibration with viscous damping

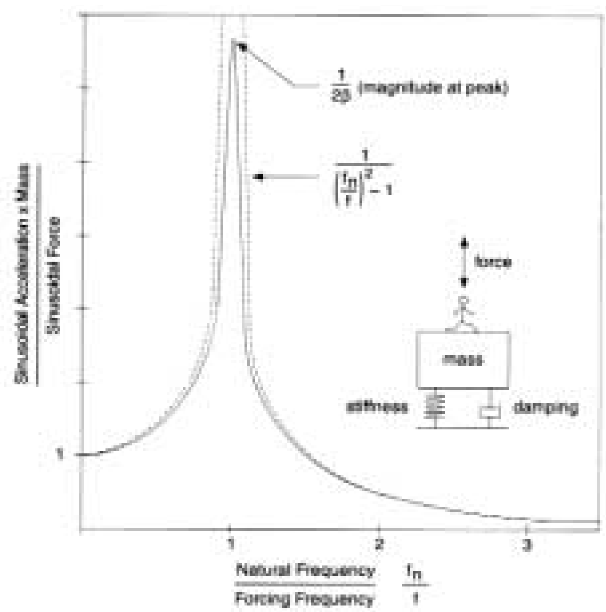


**Figure 20.** Response to sinusoidal force

***Modal Analysis.*** Modal analysis refers to a computational analytical or experimental method for determining the natural frequencies and mode shapes of structures, as well as the responses of individual modes to a given excitation.

***Spectrum.*** A spectrum shows the variation of relative amplitude with frequency of the vibration components that contribute to the load or motion. Figure 22 is an example of a frequency spectrum.

***Acceleration Ratio.*** The acceleration of a system divided by the acceleration of gravity is referred to as the acceleration ratio. Usually the peak acceleration of the system is used.

**Floor Panel.** A rectangular plan portion of a floor encompassed by the span and an effective width is defined as the floor panel.

**Bay.** A rectangular plan portion of a floor defined by four column locations.

## Floor Vibration Principles

Although human annoyance criteria for vibration have been known for many years, it has only recently become practical to apply such criteria to the design of floor structures. The reason for this is that the problem is complex, the loading complex, and the response complicated - involving a large number of modes of vibration. Experience and research have shown, however, that the problem can be simplified sufficiently to provide practical design criteria.

Most floor vibration problems involve repeated forces caused by machinery or by human activities such as dancing, aerobics or walking, although walking is a little more complicated than the others because the forces change location with each step. In some cases, the applied force is sinusoidal or nearly so.

AISC's Steel Design Guide No. 11: *Floor Vibrations Due to Human Activities* explains in detail the required engineering calculations and assessment techniques. These techniques use acceleration, as a percent of acceleration due to gravity, to measure human perception of floor movement. For example, the tolerance level for quiet environments, residences, offices, churches, etc. is 0.5 percent of gravity (0.005*g*).

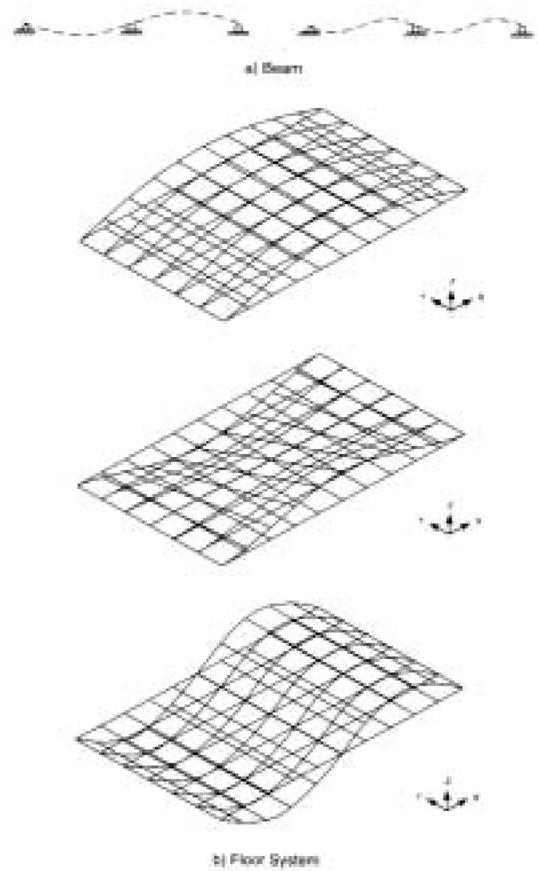


**Figure 21.** Typical beam and floor system mode shapes

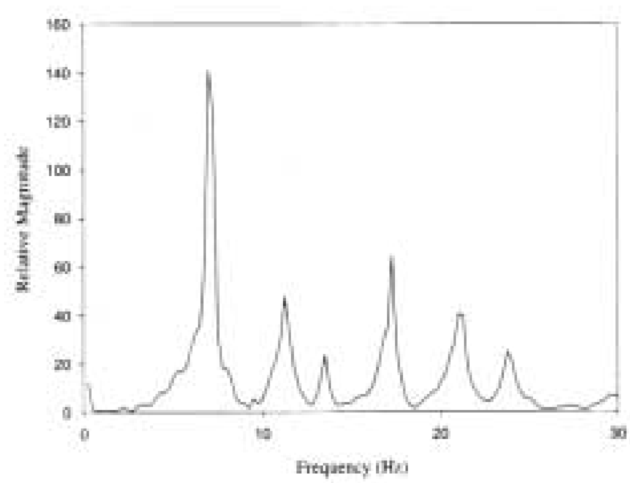


**Figure 22.** Frequency spectrum

Figure 23 shows tolerance levels for a number of situations. Note that the scale is a function of frequency and acceleration. Also, note that the tolerance acceleration level increases as the environment becomes less quiet. For instance, the tolerance level for people participating in aerobics (rhythmic activities) is ten times greater than if they are in a quiet office. To use the scale, the natural floor frequency and the estimated acceleration for an activity must be calculated.

The acceleration of a floor system depends on the activity, the natural frequency for the floor, the amount of mass that moves when the floor vibrates, and the damping in the floor. Floor acceleration increases as energy in the activity increases; thus, floor acceleration is greater for aerobics than for walking. Acceleration decreases with increasing weight; the acceleration for a lightweight concrete floor will be greater than that for the same normal weight concrete floor for the same activities. Acceleration decreases with increasing damping.

Evaluation of a floor system for potential annoying vibration requires careful estimation of the weight supported by the floor on a typical day. A fully loaded floor will never be a problem; most occupant complaints are received when the problem floor is slightly loaded. The design dead load for mechanical equipment and ceiling should never be used, nor should the design live load. An estimate of the real mechanical loading (for instance, 2 psf not 5 psf as may be used for strength design) and ceiling is required. Recommended live loads in the Floor Vibrations design guide are 11 psf for office live loading (not 50 psf as used for strength design), 6 psf for residences, and 0 psf for shopping malls.

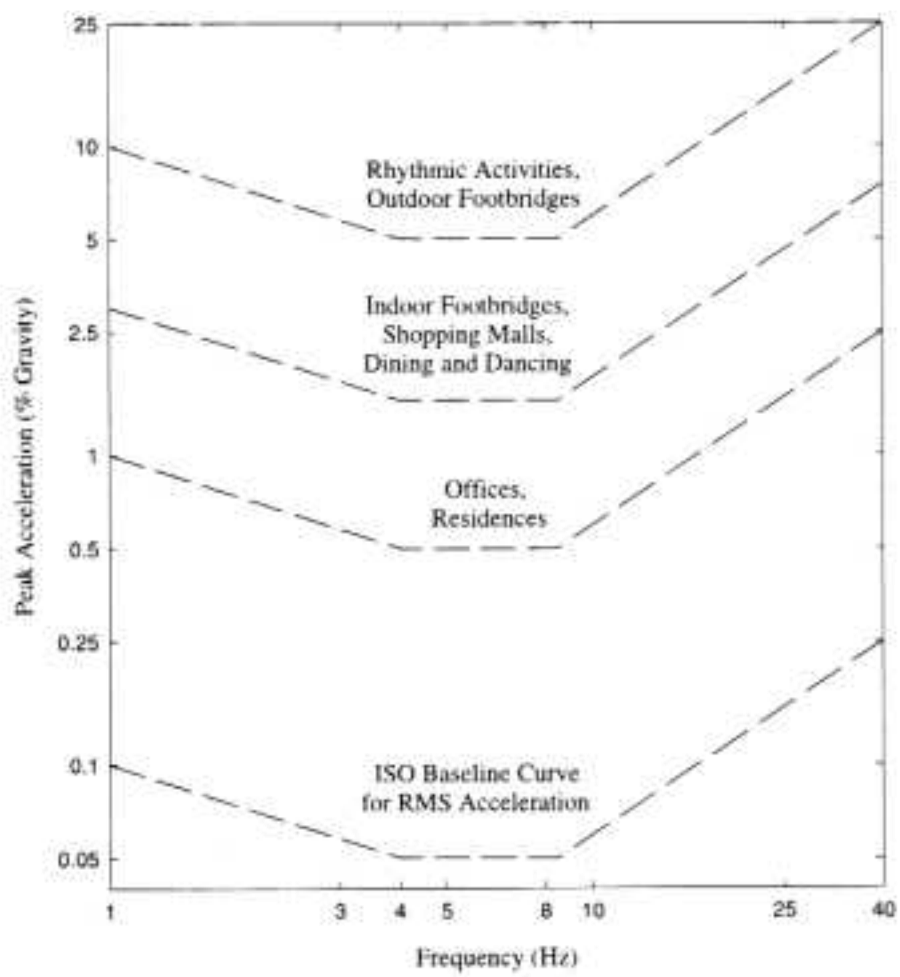


**Figure 23.** Recommended peak acceleration for human comfort for vibrations due to human activities (International Standards Organization [ISO], 2631-2: 1989)

Frequency is the rate at which a floor vibrates and is expressed in cycles per second (Hz). Floor systems generally have a frequency between 3 and 20 Hz. For a typical steel framed 30 ft by 30 ft office building bay, the frequency will be in the 5-8 Hz range. Frequency is a function of span (the longer the span, the lower the frequency) and weight supported (the heavier the floor and the supported contents, the lower the frequency). Thus, a floor constructed using normal weight concrete will vibrate at a lower frequency than the same floor constructed with lightweight concrete. When the frequency is above 15 Hz, as occurs in very short spans (say less than 15 ft), floor vibrations are generally not felt.

Damping is energy loss due to relative movement of floor components or fixtures on the floor. Damping causes a freely vibrating floor system to come to rest and is usually expresses as a percent of critical damping. Critical damping is the amount of damping required to bring a vibrating system to rest in one-half cycle. Damping for floors is usually between 2 percent and 5 percent. The lower value is for floors supporting few non-structural components, like for open work areas and churches. The larger value is for floors supporting full-height partitions. A typical office floor with movable, half-height partitions has about 3 percent damping.

Particular attention should be given to office floors with open spaces, no fixed partitions, and light loads. This situation is what results in problem floors if the design is not done correctly. Also, floors with high design loads (say 125 psf) and light actual loads (say less than 15 psf) do not have the same amount of damping as floors designed for normal office loading (say 50 psf). In this case, a lower estimate of damping should be used (e.g., 1-2 percent).

The design of floors supporting rhythmic activities, dancing, aerobics, etc. require consideration of the entire structure, not just the supporting floors. These activities introduce very high energy levels into the structure and can cause annoying floor motion quite some distance from the activity area. Aerobics on the 60th floor of a building have caused excessive floor motion twenty floors below. When a rhythmic activity floor is located above approximately six stories, column deflections must be considered.

To avoid annoying vibrations in floors supporting rhythmic activities, the fundamental natural frequency must be above frequencies associated with harmonics of the activity and the tolerance acceleration ratio. The tolerance acceleration ratio is a function of both the rhythmic activity and the affected occupancy. For instance, when dancing and dining are considered, the tolerance acceleration ratio is $0.02g$. The tolerance level is increased to $0.05g$ for participants in lively concerts or sports events.

To satisfy the criterion, a relatively large fundamental natural frequency is required. For example, if jumping exercises are shared with weightlifting with an acceleration tolerance level of $0.02g$ and floor weight of 50 psf, the required frequency is 10.6 Hz. The economical solution for this example is lightweight concrete and deep, lightweight supporting members.

Floors supporting sensitive equipment, such as operating room equipment, electron microscopes, and microelectronics manufacturing equipment must be very stiff and heavy. Tolerance levels for this type of equipment are usually expressed in velocity with numbers like 100 to 8,000 micro-in./second. The means of accommodating sensitive equipment are readily available, but usually require specialists in this area to produce a satisfactory design.

## Summary

The determination of potentially annoying floor motion for a proposed design requires careful consideration of the structural system, the anticipated activities, and the finished space. Art, as well as science, is required on the part of the designer. The most important parameter to be determined is the fundamental natural frequency of the floor structure. This calculation requires a careful estimate of the supported weight on an average day. Floor system damping, which depends on the components of the building systems, as well as occupancy furnishings and partitions, also must be estimated. Finally, an acceleration tolerance criterion must be selected and compared to the predicted acceleration of the floor structure.