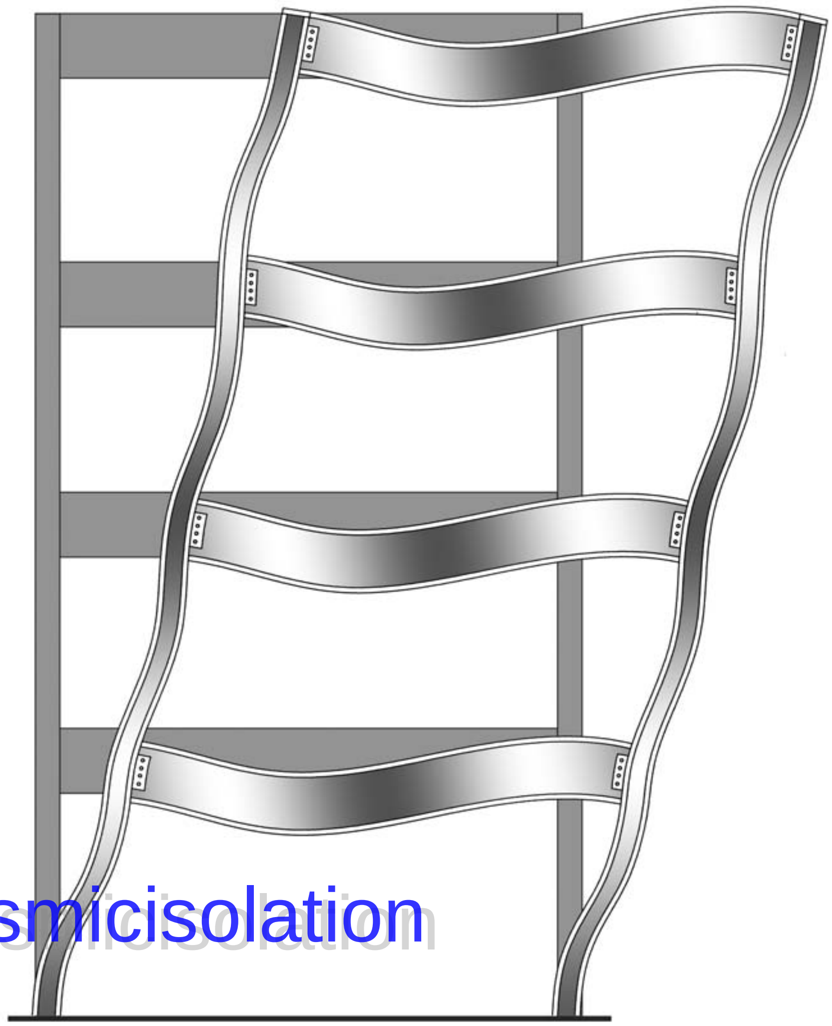
Steel Design Guide 3

Serviceability Design Considerations for Steel Buildings

Second Edition



Steel Design Guide

3

Serviceability Design Considerations

for Steel Buildings

MICHAEL WEST AND JAMES FISHER
Computerized Structural Design, Inc.
Milwaukee, Wisconsin

with contributions from

LAWRENCE G. GRIFFIS
Walter P. Moore and Associates
Austin, Texas

Second Edition

AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC.

The information presented in this publication has been prepared in accordance with recognized engineering principles and is for general information only. While it is believed to be accurate, this information should not be used or relied upon for any specific application without competent professional examination and verification of its accuracy, suitability, and applicability by a licensed professional engineer, designer, or architect. The publication of the material contained herein is not intended as a representation or warranty on the part of the American Institute of Steel Construction or of any other person named herein, that this information is suitable for any general or particular use or of freedom from infringement of any patent or patents. Anyone making use of this information assumes all liability arising from such use.

Caution must be exercised when relying upon other specifications and codes developed by other bodies and incorporated by reference herein since such material may be modified or amended from time to time subsequent to the printing of this edition. The Institute bears no responsibility for such material other than to refer to it and incorporate it by reference at the time of the initial publication of this edition.

Printed in the United States of America

First Printing: March 2004

Preface

This Design Guide is the second edition of AISC *Design Guide 3*, which was originally titled *Serviceability Design Considerations for Low-Rise Buildings*. The new title *Serviceability Design Considerations for Steel Buildings* reflects the addition of information on tall buildings and the following more general information:

1. A review of steel building types, occupancies and serviceability design considerations related to each, as applicable.

2. Revision to current editions of references.

3. Information on ponding for roof design.

4. Information on floors, including discussion regarding cambering beams and how deflection issues relate to the construction of concrete slabs.

5. Revision of floor vibration information to follow AISC *Design Guide 11, Floor Vibrations Due to Human Activity* (Murray and others, 1997).

Acknowledgements

The authors wish to thank the Metal Building Manufacturers Association for its joint support with AISC in the preparation of the first edition of this Guide.

AISC would also like to thank the following people for assistance in the review of this Design Guide. Their comments and suggestions have been invaluable.

Todd Alwood
Harry A. Cole
Charles J. Carter
Cynthia J. Duncan
Tom Ferrell
Louis F. Geschwindner
John L. Harris
Christopher M. Hewitt
Lawrence Kloiber
Jay W. Larson
Roberto Leon
William Liddy
Ronald L. Meng
Charles R. Page
Davis Parsons
David T. Ricker
Victor Shneur
William T. Segui
Eldon Tipping

Table of Contents

Chapter 1
Introduction

Serviceability is defined in the AISC Specification as "a state in which the function of a building, its appearance, maintainability, durability, and comfort of its occupants are preserved under normal usage". Although serviceability issues have always been a design consideration, changes in codes and materials have added importance to these matters.

The shift to a limit-states basis for design is one example. Since 1986, both the AISC *LRFD* and AISC *ASD Specifications* have been based upon the limit-states design approach in which two categories of limit states are recognized: strength limit states and serviceability limit states. Strength limit states control the safety of the structure and must be met. Serviceability limit states define the functional performance of the structure and should be met.

The distinction between the two categories centers on the consequences of exceeding the limit state. The consequences of exceeding a strength limit may be buckling, instability, yielding, fracture, etc. These consequences are the direct response of the structure or element to load. In general, serviceability issues are different in that they involve the response of people and objects to the behavior of the structure under load. For example, the occupants may feel uncomfortable if there are unacceptable deformations, drifts, or vibrations.

Whether or not a structure or element has passed a limit state is a matter of judgment. In the case of strength limits, the judgment is technical and the rules are established by building codes and design specifications. In the case of serviceability limits, the judgments are frequently non-technical. They involve the perceptions and expectations of building owners and occupants. Serviceability limits have, in general, not been codified, in part because the appropriate or desirable limits often vary from application to application. As such, they are more a part of the contractual agreements with the owner than life-safety related. Thus, it is proper that they remain a matter of contractual agreement and not specified in the building codes.

In a perfect world the distinction between strength and serviceability would disappear. There would be no problems or failures of any kind. In the real world all design methods are based upon a finite, but very small probability of exceedance. Because of the non-catastrophic consequences of exceeding a serviceability limit state, a higher probability of exceedance is allowed by current practice than for strength limit states.

The foregoing is not intended to say that serviceability concerns are unimportant. In fact, the opposite is true. By having few codified standards, the designer is left to resolve these issues in consultation with the owner to determine the appropriate or desired requirements.

Serviceability problems cost more money to correct than would be spent preventing the problem in the design phase. Perhaps serviceability discussions with the owner should address the trade-off between the initial cost of the potential level of design vs. the potential mitigation costs associated with a more relaxed design. Such a comparison is only possible because serviceability events are by definition not safety related. The Metal Building Manufactures Association (MBMA) in its *Common Industry Practices* (MBMA, 2002) states that the customer or his or her agent must identify for the metal building engineer any and all criteria so that the metal building can be designed to be "suitable for its specific conditions of use and compatible with other materials used in the Metal Building System." Nevertheless, it also points out the requirement for the active involvement of the customer in the design stage of a structure and the need for informed discussion of standards and levels of building performance. Likewise the AISC *Code of Standard Practice* (AISC, 2000) states that in those instances where the fabricator has both design and fabrication responsibility, the owner must provide the "performance criteria for the structural steel frame."

Numerous serviceability design criteria exist, but they are spread diversely through codes, journal articles, technical committee reports, manufacturers' literature, office standards and the preferences of individual engineers. This Design Guide gathers these criteria for use in establishing serviceability design criteria for a project.

Serviceability Requirements in the AISC *Specification*

The *LRFD Specification* (AISC, 1999) lists five topics that relate to serviceability concerns. They are:

1. camber

2. expansion and contraction

3. deflections, vibrations, and drift

4. connection slip

5. corrosion

Camber

Camber may or may not be a solution to a serviceability issue, and the authors have attempted to identify appropriate and inappropriate use of camber in this Design Guide. In most instances, the amount of total movement is of concern rather than the relative movement from the specified floor elevation, in which case camber is not an appropriate solution. There are, however, situations where camber is appropriate, such as in places where it is possible to sight down the under side of exposed framing.

Expansion and Contraction

Expansion and contraction is discussed to a limited extent. The goal of this Design Guide is to discuss those aspects of primary and secondary steel framing behavior as they impact non-structural building components. For many types of low-rise commercial and light industrial projects, expansion and contraction in the limited context given above are rarely an issue. This does not mean that the topic of expansion and contraction is unimportant and, of course, the opposite is true. For large and/or tall structures, careful consideration is required to accommodate absolute and relative expansion and contraction of the framing and the non-structural components.

Connection Slip

Connection slip has not been addressed explicitly in this Design Guide. However, it is the authors' intent that the various drift and deflection limits include the movements due to connection slip. Where connection slip, or especially the effect of accumulated connection slip in addition to flexural and/or axial deformations, will produce movements in excess of the recommended guidelines, slip-critical joints should be considered. Slip-critical joints are also required in specific instances enumerated in Section 5 of the *Specification for Structural Joints Using ASTM A325 or ASTM A490 Bolts* (RCSC, 2000). It should be noted that joints made with snug-tightened or pretensioned bolts in standard holes will not generally result in serviceability problems for individual members or low-rise frames. Careful consideration should be given to other situations.

Corrosion

Corrosion, if left unattended, can lead to impairment of structural capacity. Corrosion is also a serviceability concern as it relates to the performance of non-structural elements and must be addressed by proper detailing and maintenance. The primary concerns are the control or elimination of staining of architectural surfaces and prevention of rust formation, especially inside assemblies where it can induce stresses due to the expansive nature of the oxidation process. Again, the solutions are proper detailing and maintenance.

Serviceability Requirements in ASCE 7

ASCE 7-02, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2002) addresses serviceability in paragraph 1.3.2 Serviceability as follows:

> "Structural systems, and members thereof, shall be designed to have adequate stiffness to limit deflections, lateral drift, vibration, or any other deformations that adversely affect the intended use and performance of buildings and other structures."

ASCE 7-02 provides an appendix with commentary entitled Serviceability Considerations. While this appendix is non-mandatory, it does draw attention to the need to consider five topic areas related to serviceability in the design of structures:

- deflection, vibration, and drift
- design for long-term deflection
- camber
- expansion and contraction
- durability

The ASCE 7 appendix introduction notes that "serviceability shall be checked using appropriate loads for the limit state being considered." The commentary to the Appendix provides some suggestions with regard to loads and load combinations. For example, two load combinations are suggested for vertical deflections of framing members:

$$D + L$$

$$D + 0.5S$$

These are recommended for limit states "involving visually objectionable deformations, repairable cracking or other damage to interior finishes, and other short term effects." For serviceability limit states "involving creep, settlement, or other similar long-term or permanent effects," the suggested load combination is:

$$D + 0.5L$$

With regard to lateral drift, the commentary cites the common interstory drift limits of L/600 to L/400. The commentary also notes that an absolute interstory drift limit of 3/8 in. (10 mm) may often be appropriate to prevent damage to non-structural elements. This absolute limit may be relaxed if there is appropriate detailing in the non-structural elements to accommodate greater drift. The commentary provides the following load combination for checking short-term effects:

$$D + 0.5L + 0.7W$$

The reader is encouraged to refer to the appendix commentary, which provides additional insights into the issue of serviceability and an extensive list of references.

This Guide will address the following serviceability design criteria:

1. roofing
2. skylights
3. cladding
4. interior partitions and ceilings
5. vibrations
6. equipment

Most of these criteria limit relative and absolute deflection and, in the case of vibrations, place limits on the range of response and controls for the physical characteristics of structures and elements. Additionally, the presentation and discussion of a consistent loading and analysis approach is essential to these criteria. Without these three elements (load, analysis approach, and serviceability limit) a serviceability design criterion is useless.

This Design Guide provides serviceability design criteria are for selected applications. Source material has been documented wherever possible. Many of the design criteria are based upon the authors' own judgment and rules of thumb from their own experience. It should be noted that when applicable building codes mandate specific deflection limits the code requirements supersede the recommendations of this Design Guide.

Structures framed in structural steel accommodate numerous occupancies and building types. The following discussion addresses ten occupancy types and the specific serviceability design considerations associated with these occupancies.

Storage/Warehouses

Most modern storage facilities, unlike those of previous eras, are single story buildings. As such, modern storage occupancies usually enclose large unobstructed areas under a roof. The significant serviceability design considerations are:

- roof slope and drainage
- ponding stability
- roof deflection
- wall support and girt deflection
- frame drift
- expansion joints

Manufacturing

Like Storage/Warehouse facilities, modern manufacturing facilities are large single story structures, which may include extensive mezzanines. The most significant serviceability design considerations for this occupancy type are:

- roof slope and drainage
- ponding stability
- roof deflection
- wall support and girt deflection
- frame drift
- expansion joints
- vibration in mezzanine areas
- suspended equipment
- crane operation
- corrosion
- equipment vibration

In addition to the serviceability considerations provided in this Guide, the reader is referred to AISC *Design Guide 7, Industrial Buildings: Roofs to Column Anchorage* (AISC, 2004) for a useful discussion on manufacturing facilities.

Heavy Industrial/Mill Buildings

Heavy industrial and mill construction has many of the same serviceability considerations as Manufacturing. Additionally, care must be taken to ensure the proper operation and performance of the cranes. AISC *Design Guide 7, Industrial Buildings: Roofs to Column Anchorage* (AISC, 2004) is worthwhile reading on this subject. The significant serviceability design considerations are:

- crane operation
- roof slope and drainage
- ponding stability
- roof deflection
- wall support and girt deflection
- frame drift
- expansion joints

Mercantile/Shopping Malls

Mercantile structures are frequently large one and two story structures sharing some of the same serviceability design considerations as Storage/Warehouse occupancies. With large areas of roof drainage, roof deflections and expansion joints require special attention. As AISC *Design Guide 11, Floor Vibrations Due to Human Activity* (AISC, 1997) points out, objectionable vibrations have been observed in the second floor levels of these types of structures. Objectionable floor vibrations can result from a lack of damping in open pedestrian areas and walkways. This is discussed in detail in *Design Guide 11*. The significant serviceability design considerations for mercantile occupancies are:

- roof slope and drainage
- ponding stability
- roof deflection
- frame drift
- expansion joints
- floor vibration
- skylights
- corrosion in winter garden and large fountain areas

Health Care and Laboratory Facilities

Although hospitals and clinics are generally multi-story structures, they can be constructed as single-story facilities. The performance of the floor structures is of significant concern, and special attention should be given to the effect of floor vibration on sensitive laboratory equipment. The relationship between the frame and the curtain wall is another important design consideration, as is the performance and operation of traction elevators. The significant serviceability design considerations for health care occupancies are:

- roof slope and drainage
- ponding stability
- roof deflection
- curtain wall/spandrel deflection
- frame drift
- expansion joints
- floor deflection
- vibration of floors
- concreting of floors
- suspended equipment
- elevator operation
- skylights

Educational

Schools and other academic buildings are constructed as both single and multi-story structures. Typical serviceability considerations for floors, roofs and walls apply to all such structures. Structures in schools with swimming pools must be protected against a potentially corrosive environment. Schools with physical education facilities on upper levels must consider the impact of floor vibrations on the structure, especially those due to rhythmic excitation. Lenzen (1966), cites the case of a school in which floor vibrations were not perceptible when the teacher and students were present, but vibration was deemed to be annoying when the classroom was empty except for teachers working after classes. The significant serviceability design considerations for educational occupancies are:

- roof slope and drainage
- ponding stability
- roof deflection
- curtain wall/spandrel deflection
- frame drift
- expansion joints
- floor deflection
- vibration of floors
- concreting of floors
- skylights
- corrosion

Office Buildings

Office buildings are constructed in all heights from single-story buildings to high-rise towers. The relationship of the building frame to the curtain wall is important, as are frame drift and floor deflection. Floor vibration can be an issue.

Elevator operation is also a significant concern. The major serviceability considerations for office occupancies are:

- roof slope and drainage
- ponding stability
- roof deflection
- curtain wall/spandrel deflection
- frame drift
- perception of wind-induced acceleration
- expansion joints
- floor deflection
- vibration of floors
- concreting of floors
- suspended equipment
- elevator operation
- skylights

Parking Structures

Structural steel-framed parking structures are frequently open structures, which exposes the framing. Protection of the structural steel and connections from corrosion and good drainage are significant concerns. More detailed information on the design of steel-framed open-deck parking structures is available in AISC *Design Guide 18, Open-Deck, Steel-Framed Parking Structures* (Churches, and others 2003). The significant serviceability design considerations for parking structures are:

- deck slope and drainage
- expansion joints
- concreting of floors
- corrosion

Residential/Apartments/Hotels

Residential occupancies that are steel framed are commonly mid- to high-rise structures. Frequently the taller of these structures are mixed use buildings with portions of the space devoted to office and retail occupancies. Most, if not all, of the serviceability design considerations for office occupancies apply to residential occupancies. These are:

- roof slope and drainage
- ponding stability
- roof deflection
- curtain wall/spandrel deflection
- frame drift
- perception of wind-induced acceleration
- expansion joints
- floor deflection
- vibration of floors
- concreting of floors
- suspended equipment
- elevator operation
- skylights
- corrosion in chlorine-disinfected swimming pools

Assembly/Arenas

Assembly occupancies are not discussed extensively in this Design Guide. These buildings are by nature unique, one-of-a-kind structures with large open spans. The accommodation of large deflections and the associated cambers and thermal movements are critical aspects of the design. Additionally, the potential for rhythmic excitation of the structure by the crowd must be considered.

Seismic Applications

It should be noted that this Design Guide does not provide guidance on serviceability limit states exceeded due to the deformations and interstory drifts of a structural frame subjected to seismic loading. Such requirements are explicitly included in the building code and the reader is referred there.

Chapter 2
Design Considerations Relative to Roofing

Roof serviceability largely relates to the structure's role in maintaining the integrity of the roofing membrane and the drainage system. Although ponding relates to both the strength and stiffness of the roof structure, ponding stability is ultimately a strength design consideration; see AISC *Specification* Section K2 (LRFD, 1999; ASD 1989). Because of the importance of ponding stability as a design issue, and because ponding instability is a function of load and deflection, the following discussion of the topic is included in this design guide.

Ponding Stability

The AISC *Specification* provides that unless a roof surface is provided with sufficient slope towards points of free drainage, or adequate individual drains to prevent the accumulation of rain water, the roof system must be investigated to ensure adequate strength and stability under ponding conditions. The ponding investigation must be performed by the specifying engineer or architect. ASCE 7-02 establishes adequate slope to drain as ¼-in. per ft in Section 8.4. Additional information is provided in the Steel Joist Institute *Technical Digest No 3*, *Structural Design of Steel Joist Roofs to Resist Ponding* (SJI, 1971).

Ponding as a structural design phenomenon is of concern for two reasons:

1. The loading is water, which can fill and conform to a deflected roof surface.

2. The source of load (water) is uncontrollable, i.e. rain is a natural hazard.

When water can accumulate on a structural system due to impoundment or restriction in drainage, ponding must be checked. Reasons for the accumulation can be:

1. Dead load deflections of members in roofs designed to be flat.

2. Deflections of members, which places points in their spans below their end points.

3. Deflections of bays supporting mechanical units.

4. Members installed with inverted cambers.

5. Blocked roof drains.

6. Parapets without scuppers.

7. Parapets with blocked scuppers.

8. Intentional impoundment of water as part of a controlled-flow roof drain design.

9. Low-slope roofs, which allow water to accumulate due to the hydraulic gradient.

Ponding rainwater causes the deflection of a roof system, which in turn increases the volumetric capacity of the roof. Additional water is retained which in turn causes additional deflection and volumetric capacity in an iterative process. The purpose of a ponding check is to ensure that convergence occurs, i.e. that an equilibrium state is reached for the incremental loading and the incremental deflection. Also, stress at equilibrium must not be excessive.

The AISC *Specification* in Section K2 gives limits on framing stiffness that provide a stable roof system. They are:

$$C_p + 0.9C_s \leq 0.25 \qquad \text{(Eq. K2-1)}$$

$$I_d \geq 25(S^4)10^{-6} \qquad \text{(Eq. K2-2)}$$

where,

$C_p = (32L_s L_p^4) / (10^7 I_p)$
$C_s = (32SL_s^4) / (10^7 I_s)$
L_p = length of primary members, ft
L_s = length of secondary members, ft
S = spacing of secondary members, ft
I_p = moment of inertia of primary members, in.^{4}
I_s = moment of inertia of secondary members, in.^{4}
I_d = moment of inertia of the steel deck, in.^{4} per ft

Equation K2-2 is met in most buildings without the need for increased deck stiffness. Equation K2-1, in many cases, requires stiffer elements than would be required by loading. In the majority of cases, roofs that do not meet equation K2-2 can be shown to conform to the bending stress limit of $0.80F_y$ in the *ASD Specification* or F_y in the *LRFD Specification*. The relationship between the requirements of the two specifications is discussed in "Ponding Calculations in LRFD and ASD" (Carter and Zuo, 1999).

Appendix K of the *LRFD Specification* and the Commentary to the *ASD Specification* provide a procedure to meet the total bending stress requirement. It should be noted that the checking of bending stresses is not required if the stiffness controls of equations K2-1 and K2-2 are met. This procedure is based on:

1. A calculation of the deflection due to the accumulation of water in the deflected shape of the primary and secondary members at the initiation of ponding. These

deflected shapes are taken to be half sine waves, which is sufficiently accurate for this calculation.

2. In LRFD a load factor of 1.2 is used for dead and rain load per Appendix K with an implied value of $\phi = 1.0$ (see Carter/Zuo, 1999). In ASD a factor of safety of 1.25 for stresses due to ponding is used, which results in an allowable stress of $0.8F_y$.

3. Behavior of the members is in the elastic range so that deflection is directly proportional to stress.

4. Stress due to ponding is limited to F_y (LRFD) or $0.80F_y$ (ASD) minus the factored stress or stress in the members at the initiation of ponding, depending on the specification applied.

Thus, the method uses four variables:

U_p, the stress index for the primary member.

U_s, the stress index for the secondary member.

C_p, the stiffness index for the primary member.

C_s, the stiffness index for the secondary member.

C_p and C_s are as given in the *Specification* in Section K2.

U_p and U_s are given as:

$$(F_y - f_o) / f_o$$

$$(0.8F_y - f_o) / f_o$$

where f_o is the bending stress in the member (primary or secondary) at the initiation of ponding. In LRFD f_o is calculated using the factored load of $1.2D + 1.2R$, with D = the nominal dead load and R = the nominal rain/snow load.

Both the *LRFD Specification* Appendix and the *ASD Specification* Commentary present two figures K2.1 and K2.2. Figure K2.1 is used to find a maximum C_p when U_p and C_s are given. Figure K2.2 is used to find a maximum C_s when U_s and C_p are given. This procedure is thus a checking procedure since trial sections must be chosen to establish C_p, C_s, U_p, and U_s. Figures K2.1 and K2.2 are graphs representing combinations of stress and stiffness that control the increment of load (stress) and deflection at the initiation of ponding.

If one studies the relationships in these figures, it can be noted that the required stiffness is inversely related to initial stress. If the stress index associated with values of C_p and C_s that meet the stiffness limit of $C_p + 0.9C_s \leq 0.25$ is plotted, one can see that the stress index is very low, indicating that f_o is very near $0.9F_y$ (LRFD) or $0.6F_y$ (ASD). This is logical since the system is so rigid that the ponded accumulation is negligible. As one moves beyond the values of C_p and C_s that meet Equation K-2.1, it can be seen that the term $(F_y - f_o)$ (LRFD) or $(0.8F_y - f_o)$ (ASD) must increase to provide for the reduction in stiffness, e.g. the increase in C_p and/or C_s. Thus it can be seen that the accurate calculation of fo is the essential element in using this procedure.

The LRFD Appendix and the ASD Commentary states that f_o is "the computed bending stress in the member due to the supported loading, neglecting the ponded effect." The calculations for the increment of ponded water are a function of the initial deflection and stiffness of the primary and secondary members. The initial deflection and the initial stress are the result of the "initial loads," which are those present at the "initiation of ponding." This means that the "initial loads" may be and will probably be different from the design loads. The initial loads include all appropriate dead and collateral loads, such as:

1. Weight of structural system.

2. Weight of roofing and insulation system.

3. Weight of interior finishes.

4. Weight of mechanical and electrical systems.

5. Weight of roof top mechanical systems.

The initial loads also include some or all of the superimposed load. The requirements of the AISC *Specification* and Commentary point to the fact that the superimposed load must actually be present at the initiation of ponding. Thus the appropriate portion of design superimposed load is not necessarily 100 percent of the design superimposed load. The amount of superimposed load used is to a degree up to the judgment of the engineer.

The most significant loading in northern regions of the country is a prediction of the amount of snow present at the initiation of ponding. A significant factor in all regions is a judgment of the amount of water on the roof at the initiation of ponding. Also, consideration must be given to the combination of snow and water, where applicable. The AISC *Specification* (LRFD Appendix and ASD Commentary) demonstrate that the loading at the initiation of ponding does not include the water that produces the stresses due to ponding, but does include water trapped on the roof because the roof has not been "provided with sufficient slope towards points of free drainage or adequate individual drains to prevent the accumulation of rain water." Also, as noted above, ASCE 7-02 Section 8.3 states that roofs with a slope of at least ¼ in. per ft need not be investigated for ponding stability. However, the superimposed load at the initiation of ponding could include water trapped by plugged internal roof drains.

ASCE 7-02 Section 8.3 requires that "each portion of a roof shall be designed to sustain the load of all rainwater that will accumulate on it if the primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow." Previous model codes included similar requirements.

The use of the weight of trapped or impounded water is recommended in SJI *Technical Digest No. 3*, *Structural Design of Steel Joist Roofs to Resist Ponding Loads*. This reference also gives an approach for accounting for the potential for snow and water in combination. It recommends that "where ice and snow are the principal source of roof live load" 50 percent of the design live load be used up to 30 psf live load, and 100 percent of the design live load when the design live load is 40 psf and greater." Presumably the percentage could be interpreted as varying linearly for loads between 30 and 40 psf. When these values are used to account for rain and snow, it is not necessary to add in the weight of potential trapped water described above unless the weight of impounded water would be greater than the reduced design live load. Model building codes require that roofs with a slope of less than ½ in 12 be designed for rain on snow in accordance with ASCE 7-02 Section 1608.3.4. ASCE 7-02 requires a rain on snow load where p_g is 20 lb/ft^2 or less but not zero.

ASCE 7-02 requires that roofs with "controlled drainage" must be checked for ponding instability, as determined in the provisions for "ponding instability." When these provisions apply, they require that "The larger of snow load or rain load shall be used in this analysis. The primary drainage system within an area subjected to ponding shall be considered to be blocked in this analysis."

Note that the earlier discussion described two-way roof framing systems. There is a separate case where the secondary framing bears directly on walls. This case eliminates the primary member deflection and the AISC *Specification* (LRFD Appendix and ASD Commentary) procedures can be used by reference to Figures K2.1 and K2.2 for which C_s is calculated using the deck properties and C_p is calculated using the joist properties. Also the SJI *Technical Digest No. 3* gives a procedure for accounting for a reduction in the accumulated water weight due to camber. Logic suggests that concept could also be applied to the two-way system.

Neither AISC nor SJI procedures address the deflected geometry of a continuous primary framing system. All of the deflection and load calculations of both procedures are based on the half-sine wave shape of the deflected element. This shape is conservative with a continuous primary member, because it overestimates the volume in the deflected compound curve.

Thus,

1. Ponding stability is an important concern in roof design.
2. Using the stiffness criteria of the *Specification* can produce unnecessarily conservative designs.
3. Use of the design approach presented in the AISC Commentary is recommended.
4. Determination of the appropriate loading in the calculation of initial stress is absolutely critical for the method to produce an accurate result.

Roofing

The concerns for the integrity of the roofing lie in three main areas:

1. in the field of the roof
2. at the edges
3. at penetrations

Two types of roofing will be discussed here: membrane roofs and metal roofs on structure.

Membrane Roofs

The field of a membrane roof must be isolated from the differential thermal movement of membrane and structure. This is done by means of "area dividers" in the roof membrane. The spacing of these joints depends on the type of roofing and climate conditions. The *Roofing and Waterproofing Manual*, Fifth Edition, published by the National Roofing Contractors Association (NRCA, 2001) concedes that recent experience with newer materials indicates that area dividers can be spaced at greater intervals for certain types of membrane systems than had previously been the case. In fact the *Manual* uses the phrase "may not be required at all" in its presentation on the need for area dividers and their spacing requirements for certain membrane systems.

Area dividers are commonly required for attached or adhered systems and are generally spaced at intervals of 150-200 ft. Area dividers will, in all likelihood, be spaced at intervals smaller than the building expansion joints.

The integrity of the roofing field is affected by the underlying structure. Factory Mutual System in its *Approval Guide* gives maximum spans for various deck types and gages. The Steel Deck Institute provides different criteria:

1. A maximum deflection of span divided by 240 for uniform design live load; and,
2. a limit of span divided by 240 with a 200-lb concentrated load at midspan on a 1-ft 0-in. wide section of deck.

SDI also gives maximum recommended spans for decks subjected to maintenance and construction loads. These are repeated in the NRCA *Manual*.

Table 1. Deflection Limits, adapted from IBC Table 1604.4

CONSTRUCTION	LIVE	SNOW OR WIND	DEAD + LIVE
Roof members:			
Supporting plaster ceiling	l / 360	l / 360	l / 240
Supporting nonplaster ceiling	l / 240	l / 240	l / 180
Not supporting ceiling	l / 180	l / 180	l / 120
Roof members supporting metal roofing:	l / 150	–	l / 60
Floor Members	l / 360	–	l / 240
Exterior walls and interior partitions:			
With brittle finishes	–	l / 240	–
With flexible finishes	–	l / 120	–
Secondary wall members supporting metal siding	–	l / 90	–

Both of these standards recognize that the localized and differential deflections induced by concentrated loads are in general more important to the proper performance of the roof than the uniform load capacity. The Commentary to the *ASD Specification* recommends a minimum depth for roof purlins of "(F_y/1000) times the span, except in the case of flat roofs." The Steel Joist Institute limits the maximum live load deflection for roof joists and girders to span divided by 240 (para. 5.9, 104.10, and 1004.6). The National Roofing Contractors Association (NRCA) *Manual*, Fifth Edition, recommends a limit on the deflection of the roof deck of span divided by 240 for total load.

As mentioned in the section herein on cladding, the joint between wall and roof is a critical point. The roofing edge detail must be able to accommodate any relative vertical and/or horizontal movement between wall and roof to prevent rupture. This condition is of less concern where ballasted loose-laid membranes are used, but is a very significant problem where conventional built-up roofing systems are used. In built-up installations, unless special isolation joints are used, movement tolerances are very small and deflection and movements must be treated on an absolute basis consistent with the details.

Details at penetrations for such items as soil stacks, electrical conduit and roof drains must allow for vertical movement of the roof structure independent of these items, which may be rigidly attached to other elements such as the floor below.

Drainage Requirements

To ensure adequate drainage, the roofing industry conventionally called for roof slopes on the order of 1/8 in. to 1/4 in. per ft. The NRCA acknowledges that building codes now set limits on the minimum slope for various membrane types (see Table 1). The NRCA cautions that a strict adherence to a minimum slope such as 1/4 in. per ft may not result in positive drainage due to camber or "varying roof deflections."

The IBC and NFPA *5000 Model Building Codes* provide the following minimum slopes for standing seam and membrane roofs:

1. Standing seam metal roofs systems; 1/4 in. per ft
2. Built-up roofing; 1/4 in. per ft, except coal tar, which requires 1/8 in. per ft
3. Modified bitumen roofing; 1/4 in. per ft
4. Thermoset single-ply roofing: 1/4 in. per ft
5. Thermoplastic single-ply roofing: 1/4 in. per ft
6. Sprayed polyurethane foam roofing: 1/4 in. per ft
7. Liquid applied coatings: 1/4 in. per ft

Maximum deflections are outlined in these codes and standards:

- *AISC Specification for Structural Steel Buildings, Load and Resistance Factor Design* (AISC, 1999)
- *Specification for Steel Hollow Structural Sections, Load and Resistance Factor Design* (AISC, 2000)
- AISC 335-89s1, Supplement No.1 to the *Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design* (AISC, 2001)
- *North American Specification for Design of Cold-Formed Steel Structural Members* (AISI, 2001)
- *Standard for Cold-Formed Steel Framing—General Provisions* (AISI, 2001)

- *Standard for Cold-Formed Steel Framing—Truss Design* (AISI, 2001)

- *ASCE 3, Standard for the Structural Design of Composite Slabs* (ASCE, 1991)

- *ASCE 8-SSD-LRFD/ASD, Specification for the Design of Cold-Formed Stainless Steel Structural Members* (ASCE, 2002)

- SJI *Standard Specifications, Load Tables and Weight Tables for Steel Joists and Joist Girders*. See references.

Model building codes require that the deflection of structural members divided by the span, *l*, not exceed certain values. For example, see Table 1604.3 of the *International Building Code* or Table 35.1.2.8.1.1 of the NFPA *5000 Building Code*. Some applicable provisions from these references are excerpted in the table on page 10.

Roof slopes can be directed to drains by sloping the structure, using tapered insulation, sloping fill, or by using a combination of these methods. Roof drains, gutters or scuppers are located at the low points. As the NRCA notes, from time to time, roof drainage points do not wind up at roof low points and can cause problems for the structure.

It at first seems logical that roof drains should be located at mid-span or mid-bay to take advantage of the low point created by deflection. The elevation of this low point is, however, very difficult to control and can easily be negated by camber (such as member curvature not requested but naturally occurring nonetheless) or upward deflection due to patterned loading in continuous designs.

If, on the other hand, drain points are located at columns, more control is possible. Within the limits of fabrication and erection tolerances, columns are known points of relative elevation. To ensure proper drainage to a low point at a column, the maximum deflection in the zone around the column must result in elevations that remain higher than the drain. This criterion must be used to set elevations of supports radiating from the low point.

Metal Roofs

Metal roofs are of two types:

Through Fastener Roofs (TFR)

Standing Seam Roofs (SSR)

Standing Seam Roofs, for the purpose of this discussion, include only those of the floating type. Standing seam roofs without the floating feature should be treated as Through Fastener Roofs.

The field of a metal roof must, at times, be divided into sections. In general, the limitations on section size are as follows. For TFR the direction parallel to the ribs is limited to roughly 100 to 200 ft, to control leakage at fasteners due to elongation of the holes. Most metal building manufacturers rely upon purlin roll to reduce slotting of the roof panels. Because of their inherent greater stiffness, steel joists should not be used with through fastener systems. SSR is limited based on the “theoretical” maximum movement of the hold down clips. Depending on the manufacturer, this limitation is in the range of 150 to 200 ft.

Drainage Requirements

The strict control of vertical deflections for metal roofs is only limited near the (eave) ends and edges (rakes). In the field of the roof, the deflection of purlins can be limited to span divided by 150 for roof snow load. A maximum absolute limit on deflection has not been specified since the roofing experiences approximately the same curvature, as the deflection limit increases with span. Setting a maximum absolute limit would control behavior relative to other objects within the building. This aspect is covered in the sections on partitions and ceilings and equipment.

Along the gutters, it is essential that there be positive drainage after the roof is deflected under design load. Because the perimeter framing may be stiffer than the first interior purlin, a deflection check should be made to prevent standing water between the eave and first interior purlin. In the case of side edges, as in the case of membrane roofs, there could be separation in the flashing detail between wall and roof. This is a matter of limiting the vertical deflection to that which can be tolerated by the detail.

The concern for maintaining drainage on the overall roof is largely eliminated by the relatively large pitches used for metal roof buildings. They are on the order of $^1/_2$ in. per ft for TFR and on the order of $^1/_4$ in. per ft for SSR. Model Building Codes require a slope of at least $^1/_4$ in. per ft. However, it is essential that the deflection of purlins and rafters be checked to ensure positive drainage of the roof under load. This includes dead load and superimposed loads.

It is recommended that the superimposed load be 50 percent of the roof snow load with a minimum of 5 psf. Roof snow loads are used as opposed to roof live loads, because minimum specified live loads are a strength issue rather than a serviceability issue. For those structures without ceilings or equipment hanging from the roof, this check for drainage is the only check that needs to be made.

Because the drainage for metal roofs is universally at the eaves into interior or exterior gutters or onto the ground, a discussion of the location of drainage points is not required. The concern for the proper detail of penetrations and through roof pipes and conduits remains and the key to resolving these issues is to have details that isolate the pipes, etc., from the structure and roof.

Chapter 3
Design Considerations Relative to Skylights

The design concerns surrounding skylights relate to cladding, in that deflection must be controlled to maintain consistency with the skylight design and to ensure air and watertight performance of the skylight. As always, one could insist that the skylight manufacturer simply make the design conform to the building as designed, but as a practical matter it is more reasonable to match the limitations of the manufacturer's standard design and detailing practices.

Skylights come in a variety of geometries including planar, pyramidal, gabled, domed and vaulted. They are generally supported by the roof structure. When considering the interaction of the skylights with the primary structure, it is important to determine if they rely on horizontal as well as vertical support for stability. This will determine the loading of supports and indicate the nature of controls on support deflection.

The primary reasons for controlling support point displacements for skylights are to:

1. Control relative movement of adjacent rafters (warping of the glass plane).

2. Control in plane racking of skylight frame.

3. Maintain integrity of joints, flashings and gutters.

4. Preserve design constraints used in the design of the skylight framing.

Control of Support Movements

The control of support point movements is best related in reference to the plane(s) of glazing. The two directions of movement of concern for skylight performance are:

1. Movements normal to the plane(s) of glass.

2. Movements parallel to (in the) plane of glass.

Movements in the plane of glass are racking-type movements. The relative displacement of parallel glazing supports must be limited to maintain gasket grip and prevent the light (glass pane) from bottoming out in the glazing recesses. The limits for this movement are $\frac{1}{4}$ in. for gasketed mullions and $\frac{1}{8}$ in. for flush glazing. The relevant loadings for this limit are those that are applied after the skylight is glazed.

Movements normal to the plane of glass are more difficult to describe. These movements are in two categories:

1. Absolute movement of individual members.

2. Relative movement of adjacent members.

The movement (deflection) of individual supporting beams and girders should be limited to control movement of the skylight normal to the glass to span divided by 300, to a maximum of 1 in., where span is the span of the supporting beam. The loading for this case includes those loads occurring after the skylight is glazed.

Additionally, the relative movement of adjacent supports must be considered. There are two aspects of this. The first is spreading (or moving together) of supports. Spreading of supports is to be measured along a line connecting the supports and should be limited as follows:

$\frac{1}{8}$ in. for alpha less than or equal to 25°

$\frac{5}{16}$ in. for alpha between 25 to 45°

$\frac{1}{2}$ in. for alpha greater than or equal to 45°

where alpha is the angle between the line drawn between supports and a line drawn from a support point through the ridge of a gabled skylight or the crown of a vault or arch. The second consideration is control of relative support movement as deviations measured perpendicular to the line drawn between the support points. This limit is the support spacing divided by 240, with a maximum of $\frac{1}{2}$ in. The appropriate loading for both cases of relative movement is those loads that will be applied after the skylight is glazed. See the figures accompanying the summary tables in the Appendix.

The general issue of deflection prior to the setting of skylights is important and must be addressed. The deflections of the support structure must be controlled to provide a reasonable base from which to assemble the skylight and install the glazing. To accomplish this, the maximum deviation from true and level should be plus $\frac{1}{4}$ in. to minus $\frac{1}{2}$ in. Because the concern is the condition at the time of setting the skylight, this can be controlled by a combination of stiffness and camber as required.

Although not strictly a serviceability design consideration, the design of the interface between skylight and structure must consider gravity load thrusts at support points. It is possible to make stable structures that anticipate or ignore gravity load thrusts. If the thrust loads are anticipated and accounted for in the structural design, problems are avoided. If, on the other hand, the structural engineer has not provided for gravity load thrusts and the skylight design has counted on thrust resistance, there could be severe problems.

All vaults, pyramids, and three-hinged, arch-type structures exert lateral thrusts under gravity loading. The construction documents must clearly spell out the provisions made for gravity load thrusts and whether or not the skylight supplier is allowed to choose structure types that require gravity load thrust resistance for stability or deflection control. As always, attention to detail and coordination is critical.

Structural Design Guidelines for Aluminum Framed Skylights, published by the American Architectural Manufactures Association (AAMA) provides the following guidance for deflections as they relate to skylights. The topic addresses three considerations:

1. In-plane deflection.

2. Normal-to-the-surface deflection, and

3. Racking.

With regard to in-plane deflection, AAMA cites the Flat Glass Marketing Association, stating that "in-plane deflection of framing members shall not reduce glass bite or glass coverage to less than 75 percent of the design dimension, and shall not reduce edge clearance to less than 25 percent of design dimension or $^{1}/_{8}$ in., whichever is greater." AAMA recommends that deflection normal-to-the-surface of skylight framing members should not exceed $^{1}/_{175}$ of the span, or $^{3}/_{4}$ in. AAMA provides only a caution that racking is a critical design consideration, but provides no other specific recommendations.

With regard to sidesway of a framed skylight due to lateral loads, AAMA recommends a limit of movement between any two points of "height/160" for glass glazing materials and "height/100" for non-glass glazing materials.

Movement of supports is also addressed in the Guidelines. It states, "horizontal deflection of skylight supporting curbs should be limited to $^{1}/_{750}$ of the curb height or $^{1}/_{2}$ in. unless curb flexibility is considered in the analysis of the skylight frame."

Model building codes address supports for glass. In calculating deflections to check for conformity to deflection limits, it is permissible to take the dead load for structural members as zero. Likewise, in determining wind load deflections, it is permissible to use loads equal to 0.7 times the applicable load for components and cladding.

As stated above, the model building code requirements for deflection limits on the support of glass state "To be considered firmly supported, the framing members for each individual pane of glass shall be designed so that the deflection of the edge of the glass perpendicular to the glass pane shall not exceed 1/175 of the glass edge length or $^{3}/_{4}$ in. (19.1 mm), whichever is less, when subjected to the larger of the positive or negative load where loads are combined as specified in (Load Combinations)."

Additionally, "where interior glazing is installed adjacent to a walking surface, the differential deflection of two adjacent unsupported edges shall not be greater than the thickness of the panels when a force of 50 pounds per linear foot (plf) (730 N/m) is applied horizontally to one panel at any point up to 42 in. (1067 mm) above the walking surface."

Chapter 4
Design Considerations Relative to Cladding, Frame Deformation, and Drift

In current practice a distinction is made separating the structural frame from the non-structural systems and components of a building. The foundations and superstructure frame are primary structure whereas the curtain wall and roofing are not. Despite this separation, what is produced in the field is a single entity—a building. It is this entity that receives the ultimate scrutiny regarding its success or failure.

Cladding-Structure Interaction

The primary means of controlling the interaction between cladding and structure is isolation (divorcement in the words of the Commentary to the AISC *ASD Specification*). Divorcement prevents the inadvertent loading of the cladding by movements in the primary and secondary structure and is achieved by subdividing the cladding with joints and by attaching the cladding to the structure in a manner that is statically determinate. Using a statically indeterminate attachment would require a compatibility analysis of both cladding and structure as a composite structure.

In addition to proper connections, the other key design element is joint behavior. Joints are filled with sealants and gaskets. Movements must be controlled so that these materials function as intended in their design. The cladding for a building can be either sole-source, such as from a metal curtain wall manufacturer or can be built up from a number of disparate elements such as masonry and window units. Each type of cladding has unique design concerns beyond those related to cladding in general.

Vertical support of cladding can be accomplished in three ways. For one- and two-story buildings, it is often feasible to support the cladding on the foundation with the only ties to the frame being those connections required for stability and for lateral loads. Secondly, cladding systems consisting of bay-length spandrel panels or bay-sized panels can be supported at the columns. These connections should be appropriately detailed to maintain the statically determinate condition of support mentioned above. The third method of support is for those cladding systems that require support along the perimeter horizontal framing. The concerns for frame and cladding interaction escalate through these three methods to the special analysis, design and detailing issues associated with tall buildings.

In addition to the deformations of the structural frame due to dead and live loads, as will be discussed in detail below, the primary load affecting the performance of cladding is wind load. As mentioned earlier, one of the three factors in the assessment of serviceability is load.

For the evaluation of frame drift, ten-year recurrence interval winds are recommended due to the non-catastrophic nature of serviceability issues and because of the need to provide a standard consistent with day-to-day behavior and average perceptions. The 50-year recurrence interval winds that strength design wind loads are based upon are special events. In lieu of using the precision of a map with ten-year wind speed isobars, the authors recommend using 75 percent of 50-year wind pressure as a reasonable (plus or minus 5 percent) approximation of the ten-year wind pressures. The Commentary to Appendix B of ASCE 7-02 recommends 70 percent.

For further discussion of suggested recurrence intervals for loads in serviceability designs, see Davenport (1975), Ellingwood (1989), Galambos and Ellingwood (1986), ISO Standard 6897 (1984), Hansen, Reed and Vanmarcke (1973), Irwin (1978), Irwin (1986) and the Commentary to Appendix B of ASCE 7-02.

Foundation-Supported Cladding for Gravity Loads

When vertical support along the foundation supports the cladding, there is no connection between frame and cladding for vertical loads and the limits on vertical deflection are:

1. Roof and floor beams must have deflections compatible with the type of vertical slip connections detailed to laterally support the cladding.

2. Roof beams must have deflections compatible with the perimeter termination of the roofing membrane to cladding.

3. Floor beams must have deflection compatible with the detailing between wall and floor finish.

4. Floor and roof members must have deflection compatible with the detail of ceilings and cladding.

Because this method of vertical support is only useful for relatively short buildings (one or two stories), the shortening of columns is not a concern. However, it is possible that differential thermal expansion could be a concern and this requires care in detailing the joint between interior partitions and the cladding, requiring an isolation joint.

Horizontal deflection of the superstructure frame and its effect on the cladding is of a more serious concern in this first method of support. The two modes of frame movement are:

1. Those perpendicular to the plane of cladding.

2. Those parallel to the plane of cladding.

The concern for horizontal frame deflection varies depending on whether the cladding lateral support is statically determinate or statically indeterminate. If the cladding has only a single tieback connection to the roof, lateral deflection perpendicular to the plane of the cladding is:

a. Of little concern in the case of metal panel systems

b. Of moderate concern for tilt-up concrete and full height precast systems

c. Of great concern in masonry systems

In metal systems the limitation is the behavior of the joints at the building corners. The wall parallel to the direction of movement does not move whereas the wall perpendicular to the movement is dragged along by the frame deflection. The allowance for movement at corners is generally a function of the corner trim and its inherent flexibility. Corner trim flexibility generally explains why metal clad buildings designed to a drift limit of height divided by 60 to height divided by 100 with ten-year wind loads have performed successfully in the past.

Tilt-up Concrete Support

The case of tilt-up concrete and full-height precast is of only moderate concern because the steel frame can drift and the simple-span behavior of the panels is preserved. Again, the critical detail remains the corner. Thus, drift limits in the range of height divided by 100 are appropriate with ten-year wind loads. It should be noted that, in some cases, precast panel walls and tilt-up walls are buried in lieu of a foundation wall. In these cases, drift must be limited to control cracking since these panels are now rotationally restrained at their bases.

Metal Panel Support

Metal panel systems are usually supported by girts spaced at intervals up the frame from base to eave. The spacing of the girts is a function of the overall wall height, the height and location of openings, the loads on the wall, the properties of wall panel system and the properties of the girts themselves.

Girts are supported by the exterior columns and, in some cases, intermediate vertical elements, called wind columns. Wind columns have top connections that are detailed to transfer lateral load reactions to the frame without supporting gravity loads from above.

For the design of girts and wind columns supporting metal wall panel systems a deflection limit of span divided by 120 using ten-year wind loading is recommended for both girts and wind columns. The wind loading should be based on either the "component and cladding" values using ten-year winds or the "component and cladding" values (using the code required "basis wind speed") multiplied by 0.7, as allowed in footnote f in IBC 2003, Table 1604.3 and footnote 3 in NFPA 5000 Table 35.1.2.8.1.1.

Masonry Wall Support

Perimeter masonry walls require a more detailed presentation because of the unique nature of masonry, which has flexural stiffness with little flexural strength. For example, a 12 in. segment of 12 in. concrete block (face shell bedded) has a moment of inertia of 810 in.[4] However, it has a flexural strength of only 2.8 to 4.6 in.-kips based upon an allowable stress of 20 to 33 psi (as provided in ACI 530-02). A 12 in. wide-flange column with a comparable moment of inertia adjusted for the difference in modulii of elasticity can develop a moment of 280 in.-kips. This wide variation in strength is, of course, due to the wide variation in allowable bending stresses, which is due in part to the ductile nature of steel and the brittle nature of unreinforced masonry.

One can improve the flexural strength of masonry with reinforcement. The 12 in. wall in this example can have its strength increased by a factor of ten to fifteen times with vertical reinforcement. In unreinforced masonry, a crack at a critical cross section is a strength failure. In reinforced masonry, a crack means the reinforcement is functioning, and thus cracking is only a serviceability concern. The increased strength and ductility of reinforced masonry clearly makes it a superior choice over unreinforced masonry. Although this discussion concerns the design of masonry walls, masonry design issues concern the designers of steel building frames because masonry walls are in almost all cases supported by the steel frames for lateral stability.

The design of masonry exterior walls must take into account the nature and arrangements of supports. In general, perimeter walls are supported along their bottom edges at the foundation. They are additionally supported by some combination of girts, the roof edge, columns and wind columns. All of these elements, with the exception of the foundation, are elements of the structural frame and will deflect under load. What confronts the designers of the masonry is the problem of yielding supports. The actual behavior of the wall and its supports is dramatically different from the behavior predicted by design models based on non-yielding supports.

There are several methods for properly accounting for support conditions in the design of masonry on steel. They include:

1. Make no allowance in the steel design and force the design of the masonry to account for the deflecting behavior of the steel.

2. Limit the deflection of the steel so that it is sufficiently rigid, nearly achieving the idealized state of non-yielding supports.

3. Provide some measure of deflection control in the steel and design the masonry accordingly.

The first and second solutions are possible, but not practical. The first requires analysis beyond the scope of normal building design—a three-dimensional analysis of the structure and the masonry acting together. The second is also nearly impossible in that it requires near-infinite amounts of steel to provide near-infinite stiffness. The third approach is a compromise between the two other solutions, which involves reasonable limits for frame drift and component deflections (girts, columns, wind columns, etc.) and recognizes that the design of the masonry must conform to these deformations.

The aspect of the masonry design at issue is an analysis to determine the magnitude and distribution of shears and moments. The model commonly used is that of a plate with one- or two-way action, having certain boundary conditions. It is these boundary conditions that must be examined.

The first boundary condition to be examined is the base of the wall. Although it may be a designer's goal that the base of the wall should not crack, the authors have concluded that this is an unrealistic and unachievable goal due to the relatively low strength of unreinforced masonry. A more realistic approach is to limit frame drift so as to control crack width and to provide a detail to ensure that the crack occurs at a predictable location, presumably at the floor line. The detail itself requires careful consideration (see Figure 1). One must also inform the owner of the anticipated behavior.

It is recommended that the frame drift under the loads associated with ten-year wind be controlled so as to limit crack width to 1/8 in. when a detail such as that of Figure 1 is used, and 1/16 in. when no special detail is used. This cracked base then becomes the first boundary condition in the design of the masonry panel. The model for the panel must show a hinged base rather than a fixed base. The foregoing limits are applicable to non-reinforced walls. Where vertical reinforcing is required for strength reasons, it is recommended that the drift limit be changed to height divided by 200. A limit of height divided by 100 can be used if a hinge type base (see Figure 2) can be employed.

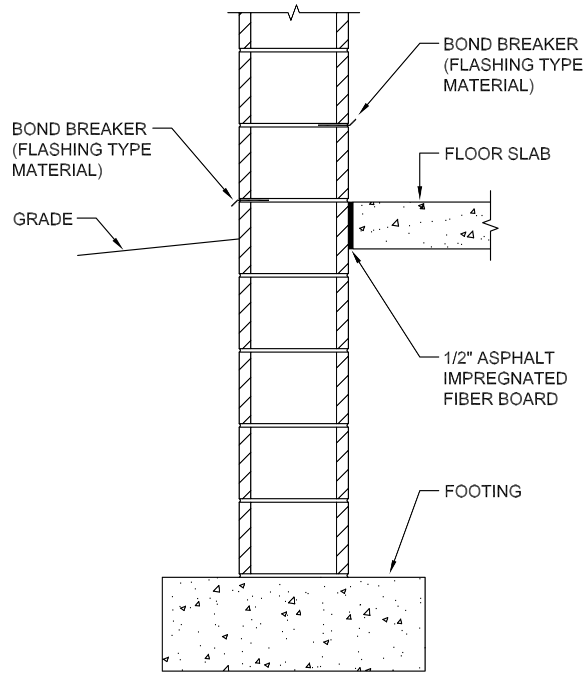


Fig. 1. Masonry horizontal control joint

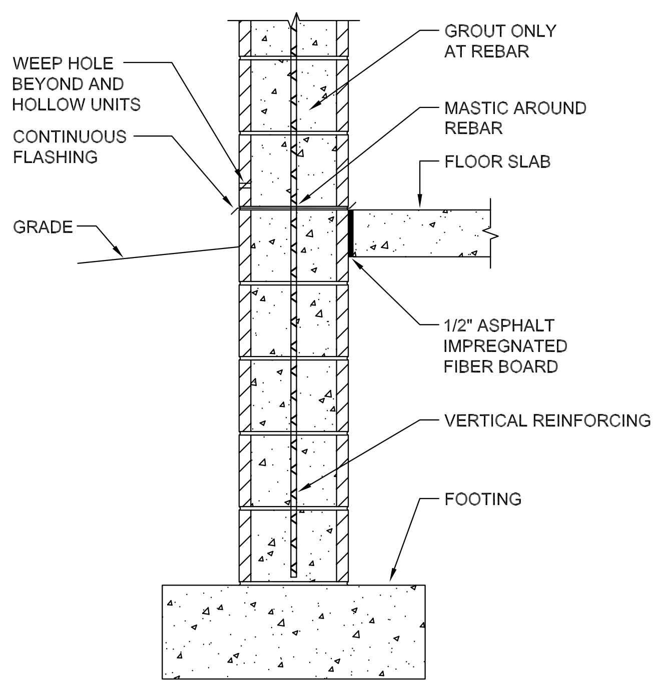


Fig. 2. Masonry horizontal control joint

The remaining panel boundary elements are the components of the structural frame, which require deflection limits compatible with the masonry. Based on numerous finite element models of wall panels and supporting framing, the authors have noted two consistent trends. First, almost categorically, the change from a rigid support to a yielding support can increase moment magnitudes by a factor of two to three. Second, because of the great stiffness of the masonry panel itself, it is very difficult to provide reasonably sized support elements with sufficient stiffness to significantly alter the distribution of panel shears and moments. Thus, design and detailing of the masonry is the critical element in this relationship, not the design of the steel frame.

A model consisting of non-yielding supports and a fixed base is not accurate. Fortunately, in practice, the increased moment results in stresses within the range of ultimate bending stresses in the masonry and in the case of reinforced masonry the material ductility mitigates the problem. What is obvious is that controlling steel deflections is not the solution. In order that support deflection not be totally neglected, a limit of span divided by 240 with maximum absolute value of 1½ in. is recommended for girts and columns supporting masonry under a load associated with a ten-year wind.

One specialized case of masonry wall is that of the wainscot wall. The top of this masonry wall is usually six to eight feet above the floor and the remainder of the wall is metal panel. The junction between top of the masonry and bottom of wall panel can be accomplished in three ways:

1. Isolation of masonry and panel with separate supports;
2. Attachment of the wall panel to an angle attached to the top of the masonry; and
3. Attachment of both the masonry and the panel to a common girt.

Each method has unique design considerations and is workable. As always, their success or failure depends on the details.

In the first case, the masonry and wall panel girt must be checked to limit relative deflection so that an air-tight and water-tight joint can be provided which will move but not leak. This system has the advantage of a smaller girt since there is lesser load on the girt.

In the second approach a girt is eliminated. However, the wall and its connections to the columns must be strong enough to carry not only the wind load on it but also the wind from the bottom span of wall panel.

The third approach requires the largest girt, but the problem of masonry/wall panel differential deflection is eliminated. Additionally, the girt/column connection provides the top of wall anchor, thus eliminating a connection between masonry and building column.

The recommended limit for the girt supporting the wall panel above the masonry wainscot wall (as in the case of the all metal panel wall) is span divided by 120 using ten-year wind loads. An absolute maximum deflection depends upon the girt supported equipment, if any, and the relative deflection between roof edge or wall base and first interior girt. The main wind-force-resisting system loads should be used. For both a full height and wainscot wall, it is not necessary to combine the drift of the frame and the roof diaphragm or the deflection of the girt at the top of the wainscot wall. Both before and after a crack forms at the base of the wall, the frames, grits and the masonry wall represent a complex indeterminate system.

Modeling the bare frame, while not perfect, is adequate for the task at hand. The simple addition of the drift and deflection values will overestimate the situation and add unnecessary cost to the construction. The wind load at the base of a structure is probably over-estimated by current standards due to obstructions and ground drag. In a life-safety code, the over-estimate of load is not a detriment. In a serviceabity check, the over-estimate of load is not necessary and is objectionable.

As mentioned earlier, there is also a concern for parallel movement of the frame behind the cladding. There should be isolation between wall and frame by means of sliding or yielding connections. Thus, the movement is only limited by the flexibility of the roofing/wall joint and the floor/wall joint. The practical limitation is joint behavior at the intersection of parallel and perpendicular walls as noted earlier.

Frame-Supported Cladding at Columns

The second method of support for cladding, i.e. cladding that spans between columns, is sometimes used for buildings. In this case, the frame carries both the vertical and horizontal forces from the cladding, but the support points are limited to points on or very close to the columns. In the idealized case, there are two support points that carry vertical and lateral loads and two that carry lateral loads only. These supports must be detailed to slide or yield under horizontal forces in the plane of the panel with the exception of one joint, which is required for horizontal shear stability. The success or failure of this method depends on the relative movement of the support points.

Vertical movement is the result of absolute and relative column shortening (and lengthening). The vertical movement affects the performance of the panel perimeter caulk joints. This movement should be limited to about ¼ in., due to ten-year wind load or 50 percent of design live load. The other concern is racking of the bay. First, the racking must be within the limit of movement of the connections, and secondly the racking must be within the limit of the movement provided for between panels in adjacent stories. The junction of four panels, where the sealant takes on a cross pattern, is a critical location (Bergmann, 1988). Relative movement between stories can introduce shearing forces in the intersection of the horizontal and vertical sealants. While the limit on racking is a function of connection design and joint detailing, one can use a maximum interstory drift of story height divided by 500 using a ten-year wind load as a target limit with reasonable assurance.

Frame-Supported Cladding for Gravity Loads Along Spandrels

The third method of support, i.e., support along the spandrels, is the most complex and results in the most problems. In this method, there are the concerns of the methods discussed earlier, with the added issue of deflections of the perimeter framing. Again there is the concern of determinate versus indeterminate attachment. The timing of the application of loads is significant. First, deflections prior to setting of cladding are important since the fabrication of cladding may, in all likelihood, be based on idealized constant story elevations. Secondly, deflections during the setting of heavy cladding must be considered as component alignment may be affected. Lastly, deflections after the completion of cladding must be consistent with its detailing.

It is inevitable that the cladding will not be in the plane of the perimeter framing, so the effects of cantilever support deflections and/or the movements created by rotation (torsion) of the parallel spandrel must be considered. In the case of determinate attachment of cladding, the concerns of perimeter beam deflection relate to the erection and in-service performance of joints and details. In the case of an indeterminate system, the concerns must also include a deflection limit that controls stresses in the cladding material.

In general, the vertical deflection of perimeter framing should be limited to span divided by 480 for total dead load, with an absolute limit of $\frac{3}{8}$ in. due to dead loads imposed prior to setting the cladding and an absolute limit of $\frac{5}{8}$ in. dead load deflection after setting the cladding.

The effect of setting heavy units sequentially down the length of a perimeter framing element should be considered when the cladding weight exceeds 25 percent of the total dead load on the beam. In this case, the deflection due to cladding and initial dead load should be limited to span divided by 600 with an absolute limit of $\frac{3}{8}$ in.

The limits on vertical deflection after the completion of cladding must be consistent with the joints and details and relate primarily to the relative deflections between floors. For example, glass can pull out of the glazing stops attached to the floor above. Interlocking mullion expansion joints can disengage. Windows in continuous slip heads could jam or disengage. Precast or stone panel vertical joints can open excessively at the base and squeeze closed at their tops (PCI, 1999). To prevent these problems and others like them, one must limit live load deflection to span divided by 360 with a maximum of $\frac{1}{4}$ in. to $\frac{1}{2}$ in. depending on the details. Consideration must be given to the magnitude of live load (that is load after erection of cladding). It is the nature of live load specifications to err on the high side. Thus, the reasonably expected live load in the perimeter zone of the building would generally be less than that specified. This is due to the relatively low density of use of the floor space near the windows. Also, consider not using the full live load because the design consideration is the differential movement between floors. It may be reasonable to assume some load on all floors (except the top and bottom stories). For these reasons, consider using 50 percent of the design live load.

Walls that are continuously supported along a floor or roof such as masonry walls or stud walls are supported in an indeterminate manner and require compatibility analysis, or more commonly strict deflection limits, to control damage to the cladding.

The limits on deflection given by the Brick Institute of America (BIA) for lintels are maximum total load deflections of span divided by 600 but not more than 0.3 in. (BIA, 1987, 1991). The absolute limit governs for spans divided by 15 feet and is consistent with typical joint details at ledges and window heads. BIA limits lintel rotation to $\frac{1}{16}$ in. The authors have taken this to mean a $\frac{1}{16}$ in. tip from heel to toe of a single support angle, which is an approximate rotation of 1 degree. ACI 531 also gives deflection limits for masonry beams and lintels as span divided by 360 for total load and span divided by 600 for dead load only. Limitations for built-up insulation systems on studs are such that the limits given for the determinate systems would apply to these as well.

It should be noted that deflection and drift limits must be compared to calculated deflections, which include the effect of creep as appropriate, as in the case of composite beams.

Installation of Aluminum Curtain Walls, published by the American Architectural Manufactures Association (AAMA) provides a useful, but general, discussion of the relationship of the curtain wall and the building frame, focusing on tolerances and clearances.

Special Considerations for Tall Buildings

Many of the issues discussed for column and frame supported cladding also apply to tall buildings, but there are additional considerations that apply as buildings increase in height. The majority of concerns center on the need for an accurate determination of the deformation and drift behavior of the frame. Needless to say, inaccuracies in modeling that are inconsequential in a short frame may result in significant problems in a tall frame. For example, the frame analysis (Griffis, 1993) should "capture all significant" effects of:

1. Flexural deformation of beams and columns.
2. Axial deformation of columns.
3. Shear deformation of beams and columns.
4. P-Δ effect.
5. Beam-column joint deformation.
6. Effect of member joint size.

The first four effects are addressed in most currently available analysis software. The last three may or may not be addressed, depending on the sophistication of the program. The effects on beam-column deformation can be significant. An in-depth discussion of this important topic is beyond the scope of this Guide. The reader is referred to Charney (1990) for a detailed discussion on beam-column deformation, including the presentation of an approximate method to correct for this effect using "modified beam and column moments of inertia and shear areas to compensate for deformations occurring inside the joint." "The P-Δ effect can easily increase total frame displacement by 10 to 15 percent depending on frame slenderness" (Griffis, 1993). An accurate determination of frame stiffness is also important in establishing the building period, when assessing seismic loads, the dynamic (resonant) component of wind loading, and in determining wind accelerations for evaluation of perception of motion.

Another aspect of tall building behavior as it relates to cladding behavior is column shortening and differential column shortening. Design and construction attention must be given to the issue of column shortening in the form of movement tolerant joints, adjustable details, and shimming of the frame as it is being erected. This last item is discussed further in the section on floors.

Chapter 5
Design Considerations Relative to Interior Partitions and Ceilings

The performance of exterior walls and roofs is generally judged by their ability to not leak air or water. The performance of interior partitions and ceilings is largely aesthetic and relates to cracks and bows. Most finish materials are brittle and thus have little tolerance for inadvertent loading due to deflections. The only notable exception to this is ceiling construction of metal grids and lay-in acoustical panels.

Support Deflection

One common criterion in literature on this topic is the limitation on floors and roofs supporting plaster ceilings that live load deflection not exceed span divided by 360. This is found in AISC ASD Section L3. Likewise, paragraph 5.9 Deflection of the *SJI K-Series Joist Specification* requires that design live load deflection not exceed $1/360$ of span for floors. Two limits are given for roofs, $1/360$ of the span where a plaster ceiling is suspended from the framing and $1/240$ of the span for all other cases. The specifying professional is required to "give due consideration to the effects of deflection and vibration in the selection of joists." These requirements are repeated in the SJI *LH-* and *DLH-Series Specification* in paragraph 104.10 and in paragraph 1004.7 in the SJI *Standard Specifications for Joist Girders*.

These limits produce deflected curvatures that are on the borderline of acceptable visual perceptibility. Other considerations may require stricter absolute limits on deflection. For example, where drywall partitions meet drywall or plaster ceilings, standard details allow for only $1/4$ in. to $1/2$ in. of movement. This is, in general, a stricter limit than span divided by 360. An alternative to providing a stiffer structure is to support the drywall ceilings from ceiling framing that is supported by the partitions rather than suspend the ceiling from the structure above. This solution may only be appropriate for relatively small rooms such as individual offices.

Another alternative is to enlarge the joint between wall and ceiling. This would require non-standard detailing and consequently a higher standard of care. Ceilings of metal grids and acoustical panels are also of concern. Ceilings of this construction generally have a high tolerance for distortion due to the loose nature of their assembly. The one exception to this general characterization is the perimeter detail. In standard installations this consists of a painted metal angle attached to the walls around the perimeter of the room. The metal ceiling grid bears on this angle, as does the perimeter row of ceiling panels. With this rigid perimeter, the remainder of the ceiling (suspended from the floor above) cannot deflect more than $1/4$ in. to $1/2$ in. without some distress. As in the case with plaster and drywall ceilings, the alternative to controlling deflections in the structure above is to isolate the ceiling perimeter. This can be done, but it requires extra hangers and a non-standard attachment of the perimeter trim. Additionally, a flexible dust membrane may be needed. In buildings where the ceiling is used as a return air plenum, a detail must be devised to maintain the effectiveness of this plenum.

The foregoing discussion is directed to downward deflections of the framing supporting the ceiling from above. There is also a concern for floor deflections, which draw the partitions downward relative to the ceiling. This situation is usually of lesser concern since the deflection magnitude is the net difference of the deflection of the two levels (except in the top and bottom stories).

Deflection of floors is also of concern as it relates to the behavior of partitions. Since the floor supports the walls, the walls are of necessity forced to conform to the deflected contour of the floor both as the walls are erected and after the walls are in place. In general, walls can be thought of as deep beams or diaphragms. Thus, they have some ability to span over places where the floor deflects downward beneath the partition. The most vulnerable point in the wall is at the upper corners of door openings for two reasons: firstly because of longitudinal shrinkage of the wall itself, and secondly because of the discontinuity of the wall acting as a beam. The door head is the weak point in the overall wall and can crack as the wall attempts to follow its deflected support.

Thus, as is frequently the case, the solution to structure-partition interaction is effective control jointing and isolation. It is recommended that control joints be placed at the upper corners of doorways and at intervals along walls that are not pierced by doors. The spacing of such joints is suggested to be 30 ft or closer (U. S. Gypsum, 2000). Other references would restrict the aspect ratio of the panel to 2:1 to 3:1 (Nemestothy and Visnovitz, 1988).

Flat and Level Floors

As in the case of a spandrel supporting a curtain wall along its length, the behavior of floors is sometimes a problem as they deflect under successive applications of dead and live load. One common example of this is beam deflection during concreting operations and the possibility of complaints from finishing contractors over uneven floors.

The most common floor construction in many low-rise and most mid- and high-rise office and other similar structures consists of a cast-in-place concrete slab on composite steel deck supported on composite steel beams and girders. In recent years, situations that have arisen during construction have raised concerns about the flatness and levelness of floors and the means required to achieve these specified conditions. Both the use of higher strengths of steel and the use of camber in the frame have amplified the degree of concern over the topic.

The owner/occupant of these structures desires that the floors be flat and level but also expects to receive the project for the most economical price possible. For the sake of economy, composite construction is often employed. By their nature, composite beams provide significantly greater strength and stiffness than the base steel beam in the noncomposite condition. Framing is commonly cambered for the expected dead load with the expectation that the beams will deflect to level during concreting. The deck or framing is rarely shored during concreting operations.

While the framing system described above is common and efficient, it is not without its pitfalls in design and construction. For example, using the nominal floor elevation and nominal top of steel as the actual condition, initially the tops of the cambered beams rise above the plane established by the nominal top of steel. In all designs, a nominal thickness of concrete is established over the top of the deck. The slab thickness is generally set by strength requirements and is frequently part of the fire rating of the floor system.

Tolerances for cast-in-place concrete construction are established by ACI Committee 117 in its report "Standard Specifications for Tolerances for Concrete Construction and Materials (ACI 117, 1990)." In paragraph 4.4.1, the tolerance for slabs 12 in. or less in thickness is plus $^3\!/_8$ in. and minus $^1\!/_4$ in.

The first preference among concrete contractors in casting slabs is to strike the concrete to a constant elevation without regard to the contour of the deck and framing. When beams are cambered and minimum slab thicknesses are maintained, this approach raises the actual top of concrete above the nominal top of concrete, potentially affecting pour stops, stairs, curtain walls, etc. This approach also increases the volume and weight of concrete on the structure, which in turn affects the required resistance and deflection response of the framing. Thus, in most cases, it becomes necessary to set screeds to follow the curvature of the cambered beams to maintain the slab thickness within tolerance. This may also be required to maintain cover over the top of the shear connectors. Per the AISC *LRFD Specification* Section 5a(2), a minimum of 2 in. of concrete is required over the top of the deck. Screeds may be set to follow the curve of the cambered beams due to either: 1) a lack of understanding on the part of the concrete contractor as to the anticipated deflection of the framing, or 2) over cambering of the framing.

The successful concreting of floors on steel deck and framing is an art. In addition to the skills required to place and finish concrete, the work is performed on a deflecting platform. It is essential that the concrete contractor be experienced in this type of work. Also, the contractor must be informed as to the basis for the cambers specified and the expectations of the structural engineer with regard to deflections during concreting. The Engineer's expectations for the behavior of the structure can be conveyed in the construction documents and during a preconstruction meeting.

It is in the nature of structural engineering and design to overestimate loads and underestimate resistance. With regard to the calculation of expected deflections during concreting, this rubric will likely result in over cambered beams and the need to have the slab follow the cambered curve. Ruddy (1986, 1996) in two papers on this subject emphasizes the need to accurately determine loads and the deflection response. For example, he notes that the deck will deflect during concreting and recommends that the nominal weight of the concrete slab be increased by ten percent to account for this. Additionally, while it is essential to account for the weight of workers and equipment for strength, these loads are transient and should not be overestimated in determining deflection. Perhaps Ruddy's more significant insight is that the effects of end connection partial restraint should be considered in the calculation of deflections even though the members in question are considered simple span members. Ruddy's proposal is to reduce the estimated simple span deflections to 80 percent of the calculated values when setting cambers.

Specifying Camber and Camber Tolerances

Camber tolerances are established in the AISC *Code of Standard Practice* as follows in Section 6.4.4:

> "For beams that are equal to or less than 50 ft in length, the variation shall be equal to or less than minus zero/plus $^1\!/_2$ in. For beams that are greater than 50 ft in length, the variation shall be equal to or less than minus zero/plus $^1\!/_2$ in. plus $^1\!/_8$ in. for each 10 ft or fraction thereof in excess of 50 ft in length."

These tolerances are set with the worthy goal of ensuring positive camber, but it should be noted that there is a bias toward over cambering.

The AISC *Code of Standard Practice*, in Section 6.4.4, states: "For the purpose of inspection, camber shall be measured in the Fabricator's shop in the unstressed condition." This requirement is further amplified in paragraph 8.5.2, which states: "Inspection of shop work by the Inspector shall be performed in the Fabricator's shop to the fullest extent possible." Paragraph 8.5.4 states: "Rejection of material or workmanship that is not in conformance with the Contract Documents shall be permitted at any time during the progress of the work." The inspection of camber is an exception to this general principle. Unlike other physical characteristics of a fabricated beam or girder, such as yield strength, dimensions, welds, etc., the camber in a beam can change as the member is handled, shipped, unloaded and raised into position. The *Code* commentary to paragraph 6.4.4 provides the following explanation of this phenomenon. Camber can vary from that induced in the shop due to factors that include:

a. The release of stresses in members over time and in varying applications:

b. The effects of the dead weight of the member:

c. The restraint caused by the end Connections in the erected state; and,

d. The effects of additional dead load that may ultimately be intended to be applied, if any.

Because of the unique nature of camber in beams and the limits on the inspection for conformity to the project requirements for camber, it is incumbent on the specifier to recognize these limits and prepare the Construction Documents accordingly. The *Code of Standard Practice*, in Paragraph 8.1.1, requires that "The Fabricator shall maintain a quality assurance program to ensure that the work is performed in accordance with the requirements in this *Code*, the AISC *Specification* and the Contract Documents. The fabricator shall have the option to use the AISC Quality Certification Program to establish and administer the quality assurance program."

The AISC Certification Program for Structural Steel Fabricators is set forth in a document entitled *Standard for Steel Building Structures, 2002*. In the section on Fabrication Process Control, it states "The Fabricator will include additional 'special procedures' that cover fabrication processes done at the facility (e.g., cambering)." In the section on Inspection and Testing, the *Standard* states: "The Fabricator shall document a procedure for inspection and testing activities in order to verify that the product quality meets project requirements. The Fabricator will establish in the procedure the level and frequency of inspection to assure expected contract quality." The *Standard* goes on to state: "The inspection procedure shall include receipt, in-process and final inspection of all product furnished to the project. The procedure will include any sampling plan, if less than 100 percent, for each type of inspection."

The inspection procedures prescribed in the *Certification Standard* should provide reasonable and documented evidence that camber was provided, meeting project requirements. In the absence of a Quality Control program such as that provided in the *Standard*, the specifier may wish to consider requiring specific inspections for the quality control of camber. Any requirements for "more extensive quality assurance or inspection…shall be clearly stated in the Contract Documents, including a definition of the scope of such inspection," as provided in paragraph 8.1.3 of the *Code of Standard Practice*.

It is common practice not to camber beams when the indicated camber is $^{3}/_{4}$ in. or less. The AISC *Code of Standard Practice* provides that if no camber is specified, horizontal members are to be fabricated and erect beams with "incidental" camber upward. The AISC *Code* also provides that beams received by the Fabricator with 75 percent of the specified camber require no further cambering. Because of the provisions, it should be expected that all framing members should have at least some upward camber at the initiation of concreting operations. However, given the limits presented there will be instances of downward deflection below level during concreting. To control the excessive accumulation of concrete in the deflected bay Ruddy (1986), quoting Fisher/West in the first edition of this Guide, recommends that the total accumulated deflection in a bay due to dead load be limited to $L/360$, not to exceed 1 in.

The foregoing discussion on determining and specifying camber is intended to impress upon the designer of the framing to be judicious in determining cambers and to be pro-active in communicating the basis of the camber determinations.

Maintaining Floor Elevation

This discussion is premised on the fact the steel framing is set at the nominal top of steel elevation. Needless to say, the actual elevation of the steel framing can vary as permitted by the tolerances established in the AISC *Code of Standard Practice*. These tolerances are presented in Section 7.13.1.2(b), which permits a deviation in the dimension from the working point at the end of a beam connection to a column to the upper finished splice line to be "equal to or less than plus $^{3}/_{16}$ in. and minus $^{5}/_{16}$ in. Note that all other things being equal, the tolerances for the actual framing approximate the deviations permitted by ACI 117 for the variation in slab thickness. AISC *Code of Standard Practice* Section 6.4.1 limits the variation in length of columns fitted to bear to plus or minus $^{1}/_{32}$ in.

In tier construction, these small variations can combine with differential thermal and differential dead load shortening to create deviation in splice elevations in floors as the frame rises. These differences must be shimmed out as the frame is erected to maintain reasonable control of actual floor elevations and differential top of steel elevations across a floor. It is common in mid- and high-rise construction to obtain an as-erected survey of the frame. This survey can be used to direct the concrete contractor as to what adjustments must be made to maintain the slab thicknesses and the top of concrete elevations specified.

The reader is encouraged to refer to the papers cited in the References by Ruddy (1986, 1996), Tipping and Suprenant (1991, 1991), Suprenant (1990), Tipping (1993), and Ritchie and Chien (1992) for a more complete treatment of this topic. Apart from the deflection standards implied in this Guide, there are no published limits for the dead load deflection of floor beams.

Both the Steel Deck Institute (SDI, 2000) and the American Society of Civil Engineers in its *Specifications for the Design and Construction of Composite Slabs*, ASCE 3-91 (ASCE, 1991), give limits for the deflection of metal deck acting as a form. Both give a limit of span divided by 180, with a maximum of ¾ in. deflection under the weight of wet concrete and the weight of the deck (SDI 3.2c (Non-Composite)/3.3(Composite) and ASCE 3 2.2.6). SDI also limits the maximum deflection for form decks to the same constraints. The limit on deflection under superimposed load on the composite section is given as span divided by 360 in SDI para. 5.4 (Composite). The ASCE document limits deflection for a range of span divided by 180 to span divided by 480 depending on conditions. This is presented in Table 2 in the ASCE document, which is an adoption of Table 9.5(b) in ACI 318-89.

Drift, Deflection, and Racking

There is also a concern for partition racking induced by interstory drift. One published source gives drift indices (deflection divided by height) of 0.0025 (1/400) for "first distress" and 0.006 (1/167) for ultimate behavior for drywall on studs (Freeman, 1977). The following deflection limits for both composite and non-composite beams and frame drift are recommended:

For dead load (roof): No limit except (1) as controlled by ponding considerations, (2) as controlled by roofing performance considerations and (3) as controlled by skylight performance. There is no limit as it relates to partitions and ceilings, since these materials are installed after the dead load is in place. In the case of roofs that are concreted, the limits for floors would apply.

For dead load (floor): Span (L) divided by 360 with a maximum of 1 in. This is to be the accumulated deflection in a bay. This is greater than the deflection allowed by ACI tolerances and requires that this deviation be adequately explained and accounted for in the plans and specifications. This deflection limit does not necessarily control ponding of wet concrete, which should be checked separately. The loading for this deflection check is the weight of wet concrete, the weight of steel deck and the weight of steel framing. For composite floors, the deflection limits should be applied to the instantaneous deflection plus one half of the expected creep deflection.

For live load (roof): Span (L) divided by 360 where plaster ceilings are used and span (L) divided by 240 otherwise. A maximum absolute value that is consistent with the ceiling and partition details must also be employed. This absolute limit should be in the range of ⅜ in. to 1 in. Note that movable and demountable partitions have very specific tolerances required for them to function. These special limits are unique to each model and manufacturer and must be strictly adhered to.

In most jurisdictions there is a distinction between live load and snow load. These deflection limits should be checked using 50 percent of the minimum code specified live load or the 50-year roof snow load (including drifting), whichever produces the greater deflection. It should be noted that roof snow loads are used at full magnitude due to their probability of occurrence whereas minimum roof live loads are reduced due to their transitory nature (rain, maintenance, etc.). In those jurisdictions where there is snow, but the roof load is expressed as live load, the use of snow loads from model codes is recommended.

For live load (floor): Span (L) divided by 360 with a maximum absolute value of 1 in. across the bay with 50 percent of design live load (unless the code imposes a stricter standard). The comments in the roof section relating to partitions also apply to floor deflection. Additionally, the limits include creep deflection, which can be significant in the long term.

For lateral load: Story height (H) divided by 500 for loads associated with a ten-year wind for interstory drift using the bare frame stiffness.

As always, these limits are intended to be reasonable limits in general. Coordination is required between the deflected structure and the non-structural components to ensure that the limits are appropriate for any particular project.

Chapter 6
Design Considerations Relative to Vibration/Acceleration

Human response to vibrations and accelerations and the reaction of machines to vibrations are also serviceability concerns. In general, human response to human or machine induced vibration takes a range from no concern, through moderate objection and concern for the building integrity, to physical sickness and rejection of the structure. In the case of machinery the function of the device can be impaired or destroyed. In general, the quality of the output of the machine is the standard of success or failure, whereas for human response the criteria are largely subjective.

Regarding the structural framework of a building, human response to vibrations can be limited to two categories: (1) frame behavior in response to wind forces or earthquake forces; and (2) floor vibration. In the opinion of the authors and other sources, frame behavior in response to lateral loads has not been a problem for low-rise multi-story buildings, which are generally stiff enough so that wind induced vibrations are not a problem. This is, of course, not the case for tall buildings. This topic is treated in the section on tall building acceleration induced by wind load.

Human Response to Vibration

Floor vibration and human response to it are of concern for all buildings. Currently, the state of the art treatise on this topic is AISC *Design Guide 11, Floor Vibrations Due to Human Activity by Murray*, Allen and Ungar, (AISC, 1997). It would be redundant for this Guide to address this topic and, thus, the reader is referred to *Design Guide 11* for a thorough explanation of the analysis and design considerations for floor vibration design.

Machines and Vibration

The behavior of machines in structures as it relates to vibration can be treated generally whether the machine is inducing the vibration or being acted upon by vibration induced by other sources. The effects of vibrations caused by machinery can be mitigated in the following ways:

1. The machine may be balanced or rebalanced.

2. The vibration source may be removed, relocated or restricted. For example, crane runways should not be attached to office areas in plants.

3. Damping in the form of passive or active devices may be added.

4. Isolation may be employed using soft springs or isolation pads.

5. The adjacent structure, floor, etc., may be tuned to a natural frequency substantially different from the critical frequency. For example, the floor or its components should have a frequency that is either less than one-half or greater than one and one-half times the fundamental frequency of the equipment.

Needless to say, the proper functioning of equipment is critical in any operation.

Thus, in the design of facilities such as labs, medical or computer areas and manufacturing plants, vibration control is essential and the active participation of the owner and equipment suppliers is required to set limits and provide performance data.

AISC *Design Guide 11* also provides a discussion on the design of floors for equipment that is sensitive to vibration.

Tall Building Acceleration—Motion Perception

Perception to building motion under the action of wind (Griffis, 1993) may be described by various physical quantities including maximum values of velocity, acceleration, and rate of change of acceleration, sometimes called jerk. Since wind-induced motion of tall buildings is composed of sinusoids having a nearly constant frequency (f) but varying phase, each quantity is related by the constant $2\pi f$ where f is the frequency of motion

$$V = (2\pi f)D$$

$$A = (2\pi f)^2 D$$

$$J = (2\pi f)^3 D$$

where D, V, A, and J are maximum displacement, velocity, acceleration, and jerk respectively.

Human response to motion in buildings is a complex phenomenon involving many psychological and physiological factors. It is believed that human beings are not directly sensitive to velocity if isolated from visual effects because, once in motion at any constant velocity, no forces operate upon the body to keep it in such motion. Acceleration, on the other hand, requires a force to act, which stimulates various body organs and senses. Some researchers believe that the human body can adapt to a constant force acting upon it whereas with changing acceleration (jerk) a continuously changing bodily adjustment is required. This changing acceleration may be an important component of motion perception in tall buildings. It appears that acceleration has become the standard for evaluation of motion perception in buildings because it is the best compromise of the various parameters. It also is readily measurable in the field with available equipment and has become a standard for comparison and establishment of motion perception guidelines among various researchers around the world.

Factors Affecting Human Response to Building Motion

Perception and tolerance thresholds of acceleration (Griffis, 1993; Kahn and Parmelee, 1971) as a measure of building motion are known to depend on various factors as described below. These factors have been determined from motion simulators that have attempted to model the action of buildings subjected to wind loads.

1. *Frequency or Period of Building*. Field tests have shown that perception and tolerance to acceleration tend to increase as the building period increases (frequency decreases) within the range of frequency commonly occurring in tall buildings.

2. *Age*. The sensitivity of humans to motion is an inverse function of age, with children being more sensitive than adults.

3. *Body Posture*. The sensitivity of humans to motion is proportional to the distance of the person's head from the floor; the higher the person's head, the greater the sensitivity. Thus, a person's perception increases as he goes from sitting on the floor, to sitting in a chair, to standing. However, since freedom of the head may be important to motion sensitivity, a person sitting in a chair may be more sensitive than a standing person because of the body hitting the back of the chair.

4. *Body Orientation*. Humans tend to be more sensitive to fore-and-aft motion than to side-to-side motion because the head can move more freely in the fore-and-aft direction.

5. *Expectancy of Motion*. Perception threshold decreases if a person has prior knowledge that motion will occur. Threshold acceleration for the case of no knowledge is approximately twice that for the case of prior knowledge.

6. *Body Movement*. Perception thresholds are higher for walking subjects than standing subjects, particularly if the subject has prior knowledge that the motion will occur. The perception threshold is more than twice as much between the walking and standing case if there is prior knowledge of the event, but only slightly greater if there is no knowledge of the event.

7. *Visual Cues*. Visual cues play an important part in confirming a person's perception to motion. The eyes can perceive the motion of objects in a building such as hanging lights, blinds, and furniture. People are also very sensitive to rotation of the building relative to fixed landmarks outside.

8. *Acoustic Cues*. Buildings make sounds as a result of swaying from rubbing of contact surfaces in frame joints, cladding, partitions, and other building elements. These sounds and the sound of the wind whistling outside or through the building are known to focus attention on building motion even before subjects are able to perceive the motion, and thus lower their perception threshold.

9. *Type of Motion*. Under the influence of dynamic wind loads, occupants of tall buildings can be subjected to translational acceleration in the x- and y-direction and torsional acceleration as a result of building oscillation in the along-wind, across-wind, and torsional directions, respectively. While all three components contribute to the response, angular motion appears to be more noticeable to occupants, probably caused by an increased awareness of the motion from the aforementioned visual cues. Also, torsional motions are often perceived by a visual-vestibular mechanism at motion thresholds, which are an order of magnitude smaller than those for lateral translatory motion (Kareem, 1985).

Root-Mean-Square (RMS) Acceleration Versus Peak Acceleration

A review of the literature on the subject of motion perception as measured by acceleration shows a difference in the presentation of the results. Some report root-mean-square or RMS accelerations and some researchers report maximum or peak acceleration. This dual definition has extended into establishing standards for motion perception.

Most of the research conducted on motion perception has been with motion simulators subjected to sinusoidal motion with varying frequency and amplitude. In these tests it has been common to report the results in maximum or peak acceleration since that was the quantity directly measured. It should be pointed out that for sinusoidal acceleration, the peak is equal to $\sqrt{2}$ times the RMS value. It appears that wind tunnel research has tended to report peak acceleration or both peak and RMS in order to correlate the wind tunnel studies with these motion simulation tests.

Many researchers believe that, when the vibration persists for an extended period of time (10 to 20 minutes) as is common with windstorms having a ten-year mean recurrence interval, RMS acceleration is a better indicator of objectionable motion in the minds of building occupants than isolated peak accelerations that may be dampened out within a few cycles (ISO, 1984; Hansen, Reed and Vanmarcke, 1973; Islam, Ellingwood and Corotis, 1990 and Tallin and Ellingwood, 1984). Also, the RMS statistic is easier to deal with during the process of temporal and spatial averaging because the 20-minute averaging period for a storm represents a time interval over which the mean velocity fluctuates very little.

The relationship between peak and RMS accelerations in tall buildings subjected to the dynamic action of wind loads has been defined by the peak factor, which varies with building frequency, but is oftentimes taken as 3.5. Correlation between peak and RMS accelerations in tall building motion may be made using this peak factor.

Relationship Between Building Drift and Motion Perception

Engineers designing tall buildings have long recognized the need for controlling annoying vibrations to protect the psychological well being of the occupants. Prior to the advent of wind tunnel studies this need was addressed using rule-of-thumb drift ratios of approximately 1/400 to 1/600 and code specified loads. Recent research (Islam, Ellingwood and Corotis, 1990), based on measurement of wind forces in the wind tunnel, has clearly shown that adherence to commonly accepted lateral drift criteria, per se, does not explicitly ensure satisfactory performance with regard to motion perception.

The results of one such study (Islam, Ellingwood and Corotis, 1990) are plotted in Figure 3 for two square buildings having height/width ratios of 6/1 and 8/1 where each is designed to varying drift ratios. Plots are shown of combined transitional and torsional acceleration as a function of design drift ratio. At drift ratios of 1/400 and 1/500 neither building conforms to acceptable standards for acceleration limits. The reason that drift ratios by themselves do not adequately control motion perception is because they only address stiffness and do not recognize the important contribution of mass and damping, which together with stiffness, are the predominant parameters affecting acceleration in tall buildings.

Human Response to Acceleration

Considerable research in the last 20 years has been conducted on the subject of determining perception threshold values for acceleration caused by building motion (Chen and Robertson, 1972; Khan and Parmelee, 1971 and ASCE, 1981). Much of this work has also attempted to formulate design guidelines for tolerance thresholds to be used in the design of tall and slender buildings.

Some of the earliest attempts to quantify the problem were performed by Chang (Chang, 1967 and Chang, 1972) who proposed peak acceleration limits for different comfort levels that were extrapolated from data in the aircraft industry. Chang's proposed limits are stated as follows:

Peak Acceleration	*Comfort Limit*
< 0.5% *g*	Not Perceptible
0.5% to 1.5% *g*	Threshold of Perceptibility
1.5% to 5.0% *g*	Annoying
5% to 15.0% *g*	Very Annoying
> 15% *g*	Intolerable

Design of Tall Buildings for Acceleration

The design of most tall buildings is controlled by lateral deflection (Griffis, 1993) and most often by perception to motion. Indeed, this characteristic is often proposed as one definition of a "tall" building.

While the problem of designing for motion perception in tall buildings is usually solved by conducting a scale model force-balance or aeroelastic test in the wind tunnel, certain criteria have been established to aid the designer. Empirical expressions now exist (Irwin, Ferraro and Stone, 1988; Islam, Ellingwood and Corotis, 1990 and National Research Council of Canada, 1990) that allow approximate evaluation of the susceptibility of a building to excessive motion. This can be very helpful in the early design stages particularly where geometry, site orientation, or floor plan is not yet fixed.

Generally, for most tall buildings without eccentric mass or stiffness, the across-wind response will predominate if $(WD)^{0.5}/H < 0.33$ where W and D are the across-wind and along-wind plan dimensions respectively and H is the building height.

In examining the across-wind proportionality, which often-times is the predominant response, it is possible to make the following deductions (Griffis, 1993):

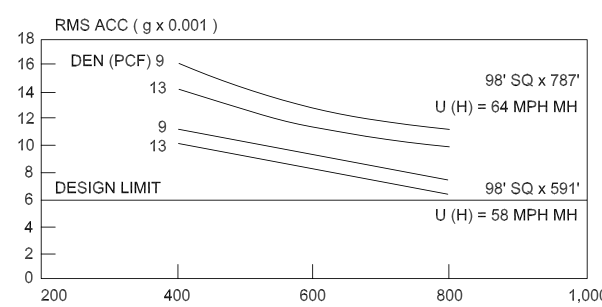


Fig. 3. RMS Accelerations vs. Drift Index

Table 2. (Griffis, 1993)
Traditional Motion Perception (Acceleration) Guidelines*
Ten-year Mean Recurrence Interval

Occupancy Type	Peak Acceleration (Milli-g)	Root-mean-square (RMS) Acceleration (Milli-g)		
		$1 \le T < 4$ $0.25 < f \le 1.0$ ($g_p \approx 4.0$)	$4 \le T < 10$ $0.1 < f \le 0.25$ ($g_p \approx 3.75$)	$T \ge 10$ $f \le 0.1$ ($g_p \approx 3.5$)
Commercial	15-27 Target 21	3.75-6.75 Target 5.25	4.00-7.20 Target 5.60	4.29-7.71 Target 6.00
Residential	10-20 Target 15	2.50-5.00 Target 3.75	2.67-5.33 Target 4.00	2.86-5.71 Target 4.29

Notation:
T = period (seconds), f = frequency (hertz), g_p = peak factor
1 milli-g = 0.001 × g = 0.0322 ft/sec^2
*RMS and peak accelerations listed in this table are the traditional "unofficial" standard applied in U.S. practice based on the author's experience.

1. If stiffness is added without a change in mass, acceleration will be reduced in proportion to $1/N^{1.54}$, which is proportional to $1/K^{0.77}$, where N is the frequency (hertz), K is the generalized stiffness $(2\pi N)^2 \times M$ and M is the generalized mass of the building

$$\sum_{i-1}^{n} m_i \phi_i^2$$

where m_i is mass of floor i and ϕ_i is modal coordinate at floor i, normalized so that $\phi = 1$ at $(Z) = H$, building height.

2. If mass is added throughout the building without changing the stiffness, acceleration will be reduced in proportion to $1/M^{0.23}$.

3. If mass is added with a proportionate increase in stiffness so that N does not change, then the acceleration will be reduced in proportion $1/M$ or $1/K$.

4. If additional damping is added, then the acceleration will be reduced in proportion to $1/\xi^{0.5}$, where ξ is the damping ratio.

It should be noted (Griffis, 1993) that torsional response can be important even for symmetrical buildings with uniform stiffness. This is because a torsional wind loading can occur from unbalance in the instantaneous pressure distribution on the building surface.

Oftentimes, in very slender buildings, it is not possible to obtain satisfactory performance, given building geometry and site constraints, by adding stiffness and/or mass alone. The options available to the engineer in such a case involve adding additional artificial damping and/or designing mass or pendulum dampers to counteract the sway (Grossman, 1990).

Numerous high-rise buildings have been designed and are performing successfully (Griffis, 1993) all over the world. Many have been designed according to an "unofficial" standard defined in Table 2. Both peak acceleration and RMS accelerations are used, their relationship generally defined by the use of a peak factor, g_p, equal to approximately 3.5-4.0. The true peak factor for a building, which relates the RMS loading or response to the peak, can be determined in a wind tunnel aeroelastic model study. Target peak accelerations of 21 milli-g's and 15 milli-g's are often used for commercial and residential buildings respectively (note: 1 milli-g = 0.001 × g = 0.0322 ft/sec^2). Corresponding RMS values are proportionally reduced accordingly using the appropriate peak factor. A stricter standard is often applied to residential buildings for the following reasons:

1. Residential buildings are occupied for more hours of the day and week and are therefore more likely to experience the design storm event.

2. People are less sensitive to motion when at work than when in the home at leisure.

3. People are more tolerant of their work environment than of their home environment.

4. Occupancy turnover rates are higher in office buildings than in residential buildings.

5. Office buildings are more easily evacuated in the event of a peak storm event.

The apparent shortcoming in the standard defined by Table 2 is the fact that the tolerance levels are not related to building frequency. Research has clearly shown a relationship between acceptable acceleration levels and building frequency. Generally higher acceleration levels can be tolerated for lower frequencies.

The International Organization for Standardization has established a design standard for occupant comfort in fixed structures subjected to low frequency horizontal motion—ISO Standard 6897-1984 (ISO, 1984).

Chapter 7
Design Considerations Relative to Equipment

The assortment of equipment used in buildings is many and varied. This discussion will be limited to equipment that is a permanent part of the building and will cover elevators, conveyors, cranes and mechanical equipment.

Elevators

Elevators are of two types: hydraulic and traction. Hydraulic elevators are moved by a piston, which is generally embedded in the earth below the elevator pit. Traction elevators are moved by a system of motors, sheaves, cables and counterweights. In both types the cars are kept in alignment by tee-shaped tracks, which run the height of the elevator shafts. Such tracks are also used to guide the counterweight in traction elevators.

Elevators impose few limits on deflection other than those previously mentioned in the sections on cladding and partitions. A building drift limit of height (H) divided by 500 calculated on the primary structure using ten-year winds will provide adequate shaft alignment for low-rise buildings. In addition to this static deflection limit, proper elevator performance requires consideration of building dynamic behavior. Design of elevator systems (guide rails, cables, sheaves) will require knowledge of predominant building frequencies and amplitude of dynamic motion. This information should be furnished on the drawings or in the specifications (Griffis, 1993). The vertical deflection limits given for floors in the partition section will provide adequate control on the vertical location of sills. The only extraordinary requirement is found in ANSI/ASME A17.1 (ANSI, 2002) rule 105.5 which states, "The allowable deflections of machinery and sheave beams and their immediate supports under static load shall not exceed 1/1666 of the span." The term "static load" refers to the accumulated live and dead loads tributary to the beams in question including the unfactored elevator loads.

Although it would not be required by a strict reading of rule 105.5 the authors recommend that the span divided by 1666 limit be used for the girders (if any) that support the beams. However, the limit need not be applied to the accumulated deflection in the bay.

Conveyors

It is very difficult to give clear-cut serviceability guidelines relative to the performance of conveyors due to their diverse configurations and the diverse nature of the materials conveyed. However, the following comments are offered.

The key to conveyor performance is the maintenance of its geometry, especially in the area of switches and transfer points. In general, the construction of conveying equipment is flexible enough to absorb some distortion due to differential deflection of support points. Thus, the deflection limits recommended in the sections on roofs and floors would be appropriate for the design of roofs and floors supporting conveyors, i.e., span divided by 150 to 240 for roofs and span divided by 360 for floors for live load including conveyor load. As always, the conveyor supplier must account for support point deflection where sections are supported with an indeterminate arrangement of supports.

There are three areas of special concern. First, because conveyors are rarely attached to all roof members, there may be cases where the differential deflection places unexpected loads on the deck and deck fasteners, which may result in localized distress. Secondly, heavy conveyor loads may cause stress reversals in light and lightly loaded roofs, which must be accounted for in this design. Third, conveyors can also cause local member distortions when they are not properly connected to the framing. Because of the potential for interface problems, it is essential that the conveyor supplier's criteria be discussed and incorporated into the design.

Cranes

There are two categories of movement related to the operation of cranes. First, there are those building movements induced by the crane operation that affect the performance of the building. The limits given in the previous sections are appropriate for the control of building movements induced by crane operation. The second category of movements includes those induced by other loads (perhaps in combination with crane forces), which affect the performance of the cranes themselves. This second area will be covered here. Three types of cranes will be discussed: pendant-operated traveling cranes, cab-operated traveling cranes, and jib cranes. The reader should refer to ASCE 7-02 for its treatment of crane loads (Section 4.10) as a special case of live loads.

Pendant-Operated Cranes

For pendant-operated cranes, there are less strict requirements. The controls related to other aspects of building performance will suffice. However, it should be noted that in the authors' experience, buildings designed with a limit on drift of height divided by 100 can exhibit observable movements during crane operation and it is recommended that this be reviewed with the building owner at the design stage.

Cab-Operated Cranes

A drift limit for cab-operated cranes is required so that the operators will perceive the system as safe. The limit for drift in the direction perpendicular to the runways is suggested to be height divided by 240, with a maximum of 2 in. (Fisher, 2004). This displacement is to be measured at the elevation of the runways. The appropriate loading is either the crane lateral force or the lateral loads associated with a ten-year wind on the bare frame. The crane lateral loads are those specified by the AISC *Specification* (which refers to ASCE 7-02) or the AISE *Specification*, as appropriate.

The longitudinal displacement of crane runways is rarely of concern when hot-rolled shapes are used in the X-bracing. If, on the other hand, rigid frames are used, column bending may result in an excessively limber structure. In the absence of any standard for this movement, the authors propose using the same limits as those proposed above for movements perpendicular to the runways. The loadings for this condition are the crane tractive forces and bumper forces. Consideration must also be given to account for the longitudinal movement that results from column torsional rotations at bracketed runway supports.

Another category of movements includes those that affect the lateral alignment of the runways. First, the lateral deflections of the runways themselves should be limited to span divided by 400 to avoid objectionable visual lateral movements. The loading in this case is the lateral crane force (CMAA, 1999). Secondly, the relative lateral movements of support points must be controlled to prevent the runways from moving apart or together. This will prevent the wheels from either jumping the rails or alternatively having the flanges bind against the rails.

Loads affecting the inward or outward movement of support points are those applied to the structure after the alignment of the rails. Inward movement is by and large the result of crane loads, whereas outward movements are caused by roof loads, chiefly high snow loads.

The allowable amount of inward movement is controlled by the arrangement and proportions of the wheel flange spacing and should be coordinated with the crane supplier. The control of inward movements can be on the order of ½ in. total.

Outward column deflections at crane runway elevations should be limited so that the total spread of the runways will not exceed 1 in. The appropriate loading is snow load. It is suggested that the roof snow load be taken as zero in areas where the 50-year snow is 13 psf or less. When evaluating this differential movement, 50 percent of the roof snow load should be taken in areas where the roof snow load is between 13 psf and 31 psf and three quarters of the roof snow should be used where the roof snow load is greater than 31 psf .

Jib Cranes

Jib cranes are usually attached to building columns. The principal concern as it relates to deflections is that the drop of the outboard end of the jib cannot be so much as to prevent the trolley from moving back towards the column with reasonable effort. The limit of the drop of the outboard end should be a maximum of the jib boom length divided by 225. This movement at the end of the jib is the summation of the cantilever behavior of the jib itself plus the bending of the column due to the jib reaction. This second component may be significant when the jib is rotated so that it applies its loads to the weak axis of the column.

Crane Runways

Crane runways must also be controlled for vertical deflections. Such deflections are usually calculated without an increase for vertical impact. The deflection limits for various crane types and classes are given below.

Top running cranes:

CMAA Classes A, B and C:	Span divided by 600 (CMAA, 1999)
CMAA Class D:	Span divided by 800
CMAA Classes E and F:	Span divided by 1000 (AISE, 2003)

Underhung and monorail cranes:

CMAA Classes A, B and C:	Span divided by 450

Note: Underhung cranes with more severe duty cycles must be designed with extreme caution and are not recommended.

AISE *Technical Report No. 13* (AISE, 2003) also provides deflection limits for crane runway girders. They are established based on the Class of Mill Building, A through D as follows:

Class A Buildings	Span divided by 1000
Class B Buildings	Span divided by 1000
Class C Buildings	Span divided by 600
Class D Buildings	Span divided by 600

Mechanical Equipment

Mechanical equipment for buildings generally consists of piping, ductwork, exhaust hoods, coils, compressors, pumps, fans, condensers, tanks, transformers, switchgear, etc. This equipment can be dispersed throughout the building, collected into mechanical rooms and/or located on the roof in the form of pre-engineered package units. The key feature of this equipment is that it represents real loads as opposed to code specified uniform loads, which may or may not ever exist in their full intensity. Because of this, a degree of extra attention should be applied to the control of deflections as they relate to mechanical equipment.

This is especially true where the mechanical equipment loads are the predominant part of the total loads on a given structure. Special Attention should be given to the tilting and racking of equipment, which, if excessive, could impair the function of the equipment and to differential deflection, which could deform or break interconnecting piping or conduits. While the actual deflection limits on each project should be carefully reviewed with the mechanical engineers and equipment suppliers, it can be stated that buildings designed to the standard span divided by 150 to 240 for roofs and span divided by 360 for live loads have generally performed well.

References

Allen, D.E., and J.H. Rainer, *Floor Vibration*, Canadian Building Digest–Division of Building Research, National Research Council Canada, September 1975.

Aluminum Curtain Wall Design Manual, American Architectural Manufacturers Association, Schaumburg, Illinois, 1996.

Approval Guide, 2000—Equipment, Materials, Service for Conservation of Property, Factory Mutual Research Corporation, an FM Global Affiliate, 2000.

Architectural Precast Concrete, Prestressed Concrete Institute, Chicago, 1973.

Bergmann, Roland, "Structural Serviceability Aspects of Building Envelopes in Tall Buildings," *Proceedings*, Symposium/Workshop on Serviceability of Buildings, National Research Council Canada, May 1988.

Building Code Requirements for Concrete Masonry Structures and Commentary (ACI 530), American Concrete Institute, 2002.

Building Code Requirements for Reinforced Concrete (ACI 318-02), American Concrete Institute, 2002.

Carter, Charles J. and Jiahong Zuo, "Ponding Calculations in LRFD and ASD," AISC *Engineering Journal*, Third Quarter, 1999.

Cermak, J., Boggs, D., Paper to be published.

Chang, F., "Wind and Movement in Tall Buildings," *Civil Engineering* Magazine, ASCE, August 1967.

Chang, F.K., "Human Response to Motions in Tall Buildings," Paper presented at ASCE National Environmental Engineering Meeting, Houston, Texas, October 16-22,1974.

Chen, P.W. and L.E. Robertson, "Human Perception Thresholds of Horizontal Motion," *Journal of Structural Engineering*, ASCE, Vol. 98, No. ST8, August 1972.

Churches, C.H., Emile W.J. Troup, and Carl Angeloff, *Open-Deck, Steel-Framed Parking Structures*. AISC *Design Guide 18*. American Institute of Steel Construction, Chicago, 2003.

Code of Standard Practice for Steel Bridges and Buildings, American Institute of Steel Construction, 2000.

Safety Code for Elevators and Escalators, A17.1, American National Standards Institute, 2002.

Ellingwood, Bruce, and Andrew Tallin, "Structural Serviceability: Floor Vibrations," *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 110, No. 2, February 1984.

Ellingwood, Bruce, "Serviceability Guidelines for Steel Structures," *Proceedings*, 1988 National Steel Construction Conference, American Institute of Steel Construction, 1988.

Expansion Joints in Buildings, Federal Construction Council, Technical Report No. 65, National Academy of Sciences-National Research Council, Washington, D.C.

Fintel, Mark, S. K. Ghosh and Hal Iyengar, *Column Shortening in Tall Structures—Prediction and Compensation*, Skokie, Illinois, 1987.

Fisher, James M. and Donald R. Buettner, *Light and Heavy Industrial Buildings*, American Institute of Steel Construction, September 1979.

Fisher, James M., "Industrial Buildings: Guidelines and Criteria," *Engineering Journal*, American Institute of Steel Construction, Vol. 21, No. 3, 1984.

Fisher, James M., *Industrial Buildings—Roofs to Column Anchorage*, AISC *Design Guide 7*, American Institute of Steel Construction, Chicago, 2004.

Freeman, S. A., "Racking Tests of High Rise Partitions," *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 103, No. 8, August 1977.

Galambos, T. V., P. L. Gould, M. K. Ravindra, H. Suryoutomo, and R. A. Crist, *Structural Deflections: A Literature and State of the Art Survey*, Washington, D.C.: U.S. Department of Commerce, National Bureau of Standards, 1973.

Galambos, Theodore V. and Bruce Ellingwood, "Serviceability Limit States: Deflection," *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 112, No. 1, January 1986.

Griffis, L. G., "Serviceability Limit States under Wind Load," *Engineering Journal*, American Institute of Steel Construction, 1993.

Grossman, J.S., "Slender Concrete Structures—The New Edge," ACI *Structural Journal*, Volume 87, No. 1, January/February 1990.

Guide for Concrete Floor and Slab Construction, ACI 302.1, reported by ACI Committee 302, American Concrete Institute, 1997.

Guide for the Design and Construction of Mill Buildings, AISE Technical Report No. 13, Association of Iron and Steel Engineers, August 2003.

Guidelines for the Evaluation of the Response of Occupants of Fixed Structures to Low Frequency Horizontal Motion (0.063 to 1 Hz.), ISO Standard 6897-1984. International Organization of Standardization, 1984.

Gypsum Construction Handbook, Centennial Edition, United States Gypsum Company, 2000.

Hansen, R.J., J.W. Reed and E.H. Vanmarcke, "Human Response to Wind-Induced Motion of Buildings," *Journal of Structural Engineering*, ASCE, Volume 99, No. ST7, July 1973.

Installation of Aluminum Curtain Walls, American Architectural Manufacturers Association, Schaumburg, Illinois, 1989.

Irwin, P.A., V. Ferraro and G.K. Stone, "Wind-Induced Motions of Buildings," *Proceedings* Symposium/Workshop on Serviceability of Buildings (Movements, Deformations, Vibrations) Volume I. University of Ottawa, May 16-18, 1988.

Islam, M.S., B. Ellingwood and R.B. Corotis, "Dynamic Response of Tall Buildings to Stochastic Wind Load," *Journal of Structural Engineering*, ASCE, Volume 116, No. 11, November 1990.

Kareem, A., "Lateral Torsional Motion of Tall Buildings to Wind Loads," *Journal of Structural Engineering*, ASCE, Volume 111, No. 11, November 1985.

Khan, F. and R. Parmelee, "Service Criteria for Tall Buildings for Wind Loadings," *Proceedings* 3rd International Conference on Wind Effects on Buildings and Structures, Tokyo, Japan, 1971.

Lenzen, Kenneth H., "Vibration of Steel Joist-Concrete Slab Floors," *Engineering Journal*, American Institute of Steel Construction, July 1966.

Load and Resistance Factor Design Specification for Structural Steel Buildings and Commentary, AISC 1999, with 2001 errata.

Loss Prevention Data for Roofing Contractors, Factory Mutual Research Corp., Factory Mutual System, various dates.

Low-Rise Building Systems Manual, Metal Building Manufacturers Association, 2002.

Meriam, J. L., *Statics and Dynamics*, John Wiley and Sons, Inc., New York, 1967.

Minimum Design Loads for Building and Other Structures, ANSI A58.1, American National Standards Institute, New York, 1982.

Minimum Design Loads for Building and Other Structures, ASCE 7-02, American Society of Civil Engineers, Reston, Virginia, 2002.

Murray, Thomas M. and William E. Hendrick, "Floor Vibrations and Cantilevered Construction," *Engineering Journal*, American Institute of Steel Construction, Vol. 14, No. 3, 1977.

Murray, Thomas M., David E. Allen, Eric E. Ungar, *Floor Vibrations—Due to Human Activity*, AISC *Design Guide 11*, American Institute of Steel Construction, Chicago, 1997.

Murray, Thomas M., "Design to Prevent Floor Vibrations," *Engineering Journal*, American Institute of Steel Construction, Vol. 12, No. 3, 1975.

N.R.C.A. *Roofing and Waterproofing Manual*, The; Fifth edition, National Roofing Contractors Association, Chicago, 2001.

Nemestothy, E. and G. Visnovitz, "Experimental Analysis of the Deformability of Partition Walls and Brittle Coverings," *Proceedings*, Symposium Workshop on Serviceability of Buildings, National Research Council Canada, May 1988.

North American Specification for Design of Cold-Formed Steel Structural Members, American Iron and Steel Institute, 2001.

PCI Design Handbook: Precast and Prestressed, 5th Edition, Prestressed Concrete Institute, Chicago, 1999.

Planning and Environmental Criteria for Tall Buildings, A Monograph on Planning and Design of Tall Buildings, ASCE, Volume PC, Chapter PC-13, 1981.

Rainer, J. H., "Vibrations in Buildings," *Canadian Building Digest*, Division of Buildings Research, National Re search Council Canada, May 1984.

Recommended Practice for Engineered Brick Masonry, Brick Institute of America, 1978.

Ritchie, J. K. and E. Y. L. Chien, *Composite Floor Systems, Constructional Steel Design—An International Guide*, Edited by Dowling, Harding and Bjorhovde, Elsevier Applied Science, London and New York, 1992.

Ruddy, John L., "Composite Beam Deflection Criteria," *Proceedings* of National Steel Construction Conference, American Institute of Steel Construction, 1996.

Ruddy, John L., "Ponding of Concrete Deck Floors," *Engineering Journal*, American Institute of Steel Construction, Vol. 23, No. 3, 1986.

Sabnis, Gajanan and J. K. *Sridhar, Fundamentals and Overview of Composite Action in Structures, Handbook of Composite Construction Engineering*, Van Nostrand Reinhold, 1979.

Salmon, Charles G. and James M. Fisher, *Composite Steel Concrete Construction, Handbook of Composite Construction Engineering*, Van Nostrand Reinhold, 1979.

Simiu, E., M. J. Changery and J. J. Filliben, *Extreme Wind Speeds at 129 Stations in the Contiguous United States*, U.S. Department of Commerce, National Bureau of Standards, 1979.

Specification for Structural Joints Using ASTM A325 or A490 Bolts, Research Council on Structural Connections, June 23, 2000.

Specification for the Design of Cold-Formed Stainless Steel Structural Members (ASCE 8-SSD-LRFD/ASD), American Society of Civil Engineers, 2002.

Specification for the Design and Construction of Composite Slabs and Commentary on *Specifications for the Design and Construction of Composite Slabs*, American Society of Civil Engineers, 1991.

Specification for the Design, Fabrication and Erection of Structural Steel Buildings with Commentary, American Institute of Steel Construction, 1989, with Supplement No. 1.

Specification for the Design, Fabrication and Erection of Structural Steel for Buildings with Commentary, American Institute of Steel Construction, 1978.

Specification for Structural Steel Buildings, Load and Resistance Factor Design, American Institute of Steel Construction, 1999.

Specification for Steel Hollow Structural Sections, Load and Resistance Factor Design, American Institute of Steel Construction, 2000.

Specifications and Commentary for Composite Steel Floor Decks, Steel Deck Institute, 2001.

Specifications and Commentary for Non-composite Steel Form Deck, Steel Deck Institute, 2001.

Specifications and Commentary for Steel Roof Deck, Steel Deck Institute, 2001.

Specifications for Electric Overhead Traveling Cranes, Crane Manufacturers Association of America, 1999.

Specifications for Structural Concrete for Buildings (ACI 301-96) American Concrete Institute, 1996.

Standard for Cold-Formed Steel Framing—General Provisions, American Iron and Steel Institute, 2001.

Standard for Cold-Formed Steel Framing—Truss Design, American Iron and Steel Institute, 2001.

Standard for the Structural Design of Composite Slabs (ASCE 3), American Society of Civil Engineers, 1991.

Standard Specifications for Joist Girders, Steel Joist Institute, 2002.

Standard Specifications for Long-Span Steel Joists, LH-series and Deep Long-Span Steel Joists, DLH-series, Steel Joist Institute, 2002.

Standard Specifications for Open Web Steel Joists, K-series, Steel Joist Institute, 2002.

Standard Tolerances for Concrete Construction and Materials (ACI 117-90), American Concrete Institute, 1990.

Steel Structures for Buildings—Limit States Design, CAN3-S16.1-M78, Canadian Standards Association, reprinted, Handbook of Steel Construction, Canadian Institute of Steel Construction, 3rd Edition, December 1980.

Steffens, R. J., *Structural Vibration and Damages*, Department of the Environment, Building Research Establishment, London, 1974.

Structural Design Guidelines for Aluminum Framed Skylights, American Architectural Manufacturers Association, Schaumburg, Illinois, 1989.

Structural Design of Tall Steel Buildings, A Monograph on Planning and Design of Tall Buildings, ASCE, Volume SB, Chapter SB-5, 1979.

Structural Serviceability: A Critical Appraisal and Research Needs, Ad Hoc Committee on Serviceability research, Committee on Research of the Structural Division, Journal of Structural Engineering, American Society of Civil Engineers, Reston, Virginia, December, 1986.

Supplement No.1 to the Specification for Structural Steel-Buildings, Allowable Stress Design and Plastic Design, AISC 335-89s1, American Institute of Steel Construction, 2001.

Supplement to the National Building Code of Canada–1990, National Research Council of Canada, Ottawa, 1990.

Suprenant, Bruce A., *Construction of Elevated Concrete Slabs—Understanding the Effect of Structural Systems*, Publication #900910, The Aberdeen Group, 1990.

Tallin, Andrew and Bruce Ellingwood, "Serviceability Limit States: Wind Induced Vibrations," *Journal of Structural Engineering*, American Society of Civil Engineering, Vol. 110, No. 10, 1984.

Technical Notes on Brick Construction, Brick Institute of America: Nos. 18, 18A and l8B Differential Movement, 1991; Nos. 28 and 28B: Brick Veneer, 1991 and 1987; No. 31B: Structural Steel Lintels, 1987.

Tipping, Eldon and Bruce Suprenant, Construction of Elevated Concrete Slabs—Measuring and Evaluation Quality, Publication #910260, The Aberdeen Group, 1991.

Tipping, Eldon and Suprenant, Bruce, *Construction of Elevated Concrete Slabs—Practices and Procedures*, Publication #910032, The Aberdeen Group, 1991.

Tipping, Eldon, G., Jr., "Achieving Level Composite Steel Floors," *Proceedings*, Structures Congress, American Society of Civil Engineers, 1993.

"Vibration of Steel Joist Floors," SJI *Technical Digest, No. 5*, Steel Joist Institute, 1988.

"Wind Drift of Steel-Framed Buildings: State-of-the-Art Report," ASCE Task Force on Drift Control of Steel Building Structures of the Committee on Design of Steel Building Structures, *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 114, No. 9, September 1988.

Appendix
Summary of Serviceability Considerations

The criteria presented in each of the previous chapters are summarized in the tables that follow. Because of the limitations of this format and the consequent abbreviation, the reader is cautioned against using the summary tables without reference to the full discussion in the text.

SERVICEABILITY CONSIDERATIONS

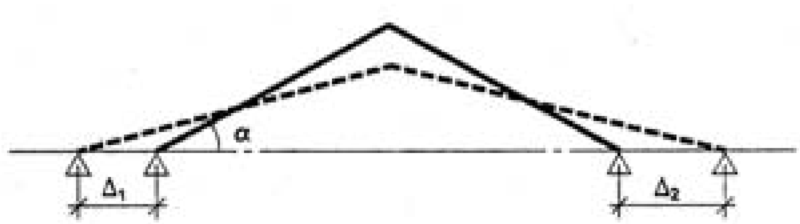
ROOFING

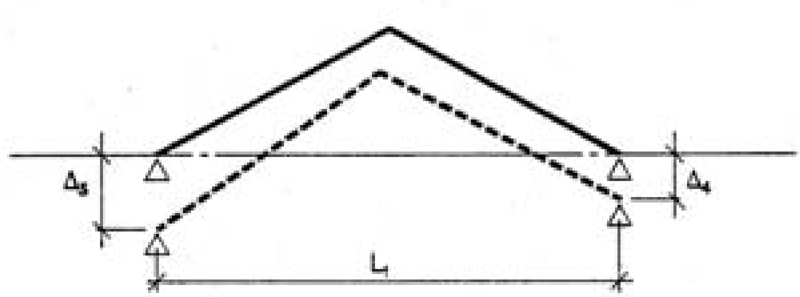
ROOFING TYPE	*STRUCTURAL ELEMENT*	*DEFORMATION*	*RECOMMEN-DATION*	*LOADING*
MEMBRANE ROOFS	ROOFING EXPAN-SION JOINTS	HORIZONTAL MOVEMENT	150' TO 200' MAXIMUM	THERMAL
	METAL DECK (TWO SPAN)	VERTICAL DEFLECTION	*L* / 200 MAXIMUM	300-LB LOAD
	METAL DECK	VERTICAL DEFLECTION	*L* / 240 MAXIMUM	*LL*
	METAL DECK	VERTICAL DEFLECTION	*L* / 240 MAXIMUM	200-LB LOAD
	METAL DECK	-	PER SDI TABLE	MAINTENANCE & CONSTRUCTION
	PURLINS	VERTICAL DEFLECTION	PURLIN DEPTH ? $(F_y/1000) \times$ SPAN	-
	STEEL JOISTS	VERTICAL DEFLECTION	*L* / 240 MAXIMUM	*LL*
	JOIST GIRDERS	VERTICAL DEFLECTION	*L* / 240 MAXIMUM	*LL*
	ROOF DECKS	VERTICAL DEFLECTION.	*L* / 240 MAXIMUM	*DL* + *DL*
	ROOFS	SLOPE	1 / 4 IN. PER FOOT MINIMUM	DRAINAGE
METAL ROOFS THROUGH FASTENER TYPE	EXPANSION JOINTS	HORIZONTAL MOVEMENT	100' TO 200' MAXIMUM	THERMAL
	ROOF	SLOPE	1 / 2 IN. PER FOOT MINIMUM	DRAINAGE
	PURLIN	VERTICAL DEFLECTION	*L* / 240 MAXIMUM	SNOW LOAD
	PURLIN	VERTICAL DEFLECTION	POSITIVE DRAINAGE	$DL + 0.5 \times S$ DL + 5 PSF
METAL ROOFS STANDING SEAM	EXPANSION JOINTS	HORIZONTAL MOVEMENT	150' TO 200' MAXIMUM	THERMAL
	ROOF	SLOPE	1 / 4 IN. PER FOOT MINIMUM	DRAINAGE
	PURLIN	VERTICAL DEFLECTION	*L* / 150 MAXIMUM	SNOW LOAD
	PURLIN	VERTICAL DEFLECTION.	POSITIVE DRAINAGE	$DL + 0.5 \times S$ DL + 5 PSF

SERVICEABILITY CONSIDERATIONS

SKYLIGHT SUPPORTS

DEFORMATION	*RECOMMEN-DATION*	*LOADING*
SKYLIGHT FRAME RACKING	1 / 4 IN. GASKETED MULLIONS	*DL + LL*
SKYLIGHT FRAME RACKING	1 / 8 IN. FLUSH GLAZING	*DL + LL*
DEFLECTION NORMAL TO GLAZING	*L* / 300 ≤ 1 IN. MAXIMUM	*DL + LL*
Ä$_1$ + Ä$_2$	± 1 / 8 IN. α ? 25 DEGREES	*DL + LL*
Ä$_1$ + Ä$_2$	± 5 / 16 IN. 25 < α < 45 DEG.	*DL + LL*
Ä$_1$ + Ä$_2$	± 1 / 2 IN. α ≥ 45 DEGREES	*DL + LL*
Ä$_3$ - Ä$_4$	*L$_1$* / 240 ≤ 1 / 2 IN. MAXIMUM	*DL + LL*





SERVICEABILITY CONSIDERATIONS

CLADDING

CLADDING SUPPORT TYPE	*STRUCTURAL ELEMENT*	*DEFORMATION*	*RECOMMEN-DATION*	*LOADING*
FOUNDATION	METAL PANELS / BARE FRAME	DRIFT PERPENDICULAR TO WALL	*H* / 60 TO *H*/ 100 MAXIMUM	10 YEAR WIND
	METAL PANELS / GIRTS	HORIZONTAL DEFLECTION	*L* / 120 MAXIMUM	10 YEAR WIND
	METAL PANELS / WIND COLUMNS	HORIZONTAL DEFLECTION	*L* / 120 MAXIMUM	10 YEAR WIND
	PRECAST WALLS / BARE FRAME	DRIFT PERPENDICULAR TO WALL	*H* / 100 MAXIMUM	10 YEAR WIND
	UNRIENFORCED MASONRY WALLS / BARE FRAME	DRIFT PERPENDICULAR TO WALL	1 / 16 IN. CRACK BASE OF WALL	10 YEAR WIND
	RIENFORCED MASONRY WALLS / BARE FRAME	DRIFT PERPENDICULAR TO WALL	*H* / 200 MAXIMUM	10 YEAR WIND
	MASONRY WALLS / GIRTS	HORIZONTAL DEFLECTION	*L* / 240 ≤ 1.5 IN. MAXIMUM	10 YEAR WIND
	MASONRY WALLS / WIND COLUMNS	HORIZONTAL DEFLECTION	*L* / 240 ≤ 1.5 IN. MAXIMUM	10 YEAR WIND
	MASONRY WALLS / LINTEL	VERTICAL DEFLECTION	*L* / 600 ≤ 0.3 IN. MAXIMUM	*DL* + *LL*
	MASONRY WALLS / LINTEL	ROTATION	≤ 1 DEGREE MAXIMUM	*DL* + *LL*
COLUMN	PRE-ASSEMBLED UNITS / COLUMNS	RELATIVE SHORTENING	1 / 4 IN. MAXIMUM	0.5 × *LL*
	PRE-ASSEMBLED UNITS / BARE FRAME	RACKING	*H* / 500	10 YEAR WIND
SPANDREL	CURTAIN WALLS / BARE FRAME	RACKING	*H* / 500	10 YEAR WIND
	CURTAIN WALLS / SPANDRELS	VERTICAL DEFLECTION	3 / 8 IN. MAXIMUM	*DL* PRIOR TO CLADDING
	CURTAIN WALLS / SPANDRELS	VERTICAL DEFLECTION	*L* / 480 ≤ 5 / 8 IN. MAXIMUM	TOTAL *DL*
	CURTAIN WALLS / SPANDRELS	VERTICAL DEFLECTION	L / 360 ≤ 1 / 4 - 1 / 2 IN. MAXIMUM	0.5 × *LL*
	CURTAIN WALLS / SPANDRELS	VERTICAL DEFLECTION	*L* / 600 ≤ 3 / 8 IN. MAXIMUM	*DL* INCL. CLADDING WEIGHT

SERVICEABILITY CONSIDERATIONS

CEILINGS AND PARTITIONS

FINISH TYPE	*STRUCTURAL ELEMENT*	*DEFORMATION*	*RECOMMEN-DATION*	*LOADING*
PLASTERED CEILING	ROOF MEMBER	VERTICAL DEFLECTION	L / 360 MAXIMUM	$0.5 \times LL$ OR 50 YEAR SNOW
NON-PLASTERED CEILING	ROOF MEMBER	VERTICAL DEFLECTION	L / 240 MAXIMUM	$0.5 \times LL$ OR 50 YEAR SNOW
	FLOOR BEAM / GIRDER	VERTICAL DEFLECTION	L / 360 ≤ 1 IN. MAXIMUM	DL
PARTITION	FRAME	HORIZONTAL MOVEMENT	H / 500 MAXIMUM	10 YEAR WIND
	ROOF MEMBER	VERTICAL DEFLECTION	3 / 8 IN. TO 1 IN. MAXIMUM	$0.5 \times LL$ OR 50 YEAR SNOW
	FLOOR BEAM / GIRDER	VERTICAL DEFLECTION	L / 360 ≤ 3 / 8 IN. TO 1 IN., MAXIMUM	$0.5 \times LL$

SERVICEABILITY CONSIDERATIONS

EQUIPMENT

EQUIPMENT TYPE	*STRUCTURAL ELEMENT*	*DEFORMATION*	*RECOMMEN-DATION*	*LOADING*
TOP RUNNING CRANES	RUNWAY SUPPORTS	TOTAL INWARD MOVEMENT	1 /2 IN. MAXIMUM	*LL* OR 50 YEAR SNOW
	RUNWAY SUPPORTS	TOAL OUTWARD MOVEMENT	1 IN. MAXIMUM	SNOW
	RUNWAY BEAM	HORIZONTAL DEFECTION	*L* / 400 MAXIMUM	CRANE LATERAL
	RUNWAY BEAM CMAA ‘A’, ‘B’ & ‘C’	VERTICAL DEFLECTION	*L* / 600 MAXIMUM	CRANE LATERAL STATIC LOAD
	RUNWAY BEAM CMAA ‘D’	VERTICAL DEFLECTION	*L* / 800 MAXIMUM	CRANE LATERAL STATIC LOAD
	RUNWAY BEAM CMAA ‘E’ & ‘F’	VERTICAL DEFLECTION	*L* / 1000 MAXIMUM	CRANE LATERAL STATIC LOAD
TOP RUNNING CAB OPERATED	BARE FRAME	DRIFT AT RUNWAY ELEVATION	*H* / 100 ≤ 1-IN. MAXIMUM	CRANE LATERAL OR 10 YR. WIND
TOP RUNNING PENDANT OPERATED	BARE FRAME	DRIFT AT RUNWAY ELEVATION	*H* / 240 ≤ 1-IN. MAXIMUM	CRANE LATERAL OR 10 YR. WIND
UNDERHUNG CRANE	RUNWAY BEAM CMAA ‘A’, ‘B’ & ‘C’	VERTICAL DEFLECTION	*L* / 450 MAXIMUM	CRANE VERTICAL
JIB CRANE	BOOM	VERTICAL DEFLECTION	*H* / 225 MAXIMUM	CRANE VERTICAL
ELEVATORS	BARE FRAME	DRIFT	*H* / 500 MAXIMUM	10 YEAR WIND
	MACHINE / SHEAVE BEAMS	VERTICAL DEFLECTION	*L* / 1666 MAXIMUM	*DL + LL*
	MACHINE / SHEAVE BEAMS SUPPORTS	VERTICAL DEFLECTION	*H* / 1666 MAXIMUM	*DL + LL*