Serviceability Limit States Under Wind Load

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INTRODUCTION

The increasing use and reliance on probability based limit states design methods, such as the recently adopted AISC LRFD Specification,[1] has focused new attention on the problems of serviceability in steel buildings. These methods, along with the development of higher-strength steels and concretes and the use of lighter and less rigid building materials, have led to more flexible and lightly damped structures than ever before, making serviceability problems more prevalent.

The purpose of this paper is to focus attention on two important serviceability limit states under wind loads; namely, *deformation* (including deflection, curvature, and drift) and *motion perception* (acceleration). These issues are particularly important for tall and/or slender steel and composite structures. A brief review of available information on these subjects will be presented followed by a discussion of current standards of practice, particularly in the United States. Finally, proposed standards will be presented that, hopefully, will focus attention, debate, and perhaps new research efforts on these very important issues in design.

IMPORTANCE OF SERVICEABILITY LIMIT STATES [12,31]

Every building or other structure must satisfy *a strength limit state,* in which each member is proportioned to carry the design loads to resist buckling, yielding, instability, fracture, etc.; and *serviceability limit states* which define functional performance and behavior under load and include such items as deflection, vibration, and corrosion. In the United States, strength limit states have traditionally been specified in building codes because they control the safety of the structure. Serviceability limit states, on the other hand, are usually noncatastrophic, define a level of quality of the structure or element, and are a matter of judgment as to their application. Serviceability limit states involve the perceptions and expectations of the owner or user and are a contractual matter between the owner or user and the designer and builder. It is for these reasons, and because the benefits themselves are often subjective and difficult to define or quantify, that serviceability limit states for the most part are not included within U.S. building codes. The fact that serviceability limit states are usually not codified should not diminish their importance. Exceeding a serviceability limit state in a building or other structure usually means that its function is disrupted or impaired because of local minor damage, deteriorations, or because of occupant discomfort or annoyance. While safety is *usually* not at issue, the economic consequences can be substantial. Interestingly, there *are* some serviceability items that can also be safety related. For instance, excessive building drift can influence frame stability because of the P-Δ effect. Excessive building drift can also cause portions of the building cladding to fall and potentially injure pedestrians below.

Serviceability limit states can be grouped into three categories as follows:

1. *Deformation* (deflection, curvature, drift). Common examples include local damage to nonstructural elements (e.g., ceilings, cladding, partitions) due to deflections under dead, live, wind, or seismic load; and damage from temperature change, moisture, shrinkage, or creep.
2. *Motion perception* (vibration, acceleration). Common examples include human discomfort caused by wind or machinery, particularly if resonance occurs. Floor vibrations from people or machinery and acceleration in tall buildings under wind load are usual areas of concern in this category.
3. *Deterioration.* Included are such items as corrosion, weathering, efflorescence, discoloration, rotting, and fatigue.

The focus on this paper will be items one and two.

CURRENT TREATMENT OF SERVICEABILITY ISSUES IN U.S. CODES

A review of the three model building codes[3,29,35] in the United States reveals a somewhat inconsistent and haphazard approach to serviceability issues. For instance, it is implied that the codes exist strictly to protect life safety of the general public. Yet, traditionally they have contained provisions for deflection control of floor members while ignoring provisions for other member types (columns, walls, mullions, etc.). No mention is made of limits for wind drift, vibration, expansion and contraction (expansion joint guidelines), or corrosion.

The author's work in professional committees and code bodies, coupled with a review of recent surveys of the profe-

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ssion[36] seem to reveal a reluctance of engineers to codify serviceability issues. This reluctance probably stems in part on differences of opinion as to the purpose of building codes (i.e., protection for life safety exclusively or establishment of complete minimum design standards including strength and serviceability), but also a genuine concern for restricting design options, stifling creativity, and removing the all-important concept of "engineering judgment" from the solution to the problem. There is also the belief, rightly so, that too little hard data exists to justify rigid standards on most serviceability issues.

It is important that engineers recognize these problems and begin to focus on the solution of serviceability related design issues. The reason for doing so is the large economic impact that serviceability items are having on the operational costs of buildings.

MEAN RECURRENCE INTERVAL WIND LOADS FOR SERVICEABILITY DESIGN

The first step in establishing a serviceability design criterion is to define the load under which it is to be checked. Wind loading criteria for strength limit states in the United States are normally based on a 50-year mean recurrence interval for normal buildings and a 100-year mean recurrence interval for critical structures. There seems to be a general consensus that basing serviceability criteria on such a severe loading that may occur only once, on the average, during the lifetime of the structure is unrealistic and too stringent a standard to apply. The average tenant occupancy in office buildings has been defined as eight years.[26] It seems reasonable to base serviceability criteria on a mean recurrence interval more in this range of time because the consequences of exceeding a serviceability limit state are usually not safety related. Various researchers have suggested mean recurrence intervals of from five to ten years for serviceability issues.[10,11,12,14,17,18,19,20,33,36] If no permanent damage results from exceeding the serviceability limit, some researchers have also suggested selecting serviceability criteria (such as floor deflection) on an annual basis.[14]

A wind load for a mean recurrence interval of 10 years is recommended for checking the two wind serviceability limit states defined herein (*deformation* and *motion perception).* This corresponds to a 10 percent probability of being exceeded in any given year. While it has become standard practice to base building accelerations under wind load on this mean recurrence interval, drift criteria typically have been formulated around the same mean recurrence interval (50 years or 100 years) as the strength limit state.[36]

The proposed 10-year mean recurrence interval compares to five years as proposed in ISO Standard 6897-1984, 10 years as proposed by the National Building Code of Canada (1990), 20 years in the Australian Standard AS 1170.2-1989 and 0.1 years as proposed by the Japanese.[28]

BUILDING DRIFT—STANDARD OF PRACTICE

Serviceability of buildings under wind loads has traditionally been checked in the design office by evaluation of the lateral frame deflection calculated on the basis of a statically applied wind load obtained from the local building code. The magnitude of the wind load is usually the same as that used in proportioning the frame for strength and typically is based on a 50-year or 100-year mean recurrence interval load. Sometimes, an arbitrary wind load (i.e., 20 PSF above 100 ft, 0 (zero) PSF below 100 ft as has been used in New York City on the design of some buildings[15]) is used in the serviceability check. This serviceability check, for all but the tallest and most slender of buildings (where wind tunnel studies are utilized), has been used to prevent damage to collateral building materials, such as cladding and partitions, and also to control the perception of building motion. None of the three national building codes in the United States specify a limit to lateral frame deflection under wind load. The degree of this serviceability check is left to the judgment of the design engineer. Lateral frame deflection is usually evaluated for the building as a whole, where the applicable parameter is *total building drift,* defined as the lateral frame deflection at the top-most occupied floor divided by the height from grade to the uppermost floor (Δ / H); and for each floor of the building, where the applicable parameter is *interstory drift,* defined as the lateral deflection of a floor relative to the one immediately below it divided by the distance between floors ($(\delta_n - \delta_{n-1}) / h$). Typical values of this parameter (commonly called *drift index*) used in this serviceability check are $H / 100$ to $H / 600$ for total building drift and $h / 200$ to $h / 600$ for interstory drift depending on building type and materials used. The most widely used values are $1 / 400$ to $1 / 500$.[36] Lateral frame deflections have historically been based on a first order analysis.

DRIFT—A REVISED DEFINITION [7]

Drift Measurement Index (DMI)

If the goal in defining a drift limit is limited to only the control of damage to collateral building elements, such as cladding and partitions, and is separated from the problem of building motion, then *frame racking* or *shear distortion* (strain) is the logical parameter to evaluate.

Mathematically, if the local x, y displacements are known at each corner of an element or panel, then the overall average shear distortion for rectangular panel ABCD as shown in Figure 1 may be termed the drift measurement index (DMI) and defined as follows:

Drift measurement index (DMI) = average shear distortion

$$\mathrm{DMI} = 0.5 \times [(X_A - X_C) / H + (X_B - X_D) / H + (Y_D - Y_C) / L + (Y_B - Y_A) / L]$$

$$\mathrm{DMI} = 0.5 \times (D1 + D2 + D3 + D4)$$

where,

X_i = vertical displacement of point i
Y_i = lateral displacement of point i
$D1 = (X_A - X_C)/H$, horizontal component of racking drift
$D2 = (X_B - X_D)/H$, horizontal component of racking drift
$D3 = (Y_D - Y_C)/L$, vertical component of racking drift
$D4 = (Y_B - Y_A)/L$, vertical component of racking drift

It is to be noted that terms $D1$ and $D2$ are the horizontal components of the shear distortion or frame racking and are the familiar terms commonly referred to as interstory drift. The terms $D3$ and $D4$ are the vertical components of the shear distortion or frame racking caused by axial deformation of adjacent columns.

If it can be accepted that the DMI is the true measure of potential damage, then it becomes readily apparent that the evaluation of interstory drift alone can be misleading in obtaining a true picture of potential damage. Interstory drift alone does not account for the vertical component of frame racking in the rectangular panel that also contributes to the potential damage, nor does it exclude rigid body rotation of the rectangular panel which, in itself, does not contribute to damage. It can be shown that evaluation of the commonly used interstory drift can significantly underestimate the damage potential in a combined shear wall/frame type building where the vertical component of frame racking can be important; and significantly overestimate the damage potential in a shear wall or braced frame building where large rigid body rotation of a story can occur due to axial shortening of columns.[7]

Consider for example, the eight-story building shown in Figure 2. This frame represents a typical windframe that may be found in any office building with 36-ft lease depths (building perimeter to center core) and a central core containing elevator, stairs, etc. The frame shown consists of a combined moment frame and X-braced frame. Figure 3 shows a plot (exaggerated) of the deflected shape of the top level under wind loads. Table 1 shows calculations for the traditional story drift and the revised drift definition DMI. The significant thing to note is that the potential damaging deformations, as represented by the DMIs, are more severe in the external bays (panels 1, 3) and much less severe in the internal bay (panel 2) than predicted by the traditional story drift calculation. Most of the deformation in the center bay (panel 2) is simply rigid body rotation that, by itself, is not damaging to partitions.

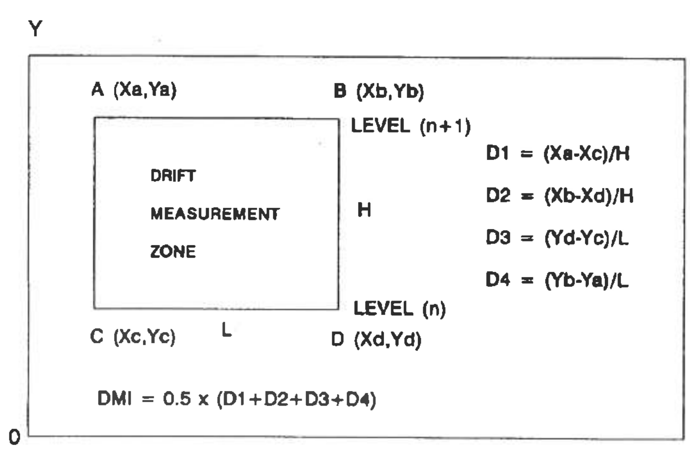

Fig. 1. Drift measurement index (DMI)

Drift Measurement Zone (DMZ)

It is logical to identify the rectangular panel ABCD in Figure 1 as the zone in which the damage potential is to be evaluated and define it the drift measurement zone (DMZ). From a practical standpoint, these zones will typically represent column bays within a building and would be incorporated as part of the building frame analysis.

Drift Damage Index (DDI)

Once the determination of the shear distortion or drift measurement index (DMI) is made for different column bays or drift measurement zones (DMZs), it must be compared to a damage threshold value for the element being protected. These damage threshold limits can be defined as the shear distortion or racking that produces the maximum amount of cracking or distress that can be accepted, on the average, once every 10 years. It is logical to define these damage thre-

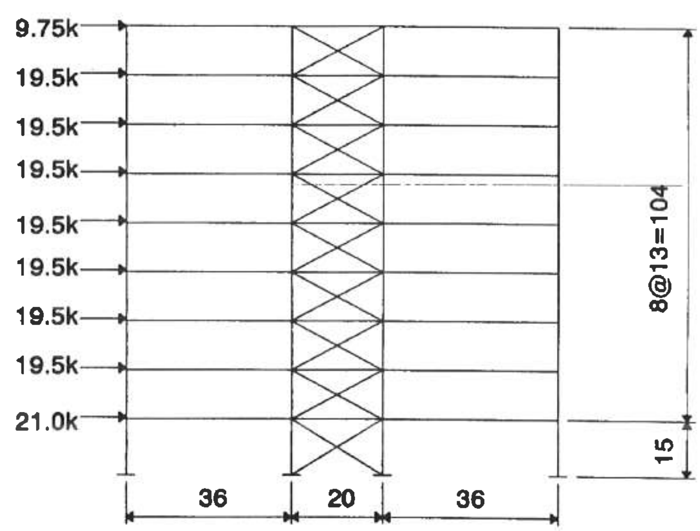

Figure 2

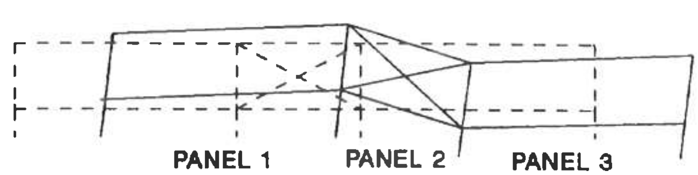

Figure 3

Table 1. Drift Comparison							
	D1	D2	D3	D4	Drift	DMI	DMI/Story Drift
Panel 1	0.00101	0.00104	0.000220	0.000215	0.00101	0.0012500	1.23
Panel 2	0.00104	0.00104	−0.001030	−0.001020	0.00101	0.0000186	0.02
Panel 3	0.00104	0.00101	0.000214	0.000209	0.00101	0.0012400	1.22

shold shear distortions as the drift damage index (DDI). From the standpoint of serviceability limit states it is necessary to observe the following inequality:

drift measurementindex ≤ drift damage index

$$\text{DMI} \leq \text{DDI}$$

A significant body of information is available from racking tests for different building materials that may be utilized to define DDIs.[2] This is discussed further below in conjunction with Figure 4.

Calculation of Building Frame Deflection

If drift measurement indices (DMIs) are to be effective in controlling collateral building material damage, there must be a consistency and accuracy in the method of calculation. A recent survey[36] on drift clearly pointed out the problems that exist in the structural engineering community on controlling damage by excessive drift. There appears to be a wide variation in the methods of structural analysis performed to calculate building frame deflection. Ideally, if DMIs are to be an effective parameter in controlling damage caused by building deflection, then the structural analysis employed must reasonably capture the significant response of the building frame under load. As previously stated, it is suggested that the wind load be defined by the 10-year mean recurrence interval storm. The designer should recognize that the wind loads used in the structural analysis are "static equivalent" wind loads that are intended to estimate the peak load effect (mean plus dynamic) caused by the vibratory nature of the building motion. The structural analysis must then capture all significant components of potential frame deflection as follows:

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

The behavioral knowledge of each of the above effects on frame deflection is sufficiently understood to permit a reasonably accurate prediction of the contribution to the total response. Computer programs and analytical models are now within reach of most engineers to afford consideration of all of the above effects.

Depending on the height, slenderness, and column bay geometry, each of these effects can have a significant influence on building deflection. A recent study[8] on the sources of elastic deformation for different height (10 to 50 stories) and number-of-bay (5 to 13 bays) frames showed the following:

1. Axial deformations in columns can be very significant for tall slender frames, amounting to 26 percent to 59 percent of the total deflection, depending on bay widths.
2. Shear deformations, as a percentage of the total frame deflection, tend to increase with the number of bays and also as the bay size (beam span) reduces. Shear deformation can account for as much as 26 percent of the total deflection. For slender "tube" structures (10- to 15-ft bays and 40 to 50 stories tall) shear deformation can contribute as much as flexural deformation to the total building deflection. Shear deformations should never be ignored in frame deflection if an accurate response prediction is expected.
3. Beam-column joint deformations, particularly for steel structures, constitute a significant portion of the total deflection for all frames studied and should never by ignored. As with shear deformations, there is a general trend for deformations to increase as the number of bays increases and the size of the bay decreases. Participation as a percentage of the total

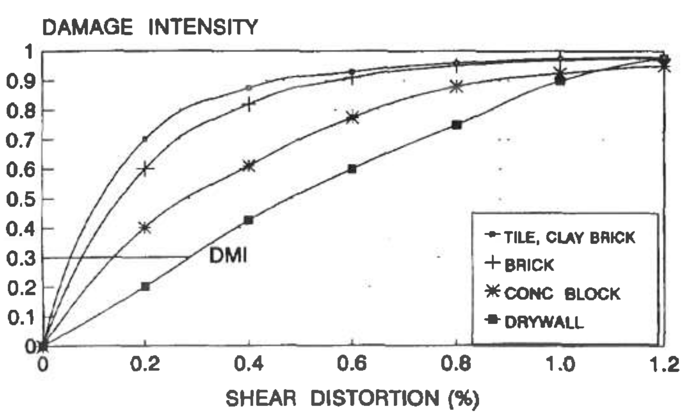

Fig. 4. Drift damage threshold—partitions.

Table 2. Serviceability Problems at Various Deflection or Drift Indices [14,31]		
Deformation as a Fraction of Span or Height	**Visibility of Deformation**	**Typical Behavior**
≤ 1/1000	Not Visible	Cracking of brickwork
1/500	Not Visible	Cracking of partition walls
1/300	Visible	General architectural damage Cracking in reinforced walls Cracking in secondary members Damage to ceiling and flooring Facade damage Cladding leakage Visual annoyance
1/200–1/300	Visible	Improper drainage
1/100–1/200	Visible	Damage to lightweight partitions, windows, finishes Impaired operation of removable components such as doors, windows, sliding partitions

varied from 16 percent to 41 percent.

4. The P-Δ effect can easily increase total frame displacement by 10 to 15 percent depending on frame slenderness.

Errors in the determination of frame stiffness can also affect proper design for strength. For example, the P-Δ effect is a function of frame stiffness; the magnitude of wind forces in tall buildings is affected by building period; and the magnitude of seismic forces is also affected by building period.

DAMAGE THRESHOLDS FOR BUILDING MATERIALS

General guidelines to the behavior that might be expected from different building elements and materials at various drift indices may be obtained from a review of the literature.[2,13,31] A summary of behavior, taken from a recent study on serviceability research needs[31] is shown in Table 2. Another source of information may be found in seismic racking tests of exterior cladding systems for buildings sometimes performed during routine testing of mock-ups at testing laboratories. One of the most comprehensive studies of damage intensity as a function of shear distortion can be found in Reference 2 which contains a summary of over 700 racking tests on various nonstructural partitions taken from more than 30 different sources. Partition types surveyed included tile and hollow brick, concrete block, brick and "veneer"; walls which consisted of gypsum wall board, plaster, and plywood. Veneer walls are often referred to as "drywall" in engineering practice. Damage intensity was defined on a scale from 0.0 to 1.0 with 0.1 to 0.3 defined as *minor* damage, 0.4 to 0.5 defined as *moderate* damage, 0.6 to 0.7 defined as *substantial* damage, and 0.8 to 1.0 defined as *major* damage. A damage intensity of 1.0 is defined as complete or *intolerable.* Figure 4 shows a plot of damage intensity versus shear distortion for the partition groups discussed. If the upper limit of the "minor damage" range is selected as the maximum acceptable damage to occur in a 10-year design period, then the deflection limit of 0.25 percent (1 / 400) is obtained for veneer or drywall in Figure 4. This number correlates reasonably well with the first damage threshold limit of ¼-in. displacement for an eight foot tall test panel as described in Reference 13 for gypsum wallboard. The 0.3 damage intensity has been used as the maximum acceptable shear distortion for the various partition types in Table 3.

SERVICEABILITY LIMIT STATE—DEFORMATION (CURVATURE, DEFLECTION, DRIFT)

Once a wind load (mean recurrence interval) has been defined for use in the serviceability check, the appropriate deformation to measure it has been defined (drift measurement index (DMI)) and damage thresholds are determined from tests or estimated, it remains only to establish an appropriate limit for different building components. Table 3 is a compilation of most common building elements with recommended deformation limits. The building elements considered include roof, exterior cladding, interior partitions, elevators, and cranes. Most of the more common building cladding and partition types are considered. Deformation types addressed include deflection perpendicular to the plane of the building element and shear deformation (racking) in the plane of the element. Notes are included at

Table 3.
Wind Serviceability Limit State Deformation

Building Element		**Supporting Structural Element**	**Deformation Type**	**Recommended Limit**	**Comments**
Roof	Membrane Roof	Purlin, Joist, Truss	Deflection ⊥ Roof Plane	L / 240	—
	Metal Roof	Purlin, Joist, Truss	Deflection ⊥ Roof Plane	L / 150	Note 1
	Skylights	Purlin, Joist, Truss	Differential Support Deflection	L / 240 ≤ ½-in	Note 2
Exterior Cladding	Brick Veneer	Metal/Wood Stud	Deflection ⊥ Wall Plane	H / 600	Note 3
		Horizontal Girts	Deflection ⊥ Wall Plane	L / 300	Note 4
		Vertical Girts/Cols.	Deflection ⊥ Wall Plane	L / 600	Note 4
		Wind Frame	Shear Strain (DMI)	H / 400	Note 5
	Concrete Masonry Unreinforced (Note 6)	Horizontal Girts	Deflection ⊥ Wall Plane	L / 300	Note 4
		Vertical Girts/Cols.	Deflection ⊥ Wall Plane	L / 600	Note 4
		Wind Frame, One-story	Shear Strain (DMI)	H / 600	Note 7
		Wind Frame, Multi-story	Shear Strain (DMI)	H / 400	Note 8
	Concrete Masonry Reinforced (Note 6)	Horizontal Girts	Deflection ⊥ Wall Plane	L / 240	—
		Vertical Girts/Cols.	Deflection ⊥ Wall Plane	L / 240	—
		Wind Frame, One-story	Shear Strain (DMI)	H / 200	Note 9
		Wind Frame, Multi-story	Shear Strain (DMI)	H / 400	Note 10
	Tilt-up Concrete	Horizontal/Vertical Girts	Deflection ⊥ Wall Plane	L/240	Note 11
		Wind Frame	Shear Strain (DMI)	H / 200	Note 12
	Plaster, Stucco	Metal/Wood Stud	Deflection ⊥ Wall Plane	H / 600	Note 13
		Horizontal/Vertical Girts	Deflection ⊥ Wall Plane	L / 600	Note 13
		Wind Frame	Shear Strain (DMI)	H / 400	Note 14
	Architectural Precast Concrete Panels, Stone Clad Precast Concrete Panels	Horizontal/Vertical Girts	Deflection ⊥ Wall Plane	L / 240	Note 11
		Wind Frame	Shear Strain (DMI)	H / 400	Note 15
	Architectural Metal Panel	Metal Stud, Vertical/Horizontal Girts	Deflection ⊥ Wall Plane	L / 120	Note 16
		Wind Frame	Shear Strain (DMI)	H / 100	Note 17
	Curtain Wall, Window Wall	Mullions, Horizontal/Vertical Girts	Deflection ⊥ Glass Plane	L / 175	Note 18
		Wind Frame	Shear Strain (DMI)	H / 400	Note 19
Interior Partitions	Gypsum Drywall, Plaster	Wind Frame	Shear Strain (DMI)	H / 400	Note 20
	Concrete Masonry Unreinforced (Note 6)	Wind Frame	Shear Strain (DMI)	H / 667	Note 20
	Concrete Masonry Reinforced (Note 6)	Wind Frame	Shear Strain (DMI)	H / 400	Note 10, 20
	Tile, Hollow Clay Brick	Wind Frame	Shear Strain (DMI)	H / 2000	Note 20
	Brick	Wind Frame	Shear Strain (DMI)	H / 1250	Note 20

Table 3, cont'd Wind Serviceability Limit State Deformation					
Building Element		**Supporting Structural Element**	**Deformation Type**	**Recommended Limit**	**Comments**
Elevators		Wind Frame	Shear Strain (DMI)	*H* / 400	Note 21
Cranes	Cab Operated	Wind Frame	Shear Strain (DMI)	*H* / 240≤2-in.	Note 22
	Pendant Operated	Wind Frame	Shear Strain (DMI)	*H* / 100	Note 23

Notes to Table C.

H = story height *L* = span length of supporting member DMI = drift measurement index

1. Metal roofs include standing seam and thru fastener type roofs.[12]
2. Deflection limit shown is relative support movement measured perpendicular to a line drawn between skylight support points. Racking movements in the plane of the glass should be limited to ¼-in. for gasketed mullions and ⅛-in. for flush (butt) glazing. [12]
3. Deflection limits recommended by the Brick Institute of America[34] are *L*/600 - *L*/720.
4. *L*/600 is recommended for the case when predominant flexural stress in masonry is perpendicular to bed joint *L*/300 may be used for the case when predominant flexural stress in masonry is parallel to bed joint.
5. *H*/400 limit applies if brick is supported on relief angles at each floor with ⅜-in. soft joint and ⅜-in. control joints are used in each column bay.
6. Reinforced concrete masonry implies vertical reinforcing bars in grouted cells and/or horizontal reinforcing bars in bond beams.
7. Assumes only windframe designed to carry lateral loads and flexible connections used between wall and parallel windframe. *H*/600 limit also protects wall perpendicular to plane of windframe from excessive flexural cracking. A horizontal crack control joint at base of wall is recommended. Limit crack width under wind load to 1/16-in. if no joint is used and ¼-in. if control joint is used. [12]
8. Assumes only windframe designed to carry lateral loads and flexible connections used between wall and parallel windframe. *H*/400 applies only if in-fill walls have ⅜-in. soft joints against structural frame.
9. Assumes only windframe designed to carry lateral loads and flexible connections used between wall and parallel windframe. Stricter limit should be considered if required to protect other building elements. If walls designed as shear walls, then design DMI should be based on damage control of other building elements. *H*/200 limit also protects wall perpendicular to plane of windframe from excessive flexural cracking. If a horizontal control joint at base of wall is used, then limit may be changed to *H*/100. [12]
10. *H*/400 limit applies to reinforced masonry walls designed as shear walls unless stricter limit is required to protect other more critical building elements. Reinforced masonry walls infilled "hard" against structural windframe should not be used without assessing their stiffness in a compatibility analysis with windframe, unless isolation joints are provided between wall and building frame.
11. In cases where wall support is indeterminate, differential support deflection should be considered in design of wall panel.
12. Assumes only windframe designed to carry lateral loads and flexible connections are used between wall and parallel windframe. Stricter limit should be considered if required to protect other building elements. If panels designed as shear walls then *H*/400 is recommended limit with minimum ¾-in. panel joints.
13. Control joints are recommended to limit cracking from shrinkage, thermal, and building movement.
14. *H*/400 limit applies if wall is panelized with ⅜-in. control joints and relief joints are used between floors and at each column bay. If plaster applied to unreinforced masonry, then limits should be same as masonry.
15. *H*/400 applies if panel connection to frame is determinate, flexible connections are used between panel and parallel windframe and minimum ¾-in. panel joints are used. Panels with indeterminate support to frame should be designed for differential support movement.
16. Consult metal panel manufacturer for possible stricter requirements.
17. *L*/100 limit applies for metal panel only. Other building components may warrant stricter limit.
18. *L*/175 recommended by American Architectural Manufacturers Association.[27] Recommended limit changes to *L*/360 when a plastered surface or dry wall subjected to bending is affected. At roof parapet or other overhangs recommended limit is 2*L*/175 except that the deflection of a member overhanging an anchor joint with sealed joint (such as for roof flashing, parapet cover, soffit) shall be limited to no more than one half the sealant joint depth between the framing member and fixed building element.
19. *H*/400 limit is to protect connections to building frame and also sealants between panels. More liberal limits may be applicable for custom designed curtain/window walls where racking can be accounted for in design and where wall will be tested in a labortory mock-up. Consult manufacturer for racking limits of off-the-shelf systems.
20. Recommended limits shown assume partition is constructed "hard" against structural frame. More liberal limits may be appropriate if isolation ("soft") joints are designed between partition edge and structural frame. Design of structural frame for DMI limits stricter than *H*/600 is probably not practical or cost effective.
21. In addition to the static deflection limit shown, 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.
22. Limit shown applies to wind loads or crane forces, either lateral or longitudinal to crane runway. Deflection limit specified is to be measured at the elevation of crane runways. [12]
23. Buildings designed to *H*/100 limit will exhibit observable movements during crane operation. Stricter limits may be appropriate to control this and protect other building components.[12]/or to

the end of the table to explain or clarify a recommendation.

It should be pointed out that the recommended limits shown are guidelines based on past successful performance. The degree of distress in any of the building elements (cladding, partitions) under the action of wind loads is highly dependent upon the nature and design of the attachments or joints to the building frame. If specific attention is paid to this aspect then oftentimes any reasonable deformation can be accommodated without damage. Indeed, it may be more prudent and cost effective to detail joints to accommodate a higher deflection than to design a higher level of stiffness into the building wind frame.

SERVICEABILITY LIMIT STATE—MOTION PERCEPTION

Motion Perception Parameter—Acceleration [28,30]

Perception to building motion under the action of wind 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 [25]

Perception and tolerance thresholds of acceleration 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. *Sex.* The general trend of response between men and women is the same although women are slightly more sensitive than men.
3. *Age.* The sensitivity of humans to motion is an inverse function of age, with children being more sensitive than adults.
4. *Body Posture.* The sensitivity of humans to motion is proportional to the distance of the persons 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.
5. *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.
6. *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.
7. *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.
8. *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.
9. *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.
10. *Type of Motion.* Under the influences 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.[24]

Root-Mean-Square (RMS) 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 researchers report maximum or peak acceleration and some report root-mean-square or RMS accelerations. 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 10-year mean recurrence interval, that 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.[17,18,22,33] 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 which 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 of 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,[22] 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 a satisfactory performance with regard to motion perception. The results of one such study[22] are plotted in Figure 5 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. This is discussed further later in the paper.

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.[9,25,28] 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[5,6] who proposed peak acceleration limits for different comfort levels that were extrapolated from data in the aircraft industry. Chang's proposed limits, plotted in Figure 6 as a function of building period, 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

Additional data has been reported by researchers who utilized motion simulators to define perception levels.[9,28] A summary of this work is shown in Figures 7 and 8 showing plots of perception thresholds for both peak and RMS acceleration as a function of building period.

Perhaps the most comprehensive studies of the problem have been performed in Japan[28] for a wide range of variables.

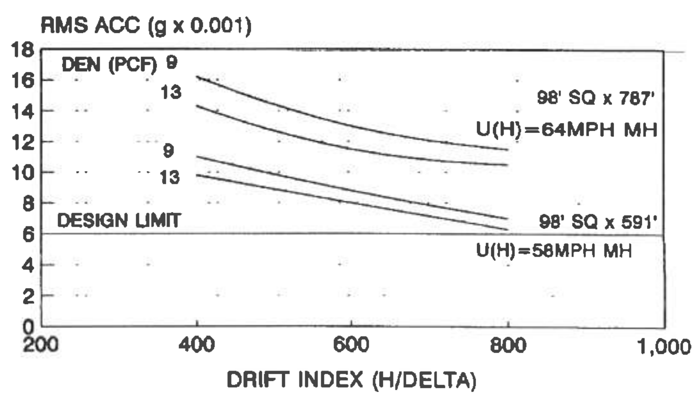

Fig. 5. RMS acceleration vs drift index.

This work is summarized in Figure 8 where peak acceleration is plotted as a function of building period. Each curve and zone between curves is identified in the figure. The discussion below is keyed to the letters and numbers in the figure and is taken from Reference 28:

1. Zone A, below Curve 1, identifies peak acceleration less than about 0.5 percent g. In this zone, a human cannot perceive motion at all. No evidence of motion exists except for possible rubbing of building component surfaces in contact. Curve 1 defines the limit of perception threshold for an average population.
2. Curve 2 (0.5 percent g) defines the point where some building objects (furniture, hanging lights, water) begin to move.
3. Curve 3 separates zones between "very normal walking" and "nearly normal walking."
4. Zone B (between 0.5 percent g and 1.0 percent g) identifies a zone where some people can perceive motion. Some building fixtures and objects will begin to move slightly, but these movements are generally not observable except to a person who looks directly at them.
5. Curve 4 (1 percent g) separates the zones where people can be affected by working at a desk.
6. Curve 5 defines the threshold where people can start to become subjected to motion sickness when exposed to this level of motion for extended periods.
7. Zone C (between 1.0 percent g and 2.5 percent g) is where most people are able to perceive motion and become affected by desk work. Generally, in this zone, people can be subjected to motion sickness if exposed for extended periods but can walk without hindrance.
8. Curve 6 defines the limit between normal and hindered walking.
9. Zone D (between 2.5 percent g and 4.0 percent g) defines the acceleration range where desk work becomes difficult and at times impossible. Most people can walk and go up and down stairs without too much difficulty.
10. Curve 7 (3.5 percent g) defines the point where working at a desk is difficult.
11. Curve 8 (4.0 percent g) defines the acceleration where furniture and fixtures begin to make sounds, which may evoke a strong concern or alarm among some people.
12. In Zone E people strongly perceive motion and standing people lose their balance and find it hard to walk naturally.
13. Curve 9 marks the point where people are unable to walk.
14. Curve 10 defines the maximum tolerance for motion.
15. In Zones F and G (above 5.0 percent g) most people cannot tolerate the motion and are unable to walk. These zones are considered to be at the limit of walking ability.
16. In Zone H people cannot walk. Motion is intolerable.

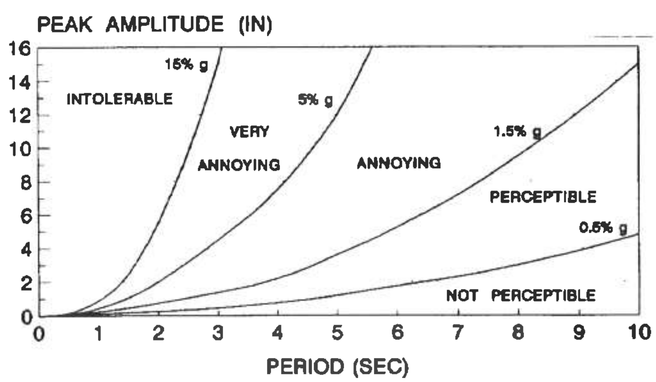

Fig. 6. Tolerance thresholds proposed by Chang.

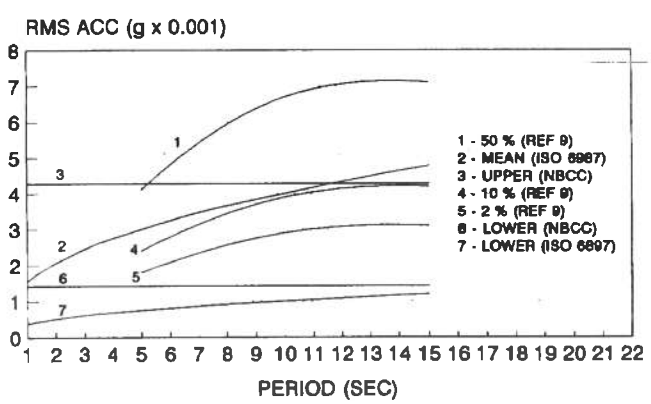

Fig. 7. Perception threshold—RMS acceleration.

Design of Tall Buildings for Acceleration

The design of most tall buildings is controlled by lateral deflection 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

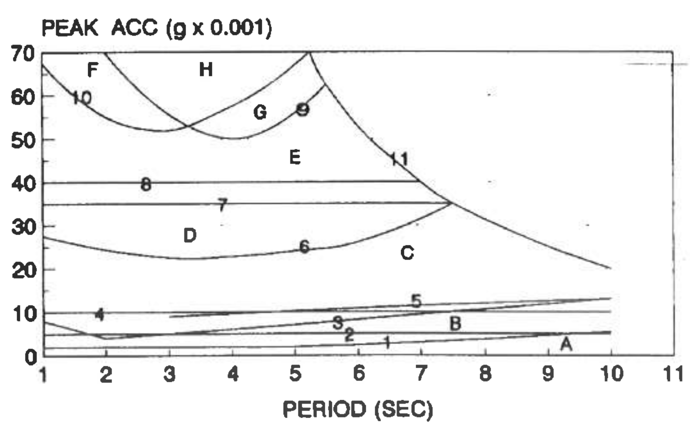

Fig. 8. Perception thresholds—peak acceleration.

force-balance or aeroelastic test in the wind tunnel, certain criteria have been established to aid the designer. Empirical expressions now exist[21,22,32] 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 are not yet fixed.

The following simple expressions[22] for along-wind (drag), across-wind (lift), and torsional RMS acceleration have been derived for square, symmetric (coincident centers of mass, rigidity, and geometry), tall buildings in an urban environment:

Along-wind:

$$A_D(Z) = C_D(Z)\frac{U_H^{2.74}}{K_D^{0.37} \times \zeta^{0.5} \times M_D^{0.63}} \tag{1}$$

Across-wind:

$$A_L(Z) = C_L(Z)\frac{U_H^{3.54}}{K_D^{0.77} \times \zeta^{0.5} \times M_L^{0.23}} \tag{2}$$

Torsional:

$$A_\theta(Z) = C_\theta(Z)\frac{U_H^{1.88}}{K_\theta^{-0.06} \times \zeta^{0.5} \times M_\theta^{1.06}},\ \frac{N_\theta B}{U_H} \le 0.25 \tag{3a}$$

$$A_\theta(Z) = C_\theta(Z)\frac{U_H^{2.76}}{K_\theta^{0.38} \times \zeta^{0.5} \times M_\theta^{0.62}},\ \frac{N_\theta B}{U_H} > 0.25 \tag{3b}$$

The proportionality constants $C_D(Z)$, $C_L(Z)$, and $C_\theta(Z)$ are defined as follows:

$$C_D(Z) = 0.0116 \times B^{0.26} \times Z$$

$$C_L(Z) = 0.0263 \times B^{-0.54} \times Z$$

$$C_\theta(Z) = 0.00341 \times B^{2.12} \times Z,\ \frac{N_\theta B}{U_H} \le 0.25$$

$$C_\theta(Z) = 0.00510 \times B^{1.24} \times Z,\ \frac{N_\theta B}{U_H} > 0.25$$

The definition of terms in the above expressions are listed below:

$A_D(Z), A_L(Z), A_\theta(Z)$ = along-wind, across-wind, and torsional RMS acceleration at height Z (meters/sec^2, adians/sec^2)

U_H = mean hourly wind speed at the top of the building (meters/sec.)

H = building height (meters)

B = plan dimension of square building (meters)

M = generalized mass of the building (kilogram) $= \sum_{i=1}^{n} m_i\phi_i^2$ m_i is mass of floor i and ϕ_i is modal coordinate at floor i, normalized so that $\phi = 1$ at $(Z) = H$

N = frequency (hertz)

K = generalized stiffness (newton/meters) $= (2\pi N)^2 \times M$

ς = damping ratio

For rectangular buildings, B may be taken as the square root of the plan area. The resultant RMS acceleration at the corner of the building, A_R, is calculated as follows:

$$A_R = (A_D^2 + A_L^2 + (B/\sqrt{2} \times A_\theta)^2)^{0.5} \tag{4}$$

These expressions were used in a parametric study of a 150-ft square building having slenderness ratios (H / B) of five through ten (building heights varying from 754 feet to 1,495 feet). The buildings were subjected to basic wind speed of 70 mph in an Exposure B (suburban) environment as defined in ASCE 7-88. The buildings were assumed to be all-steel with steel weights typical of tall buildings of these heights, varying from 25 psf to 44 psf. Building densities were assumed to vary from 7.77 pcf to 9.23 pcf, typical for office buildings having lightweight concrete metal deck floors and curtain wall cladding. Translational building periods were calculated using the well-known Rayleigh formula,[35] which for uniform prismatic buildings with a linear deflected shape can be approximated by the following expression:

$$T = 0.904H\left(\frac{\rho D_R}{pR}\right)^{0.5} \tag{5}$$

In this expression, T is the building period in seconds, H is the building height (feet), ρ is the density (PCF), D_R is the design drift ratio (Δ/H), p is the equivalent uniform pressure (PSF) and R is the aspect ratio H/B. Torsional periods were taken as 85 percent of the translational periods. For this study, the drift ratio under design wind load as defined by ASCE 7-88 is set at 1/400 or 0.0025. This practice is typical of the procedure used in many building designs.

Along-wind, across-wind, and torsional RMS accelerations were calculated at the building top corner using 10-year mean recurrence interval wind loads. Complete building data is shown in Table 4 and the accelerations are plotted in Figure 9. Also shown in Figure 9 is the design limit as defined later in this paper. The results clearly show that controlling drift limits to the traditional design value of 0.0025 does not ensure satisfactory performance from the standpoint of motion perception. In examining Figure 9, it is interesting to note that for the common aspect ratios of 5-6, torsional acceleration is comparable to across-wind acceleration and both are significantly larger than the along-wind acceleration.

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

Table 4. Parametric Study RMS Acceleration 150-ft. Square Building										
H Ft.	*H / B*	T_L, T_D (SEC)	T_θ (SEC)	STL. WT. (PSF)	ρ (PCF)	U_H (MPH)	A_D (Milli-g)	A_L (Milli-g)	$B / \sqrt{2} \times A_\theta$ (Milli-g)	A_R (Milli-g)
754	5	6.85	5.82	25	7.77	58.6	2.96	4.60	4.83	7.30
897	6	7.31	6.21	29	8.08	63.3	3.69	6.43	6.04	9.57
1053	7	7.77	6.60	33	8.38	68.0	4.53	8.78	6.82	12.00
1196	8	8.19	6.96	37	8.69	71.8	5.26	11.10	7.22	14.25
1352	9	8.61	7.32	41	9.00	72.0	5.32	11.73	6.98	14.65
1495	10	8.97	7.62	44	9.23	72.0	5.35	12.18	6.78	14.93
NOTE: RMS accelerations are calculated using Equations 1 2 and 3.										

Table 5. Traditional Motion Perception (Acceleration) Guidelines (Note 1) 10-year Mean Recurrence Interval				
		Root-mean-square (RMS) Acceleration (Milli-g)		
Occupancy Type	Peak Acceleration (Milli-g)	$1 \leq T < 4$ $0.25 < f \leq 1.0$ ($g_p \approx 4.0$)	$4 \leq T < 10$ $0.1 < f \leq 0.25$ ($g_p \approx 3.75$)	$T \geq 10$ $f \leq 0.1$ ($g_p \approx 3.5$)
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

NOTE:
1. RMS and peak accelerations listed in this table are the traditional "unofficial" standard applied in U.S. practice based on the author's experience.

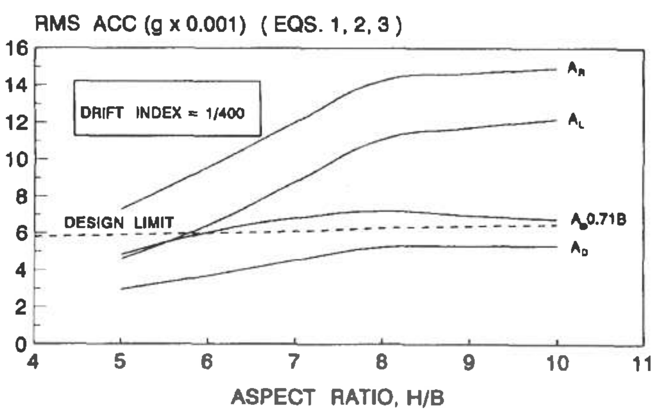

Fig. 9. Parametric study—150 -ft sq bldg.

along-wind plan dimensions respectively and *H* is the building height.[32]

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

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 *K* is the stiffness.
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 stif-

fitness 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 $1 / \xi^{0.5}$.

It should be pointed out 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.[16]

Standards of Motion Perception

Numerous high-rise buildings have been designed and are performing successfully all over the world. Many have been designed according to an "unofficial" standard observed in the author's practice as defined in Table 5. Both peak acceleration and RMS accelerations are used, their relationship generally defined by the use of a peak factor, g_p, 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.[32] Target peak accelerations of 21 milli-g's and 15 milli-g's are often used for commercial and residential buildings respectively. Corresponding RMS values are ratioed accordingly using the appropriate peak factor. A stricter standard is often applied to residential buildings for the following reasons:[4]

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.

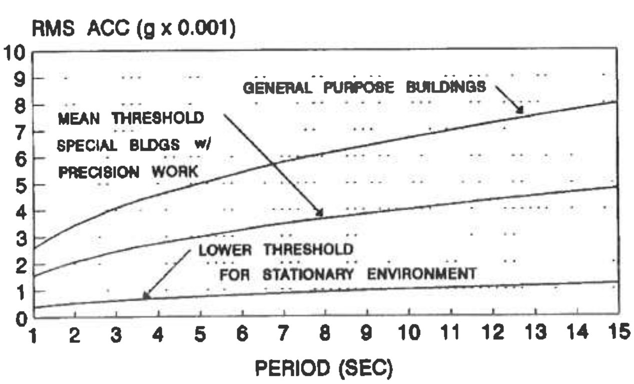

Fig. 10. RMS acceleration—ISO 6897–1984 5-yr return period.

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 5 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 (see Curves 1, 4 and 5 in Figure 7 and Curves 5 and 6 in Figure 8).

The International Organization of Standardization has established a design standard for occupant comfort in fixed structures subjected to low frequency horizontal motion—ISO Standard 6897-1984.[17] This standard is based on a five-year mean recurrence interval and seems to agree quite well with the experimental work described in Figures 7 and 8. The mean threshold curve from this standard is plotted for comparison to the research of Reference 9 in Figure 7. The ISO Standard 6897 design curves are plotted in Figure 10. The interesting feature of the ISO approach is that acceleration limits increase as the building period increases and therefore it represents a better correlation to available research. The acceleration limits defined by the "General Purpose Buildings" curve of Figure 10 agree very well with

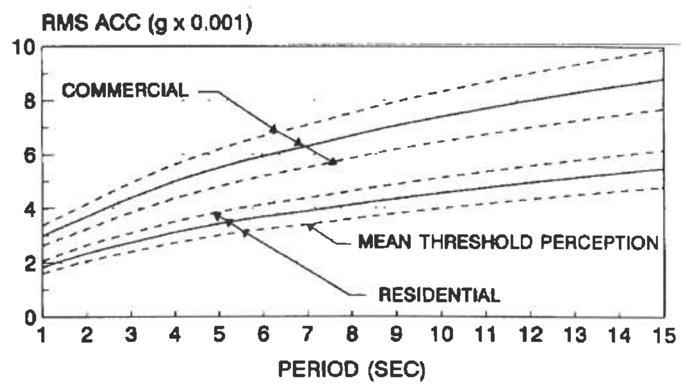

Fig. 11. Design standard—RMS acceleration 10-yr return period.

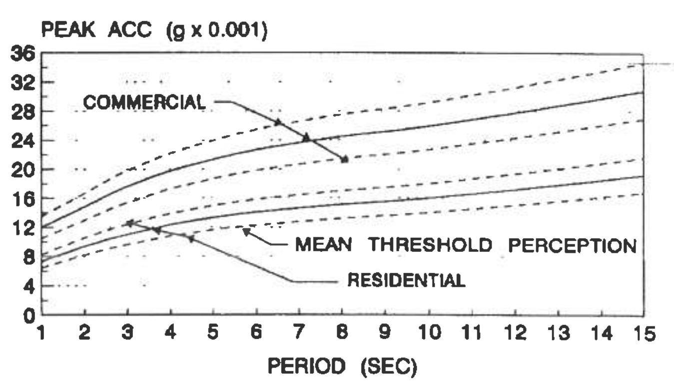

Fig. 12. Design standard—peak acceleration 10-yr return period.

U.S. practice for commercial buildings if an upward adjustment of approximately 10 percent is used to account for the difference in mean recurrence intervals for U.S. practice (10-year versus the ISO five-year mean recurrence interval). The 10 percent adjustment seems reasonable in light of the author's experience in wind engineering studies performed on office buildings.

The author's observation and experience with U.S. practice, combined with a study of the available research previously described and also the ISO Standard 6897-1984, form the basis of a proposed new standard defined in Figures 11 and 12. Design limits are proposed for both peak and RMS acceleration using a 10-year mean recurrence interval wind as customarily used in U.S. practice. The logic used in the formulation of these curves is described below:

1. Design curves are established for residential buildings and for commercial buildings. Residential buildings demand a separate stricter standard for the reasons previously stated. Target values are given for each building type centered between an upper and lower bound. The upper bound values are 12.5 percent above and the lower bound values 12.5 percent below the target values. The concept of a design range seems reasonable considering the limited available research and the uncertainty in the present state-of-the-art.
2. The ISO 6897 curve for mean threshold acceleration (middle curve of Figure 10) is taken as a lower bound for the residential building curves shown in Figure 11.
3. The target and the upper bound values are established considering the design range defined in Item 1.
4. The commercial building target curve is defined by using the ISO Standard "General Purpose Building" curve of Figure 10, increased by 10 percent to reflect the change in mean recurrence intervals. The upper and lower bounds are defined 12.5 percent above and below the target curve respectively.
5. The peak acceleration curves defined in Figure 12 are based on the corresponding RMS acceleration curves of Figure 11 multiplied by a peak factor as defined in Table 5.

Additional research and experience will be required to confirm the validity of this proposed new standard. The acceleration levels relate reasonably well (slightly higher) with the successful experience of Table 5 and the new standard has the advantage of frequency dependency that seems to be confirmed by research.

CONCLUSIONS

This paper has focused on two serviceability limit states for buildings (particularly tall and/or slender buildings); namely, *deformation* (deflection, curvature, and drift) and *motion perception* as measured by acceleration.

The conclusions reached in this paper are summarized below:

1. The current practice of using 50-year or 100-year mean recurrence interval wind loads to evaluate building drift with currently accepted drift limits is overly conservative. Wind drift and acceleration are proposed to be based on a mean recurrence interval of 10 years.
2. A revised definition of building drift is proposed to better reflect the potential for damage to building elements. The new definition, termed herein as the drift measurement index (DMI) is a mathematical formulation of shear deformation or racking that occurs in a building element. It includes the vertical component of racking and filters out the effect of rigid body rotation, both of which are shortcomings in the present definition of building drift. The term given to the rectangular panel forming the zone over which shear deformation is to be measured is drift measurement zone (DMZ). The threshold damage distortion that represents the limit of shear deformation that causes distress is termed the drift damage index (DDI). The drift limit state may then be stated as DMI $\leq$ DDI under 10-year wind loads.
3. If rational drift limits are to be effective, the calculation of building drift must capture all significant components of frame deflection including flexural deformation of beams and columns, axial deformation of columns, shear deformation of beams and columns, beam-column joint deformation (panel zone deformation), effect of member joint size, and the $P\text{–}\Delta$ effect.
4. A review of available racking distortion data for different partition types is made. Based on this information, and past successful experience, guidelines (Table 3) are proposed for different building elements (roofs, cladding, partitions, elevators, and cranes) subjected to 10-year wind loads.
5. Factors affecting human response to building motion are reviewed and include building frequency, sex, age, body posture, body orientation, expectancy of motion, and body movement of the occupants; visual cues, acoustic cues, and type of motion.
6. Acceleration appears to be the best indicator of building motion at present.
7. Both RMS (root-mean-square) and peak acceleration values are commonly used to represent building motion. There appears to be a difference of opinion among engineers and researchers as to the relative importance and merits of each. This issue should be resolved to avoid confusion in the development of design standards.
8. Contrary to early attempts by engineers to control annoying lateral vibrations in buildings, building stiffness, represented by drift ratios, by itself is *not* a good indicator of occupant susceptibility to building motion (Figure 5). Perception of building motion is influenced by available damping and also building mass as well as building stiffness.

9. Research seems to indicate that human perception to acceleration begins at about 0.5 percent (peak acceleration) and appears to increase as the building period increases (Figure 8).
10. Human tolerance to acceleration tends to increase with building period above about three to four seconds (Figure 8).
11. Current practice in tall building design has targeted design values for acceleration at 21 milli-g's peak acceleration (6 milli-g's RMS) for office buildings and 15 milli-g's peak acceleration (4.3 milli-g's RMS) for residential buildings. These limits do not recognize the apparent trend for the dependence of acceleration limits on building period.
12. Humans appear to be particularly sensitive to torsional acceleration and so this component should be minimized in the assignment of building mass and stiffness as much as possible during the design stage.
13. The factors affecting building acceleration are best evaluated in a wind tunnel study. Acceleration involves the complex inter-relationship of the variables of mass, stiffness, and damping and also the influence of building orientation on the site and the surrounding wind environment.
14. Most tall slender building motion is controlled by across-wind effects (vortex shedding). Generally speaking, this component of acceleration is proportional to the wind velocity to a power of about 3.5 and the period of the building to a power of about 1.5; and inversely proportional to mass and the square root of damping.
15. Proposed standards for building perception (commercial and residential buildings) are shown in Figures 11 and 12 which show acceleration limits increasing with building period. This seems to follow the results of past research and is an improvement over current standards.
16. The current approach to serviceability design is inconsistent in U.S. building codes and seems to reflect a general reluctance by practicing engineers to codify serviceability standards.
17. Structural engineers must begin to address serviceability issues in design and establish rational standards because of the increasing economic impact serviceability issues are having on construction.

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