

# Serviceability Guidelines for Steel Structures

BRUCE ELLINGWOOD

Serviceability limit states are conditions in which the functions of a building are disrupted because of local minor damage to or deterioration of building components or because of occupant discomfort.<sup>1,8,17</sup> Many serviceability limit-states are the result of excessive structural deformations or motions. Current codes and standards used in steel design deal with the complex problems of serviceability limit-states with simple rules.<sup>18</sup> Some of these rules have remained essentially unchanged for over a century. For the most part, they require only a check of deflections of the floor or frame under service load conditions. Thus, approach to serviceability presumes that a broad spectrum of building structure performance issues can be dealt with simply by means of static deflection checks.

A closer look at serviceability issues is warranted by trends in modern construction.<sup>1</sup> Changing architectural and building-use requirements, improvements in structural analysis and design made possible by the widespread use of computers, the use of lighter and less rigid cladding, uncoupling of nonstructural elements from the structural frame, the increasing use of high-strength steel leading to longer spans and bay sizes; all have resulted in building structural systems that are less stiff and massive. Such systems may deflect or vibrate excessively at service-load levels.

Recent advances in structural codes have been accomplished within the framework of probability based limit-states design.<sup>4,6,9</sup> Comprehensive loading and resistance criteria have been developed for the ultimate or safety-related limit-states. The new Load and Resistance Factor Design (LRFD) Specification for Steel Structures,<sup>15</sup> published by the American Institute of Steel Construction in late 1986, is among the first of a new generation of design documents for engineered construction based on the unifying concepts of probability-based limit-states design. Substantial research data were available on which to base these criteria. There has been relatively less research on serviceability limit-states. The problems tend to be difficult to identify and analyze, depending as they do in part on human perceptions of adequate performance. Moreover, because building economics rather than public safety

generally is at issue, many designers and building officials are ambivalent on whether serviceability criteria should be included in codes and specifications.

## SERVICEABILITY LIMIT STATES IN STEEL CONSTRUCTION

### Deformation-Related Limit-States

Excessive static deflections due to gravity or wind loads, creep or differential settlement, or construction tolerances may cause unsightly sagging of floors and roofs, cracking or other forms of distress to nonstructural components and attachments; racking of walls or door and window frames; and, in extreme cases, development of secondary forces. Deformations leading to cracking or leakage of the building envelope may lead to moisture penetration, one of the most pervasive of all serviceability problems.

The point at which structural deformations become sufficient to cause serviceability problems depends on the nature of the structure and its detailing, as well as on the perceptions of the occupants. A review of the literature indicates that deflection or drift limits in the range of 1/200 to 1/500 of the floor span or story height that designers customarily use appear to be useful as general indices of nonstructural damage or unsightliness.<sup>1,10</sup> These limits indeed seem to have protected against such unserviceability problems in most instances. In addition, they also serve to limit the development of large secondary forces in walls and columns.

### Motion-Related Limit-States

Structural motions arise from normal activities of the building occupants, from the operation of mechanical equipment within the building, from external traffic, or from windstorms and earthquakes. Structural vibrations of floors or of the building as a whole can cause occupant discomfort and alarm. Vibrations under conditions of normal use were not usually a problem when working stress methods were used to design conventional floor and framing systems. However, reductions in mass, stiffness and damping that result from using lighter structural systems have led to an increasing number of complaints about objectionable vibrations. Occupant activities of a rhythmic nature, such as dancing, jumping, exercising and cheering, cause essentially steady-state excitations with frequencies in the range of 2 to 6 Hz. Many modern long-span floor systems have fundamental frequencies in this range.<sup>1,2,3,16</sup>

Numerous studies have been conducted in efforts to re-

---

*Bruce Ellingwood is professor of civil engineering, Johns Hopkins University, Baltimore. Bruce Ellingwood T.R. Higgins Award, 1988.*

---

late characteristics and levels of structural motion to human comfort levels.<sup>8,11,13,17</sup> Despite the lack of control conditions, several trends have emerged from these studies that are useful in identifying thresholds of vibration perception and annoyance. First, structural acceleration is the best overall indicator of potential discomfort; over certain frequency ranges people also may be sensitive to velocity.<sup>2,5</sup> Acceleration is directly related to the forces on the human body. Second, larger amplitudes of transient vibrations that damp out within a few cycles are tolerated more readily than smaller amplitudes of steady-state vibration that persist for an extended period of time.<sup>8</sup> If the motion damps out within 10 cycles or less, the tolerable acceleration is higher by a factor of 10 or more than if the motion is continuous.<sup>5</sup> Third, the perception or tolerance of vibration depends on the activity of the building occupant and the presence of visual or auditory cues. Within

the frequency range of 1 to 8 Hz, the threshold of annoyance for continuous vibrations is approximately 0.005g to 0.01g (0.05 to 0.10 m/s<sup>2</sup>)<sup>1,2</sup>; g is acceleration due to gravity (9.81 m/s<sup>2</sup> or 386 ft/s<sup>2</sup>). People engaged in quiet activities tend to find continuous vibrations much above 0.005g objectionable. In contrast, the annoyance threshold for transient vibration of finished floors in which structural damping is about 6 percent of critical may be about 0.05g (0.5 m/s<sup>2</sup>).<sup>8</sup> Note that the levels of vibration necessary to cause structural damage are well above these levels.

### Critical Appraisal of Current Serviceability Guidelines

Current serviceability criteria for steel beams and girders supporting floors and roofs take basically two forms.<sup>18</sup> The first involves limiting elastically computed static vertical deflection to some fraction of span  $\ell$ . For flexural members that support plaster ceilings, this deflection limit is  $\ell/360$  (this provision dates back to the early 19th century); for other members, the customary limit usually is  $\ell/240$  to  $\ell/180$ , depending on the function of the elements supported by the member. The second establishes a minimum depth for flexural members with respect to their span. For fully stressed beams supporting large open floor areas, it is recommended that the member span-depth ratio  $\ell/d$  not exceed  $800/F_y$ . In addition,  $\ell/d$  should not exceed 20; this limit has been used since the early 20th century as a vibration control measure.

Multistory buildings often are designed using non-mandatory guidelines on maximum drift (lateral displacement) under wind effects. The normally accepted range of overall drift limits for buildings appears to be 1/600 to 1/300 times the building height  $H$  with  $H/400$  being typical.<sup>17</sup> In addition, limits on interstory drift of 1/500 to 1/200 times the story height  $h$  are common.

The existing serviceability guidelines on maximum deflections, lateral drifts and span-depth ratios are aimed at controlling the stiffness of the structure. However, these

limits are tied very loosely to the functional requirements of the building. The current deflection limit of  $\ell/360$  for vertical deflection under live load is applied without regard to occupancy, even though performance requirements can vary widely for different occupancies. Moreover, by focusing on stiffness, other serviceability issues are addressed only indirectly. Although stiffness is a key serviceability parameter, mass and damping are also important in controlling vibrations.<sup>7</sup>

Consider, for example, the dynamic response of a simple steel beam. The flexural rigidity  $EI$  of the beam can be related to its fundamental natural frequency of vibration  $n_1$  (in Hz) by,

$$n_1 = \frac{\pi}{2\ell^2} \sqrt{\frac{EI}{\rho}} \quad (1)$$

in which  $\rho = (w_D + pw_L)/g$  is the mass of the beam and its tributary loads,  $w_D$  and  $w_L$  are dead and live load per length, and  $p$  is the fraction of the live load that participates in the dynamic motion. For general and clerical offices, the mean live load acting on the floor at any instant of time is about 12 psf (527 Pa), while the nominal design live load is between 25 and 50 psf (1197–2394 Pa), depending on the loaded area. Thus,  $p$  might typically be about 0.25. If the beam meets its static deflection limit  $\ell/360$  under full live load  $w_L$  its fundamental frequency  $n_1$  must be,

$$n_1 \geq 3.4 \sqrt{\frac{w_L/w_D}{1 + pw_L/w_D} \frac{g}{\ell}} \quad (2)$$

For a typical  $w_L/w_D$  of 2 for steel structures,<sup>9</sup> Eq. 2 becomes  $n_1 \geq 22/\sqrt{\ell}$ , where  $\ell$  is measured in feet. If  $\ell = 26$  ft (8 m),  $n_1 \geq 4.3$ Hz, while if  $\ell = 52$  ft (16 m),  $n_1 \geq 3.1$ Hz.

These natural frequencies are close to frequencies associated with several common human activities. Thus, current deflection limits might not be adequate to control objectionable vibrations for spans much in excess of 30 ft (9 m). Such spans are likely to be found in offices and public assembly occupancies, where there also are large open areas without floor coverings or partitions to provide damping.<sup>1,2</sup>

## CRITERIA FOR STATIC DEFORMATION-RELATED LIMIT STATES

### Serviceability Load Models

Probabilistic load models are necessary to evaluate serviceability criteria from a similar reliability viewpoint as that used for LRFD. Poisson pulse processes have been used to model the occurrence, duration and intensity of many structural loads. These models usually are sufficient for static loads, and allow their complex time variation to be represented as a sequence of random variables. Table 1 summarizes statistical data of equivalent static loads ob-

tained from analyses of load processes for reference periods of 1 and 50 years.<sup>10</sup> The 50-year data were used to develop the load requirements in LRFD.<sup>4,15</sup> The 1-year statistics might be used to develop load combinations rules for checking static deflections.

Table 1. Typical Statistical Data for Common Structural Loads			
(1a) T = 1 year			
Load	Mean/Nominal	COV*	PDF*
Dead, D	1.05	0.10	Normal
Live, L	0.45	0.40	Gamma
Snow, S	0.20	0.85	Lognormal
Wind, W	0.33	0.60	Type I
(1b) T = 50 years			
Load	Mean/Nominal	COV*	PDF*
Dead, D	1.05	0.10	Normal
Live, L	1.00	0.25	Type I
Snow, S	0.82	0.26	Type II
Wind, W	0.78	0.37	Type I
*COV = coefficient of variation PDF = probability distribution function			

### Analysis of Existing Static Deflection Criteria

The static deflection limit state for a uniformly loaded steel beam is,

$$g = \Delta_a - (K\ell^4/EI) L \leq 0 \quad (3)$$

in which  $\Delta_a$  is allowable deflection corresponding to some limit state,  $L$  is uniform live load, and  $K$  is a constant that depends on the end conditions and spacing of the beams.<sup>10</sup> A reliability index  $\beta$  is defined as  $\beta = m_g/\sigma_g$ , in which  $m_g$  and  $\sigma_g$  are the mean and standard deviation of  $g$ ;  $\beta$  can be related to the limit state probability,  $P_f$ , by  $P_f = \Phi(-\beta)$ , in which  $\Phi$  is the standard normal probability integral tabulated in statistical textbooks. It is assumed that the deflection limit is deterministic (this assumption may be invalid if the limit is related to unsightly sagging or some other state that depends primarily on the perceptions of the occupants) and that the variability in  $EI$  is negligible in comparison with variability in  $L$ . The customary deflection check would be,

$$(K\ell^4/EI) L_n \leq \Delta_a \quad (4)$$

in which  $L_n$  is the nominal A58 live load.<sup>4</sup> Eqs. 3 and 4 yield the following expression for reliability index  $\beta$ :

$$\beta \geq [(m_L/L_n)^{-1} - 1]/V_L \quad (5)$$

The mean/nominal  $m_L/L_n$  and coefficient of variation  $V_L$  can be taken from Table 1. The equality holds if the beam

deflects precisely to  $\Delta_a$  under  $L_n$ . A similar result is obtained for flexural members supporting roof loads due to snow.

The reliability indices for beams loaded by live loads are 3.1 and 0.0 when evaluated on a 1-year and 50-year basis, respectively. The first number is approximately the same as the reliability index in LRFD for the ultimate limit state of yielding of tension members.<sup>9</sup> The value zero implies that there is approximately a 50-percent chance that the deflection limit will be exceeded at least once during a 50-year period of occupancy. Similarly, for beams loaded by snow loads, the reliability indices are 5.3 and 0.8, respectively, for periods of 1 year and 50 years. Thus, there appears to be a disparity in current serviceability checks for beams subjected to live or snow loads.

The lateral drift of a steel frame due to wind generally is determined in current practice using unfactored nominal wind loads.<sup>4</sup> The variabilities in response due to the flexibility of the frame can be assumed to be negligible in comparison with the variability in the wind load itself.<sup>10</sup> Thus, the reliability index can be computed from a knowledge of  $m_w/W_n$  and  $V_w$  from Table 1 using Eq. 5. The corresponding values of  $\beta$  are 3.4 and 0.8 for reference periods of 1 year and 50 years, respectively. The inclusion of second-order ( $P-\Delta$ ) effects does not have a significant effect on these results.<sup>10</sup>

These measures of reliability provide benchmarks for the development of new load combinations for serviceability-checking along the lines of those developed for LRFD. It is interesting to note that the  $\beta$ -values for unserviceability on an annual basis are comparable to the  $\beta$ -values for ultimate limit states in LRFD on a 50 year basis.<sup>9</sup>

### Probability-Based Loading Criteria

Deformation or damage-related limit states occurring in steel structures and under static loads can be checked using load combinations that are similar in format to the load combinations used to check strength limit-states in LRFD. The nature of the limit state would determine the appropriate combination.<sup>1,10</sup> Serviceability checks might reasonably involve a total load effect with a 10 percent probability of being exceeded in any given year. For example, in evaluating unsightly sag or local cracking of walls or partitions, the corresponding load combinations might be,

$$D_n + 0.8L_n \quad (6a)$$

$$D_n + 0.4L_n + 0.5W_n \quad (6b)$$

Not that load combinations with load factors less than unity might be appropriate for checking serviceability-limit-states. The nominal loads in the A58 Standard used for checking ultimate-limit-states have a probability of less

than 0.02 of being exceeded in a given year. Such loads may be unduly conservative for checking serviceability-limit-states, which are less severe.

### CRITERIA FOR MOTION-RELATED LIMIT STATES

Objectionable vibrations of modern floor systems or of buildings as a whole cannot be controlled through the use of conventional load combinations such as those in Eqs. 6 and deflection or drift limits. Criteria for motion-related limit states must reflect the time-dependent nature of the excitation and the dynamic response characteristics of the structural system.

### Analysis of Floor Vibrations

The main sources of floor vibration are activities of human occupants, including walking, dancing, jumping and other rhythmic activities, and events that give rise to impacts. The excitations imparted by these activities may be steady-state or transient in nature. Realistic models of forces in the time or frequency domains are needed for establishing general serviceability criteria for floor systems.<sup>7</sup> Some of these models can be complex for even simple activities such as pedestrian motion. Consider, for example, the force-time relation for a single footfall during normal walking, shown in Fig. 1a.<sup>7</sup> The first and second peaks correspond to heel strike and toe liftoff contact. A Fourier analysis of this pulse reveals that its first and third harmonics are the major contributors to dynamic response. As the gait increases from a walk to a run, the peaks in Fig. 1a merge together and the peak forces approach roughly 2.75 times the body weight.

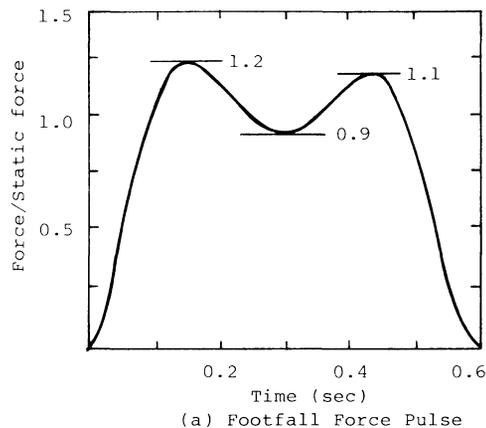


Fig. 1. Forces Due to Pedestrian Movement

The excitation of a floor system by one person or a group of people walking in step can be analyzed as shown in Fig. 1b. The predicted response to pedestrian movement of a floor system with a simple span of 26 ft (8 m), fundamental frequency of 5 Hz, and damping 3% of critical is illustrated in Fig. 2.<sup>7</sup> The static as well as dynamic

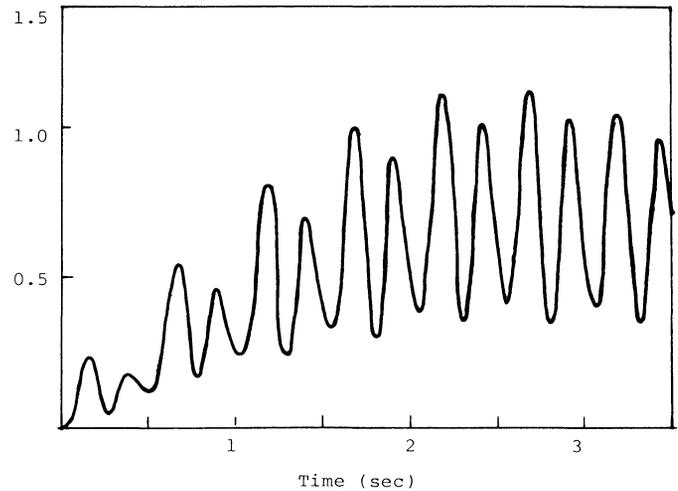
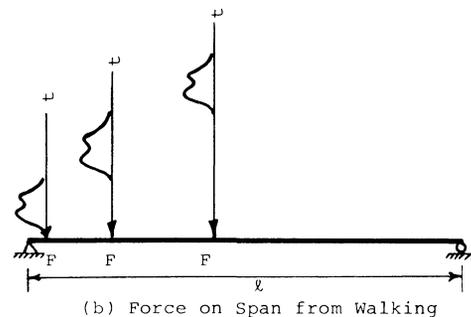


Fig. 2. Response of Floor to Pedestrian Movement ( $n_1 = 5$  Hz)

components of deflection are clearly evident. The dynamic response approaches a steady-state condition, with a maximum acceleration of 0.026g. Figure 3 illustrates the response of a similar 26-ft floor system in which the fundamental frequency has been increased to 10 Hz by increasing the stiffness. Here, the computed peak accelerations are approximately 0.006g, which borders on the



perception threshold.<sup>5</sup> The displacement amplitudes are nearly the same for both damping ratios of 3% and 12%. Increasing the damping in this floor would have little effect on the essentially steady-state vibrations arising from pedestrian motion.

The accelerations from an impact test such as a "heeldrop" at midspan are from 8 to 100 times larger than the accelerations computed above.<sup>7</sup> The range in these accelerations, when plotted on a vibration-tolerance chart,<sup>2,5</sup> is sufficient to cause an acceptable floor system to be rated as unacceptable. Thus, the use of realistic force functions is important in assessing the sensitivity of floors to dynamic motion.

Sinusoidal loading functions may be acceptable as models of some rhythmic activities, e.g., dancing or jogging in place, where the spatial variation in the force is negligible in comparison to its temporal variation.<sup>2,3</sup> Figure 4 illustrates a sample force function for several common activities. The factor  $\alpha$  by which the static force is multiplied to obtain the dynamic component, depends on the activity.

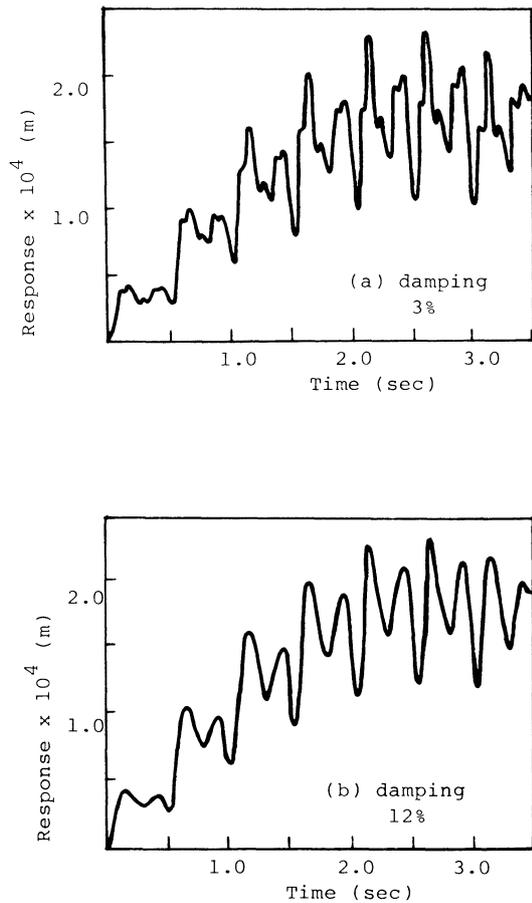


Fig. 3. Response of Floor to Pedestrian Movement ( $n_1 = 10$  Hz)

### Criteria for Floor Vibration

To ensure that building occupants are not unduly disturbed by floor movement under normal conditions, the acceleration of the floor should be limited to some tolerance threshold. Tolerance levels should be related to the primary occupancy of the building.<sup>1,7</sup> The different needs of the following occupancy groups should be recognized: (1) residential and institutional; (2) offices; (3) commercial; and (4) public assembly. Table 1 of Ref. 7 offers guidelines on acceleration limits for steady state and transient vibrations for several of the general occupancy classifications in the A58 Standard.

If the excitation of the floor is essentially sinusoidal due to rhythmic activity with frequency,  $n$ , the maximum acceleration at midspan for a lightly damped system is approximately,

$$\ddot{x}_{max} \approx (2\pi n)^2 (\alpha F_s / k) [1 - (n/n_1)^2]^{-1} \quad (7)$$

in which  $n_1$  is fundamental natural frequency of the floor. The term  $(\alpha F_s / k)$  is simply the static deflection due to the dynamic force. Substituting the maximum permissible acceleration into Eq. 7, one can develop requirements on minimum allowable fundamental frequency of vibration or on maximum static deflection. For example, one might require that the fundamental frequency of vibration  $n_1$  of floor systems supporting rhythmic activities should not be less than,<sup>3,7</sup>

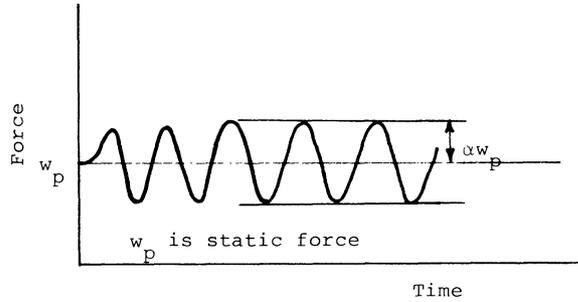
$$n_1 \geq n \left[ 1 + \frac{1.3}{a_{max}} \frac{\alpha w_p}{w_T} \right]^{1/2} \quad (8)$$

in which  $a_{max}$  is the limiting steady-state rms acceleration for the occupancy, expressed as a fraction of the acceleration due to gravity;  $w_T$  is the total unit weight (force/area) of floor plus personnel;  $w_p$  is the static unit weight of personnel on the floor; and  $\alpha w_p$  is the dynamic component of unit personnel load modeled as an equivalent sinusoidal excitation, as shown in Fig. 4.

Similar criteria can be developed for limiting floor vibration due to transient or impact-type events.<sup>7,16</sup> These criteria are dependent on the damping provided by the floor system. Research on the performance of light steel joist-slab floor systems during impact tests has demonstrated the importance of damping in controlling objectionable motion in floors subjected to transient excitations.<sup>16</sup>

### Building Vibrations due to Wind

Fluctuating wind forces can cause excessive motion of tall buildings.<sup>1,11,13,17,19</sup> Traditional static lateral-drift checks are not adequate to prevent this limit state. Although aerodynamic effects of wind forces on buildings are complex, relatively simple motion-sensitivity checking procedures can be devised for use in lieu of static-drift checks; these also may identify cases where a more de-



Activity for Group	$\alpha$	f (Hz)	$a_{\max}$ (g)
Dancing	0.25 - 0.5	1.5 - 3	0.02
Walking in place	0.25	1.5 - 3	0.02
Running or jumping in place	1.5 - 2	1.5 - 3	0.05
Cheering at sporting event or concert	0.25	1.5 - 3	0.05

Fig. 4. Forces Caused by Rhythmic Activities

tailed investigation involving wind tunnel testing might be warranted.<sup>19</sup> Recent research has shown that the acrosswind and torsional wind forces are more likely to be the source of objectionable building vibrations than the alongwind force. The best overall indicator of whether the motion of a building during a windstorm is objectionable appears to be its rms acceleration.

The rms acceleration of a building subjected to fluctuating wind forces can be computed by random vibration theory.<sup>14</sup> Assume, for illustration, that the building is modeled as a lightly damped linear system with three degrees of freedom (alongwind, acrosswind, and torsion) and that the centers of mass and rigidity of the building coincide with its geometric center. The fluctuating components of force and response are modeled as zero-mean stationary random processes. The meansquare acceleration in each mode is<sup>19</sup>,

$$a_j^2 = \int_{-\infty}^{\infty} |H_j(\omega)|^2 S_{F_j}(\omega) d\omega; j = 1,2,3 \quad (9)$$

in which  $S_{F_j}(\omega)$  is the power spectral density of the  $j$ th generalized force  $F_j(t)$ ;  $H_j(\omega)$  is the complex admittance-function relating force to acceleration in the frequency domain;

$$H_j(\omega) = \omega^2 m_j^{-1} [\omega_j^2 - \omega^2 + 2i\zeta_j \omega_j \omega]^{-1} \quad (10)$$

and parameters  $\omega_j$ ,  $m_j$  and  $\zeta_j$  are, respectively, the natural frequency (radians/sec), generalized mass and damping ratio. For lightly damped systems, Eq. 9 can be approximated by<sup>19</sup>,

$$a_j^2 \approx \pi \omega_j S_{F_j}(\omega_j) / (2\zeta_j m_j^2) \approx \pi S_{F_j}(\omega_j) k_j^{1/2} m_j^{-5/2} \quad (11)$$

in which  $k_j$  is generalized stiffness. The total rms accelera-

tion of the building is equal to the square root of the sum of the  $a_j^2$ ,  $j = 1,2,3$ . Recent studies have shown that the across wind term dominates the overall rms acceleration.

Spectra for alongwind and acrosswind forces and torque are determined from an analysis of wind tunnel pressure measurements. The acrosswind force and torque depend on wake instabilities, which are difficult to derive analytically. Figure 5 illustrates spectra of generalized alongwind and acrosswind forces and torque acting on a tall building

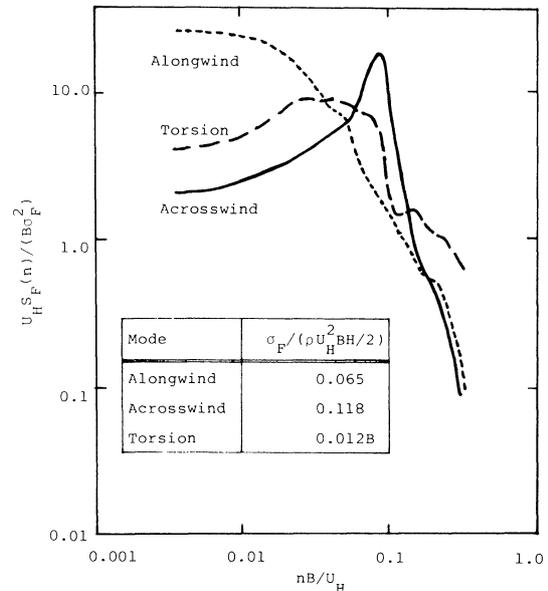


Fig. 5. Wind Force Spectra

with a square plan in an urban exposure. In Fig. 5,  $\sigma_F$  is rms generalized force,  $U_H$  is mean wind speed at the top of the building,  $B$  is building plan dimension,  $\rho$  is the mass density of air, and  $n$  is frequency in Hz. In applying Eq. 9 or 11 with the force spectra in Fig. 5, the frequencies and spectra must be transformed by  $\omega = 2\pi n$  and  $S_F(\omega) = S_F(n)/4\pi$ .

The above analysis is applied to a group of square buildings with urban exposures, height-to-width ratios ranging from 4:1 to 8:1, and mass densities of from 150 to 200 kg/m<sup>3</sup> (9 to 13 lb/ft<sup>3</sup>.) Such general characteristics are typical of modern tall steel building construction. The stiffness of each building is set by a traditional drift analysis in which the alongwind deflection at its top due to a statically applied uniform lateral wind load does not exceed some fraction of the building height, say  $H/400$ . The static wind load used in this drift check is computed using the wind profile and pressure coefficients from the A58 Standard<sup>4</sup> and a reference fastest-mile wind speed of 65 mph (29 m/s) at an elevation of 10 m.<sup>4</sup> This wind speed has an estimated mean recurrence interval of approximately 10 years at many sites in the continental United States, and might be selected as a basis for checking lateral drift.

The relation between acrosswind rms acceleration and the drift limit for this group of buildings is presented in Fig. 6. Although the rms acceleration decreases as the permissible drift index decreases, as one might expect, it is obvious that designing to a given drift limit, such as  $H/400$ , does not guarantee an acceptable rms acceleration level. The vibration of buildings falling above the hatched line (0.01g) would seem unacceptable to many occupants during a 10-year mean recurrence interval windstorm.

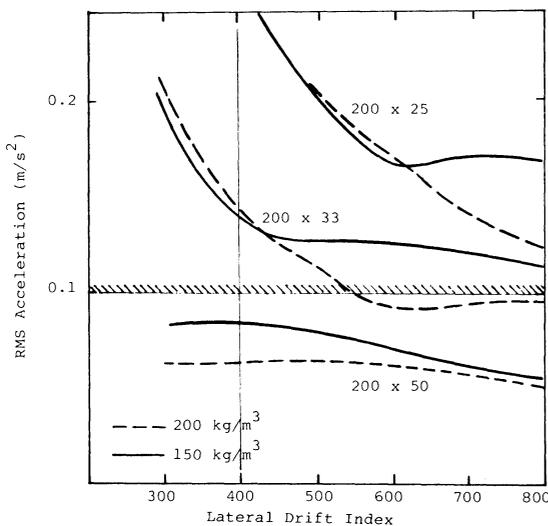


Fig. 6. Accelerations in Tall Buildings Due to Wind

As a general performance requirement, one might require that the rms acceleration due to a windstorm with a mean recurrence interval of 5 to 10 years should not exceed 0.01g (0.1 m/s<sup>2</sup>). The generalized mass, damping and frequency required to ensure that motion would not be objectionable then is determined from Eq. 11, as

$$n_1 \pi S_F(n_1) / (4\zeta m^2) \leq (0.1)^2 \quad (12)$$

in which  $S_F(n_1)$  is the spectrum of acrosswind force. Once the building site is selected and a preliminary structural design is established, the fundamental frequency  $n_1$  can be estimated, windspeed  $U_H$  can be determined, and  $n_1 B / U_H$  can be calculated. All parameters in Eq. 12 except  $S_F(n_1)$  can be defined by conventional structural analysis and design data;  $S_F(n_1)$  could be made available in a graph such as Fig. 5 or a table.

## CONCLUSION

Serviceability checks for deformation or damage-related limit states can be developed using load criteria that resemble the criteria used in LRFD for checking ultimate limit states. Floor systems and building frames can be checked for susceptibility to motion using criteria that reflect the general dynamic characteristics of the structural system. While these dynamic checks are not difficult to apply, they bear little resemblance to traditional approaches for checking serviceability. Serviceability guidelines ought to be flexible and adaptable to different occupancies and performance requirements. However, it seems desirable to have a few simple minimum criteria to which most buildings should conform.

Considerable research on serviceability problems has been conducted during the past decade in the United States, Canada, Western Europe, and Japan. It is apparent that practical serviceability criteria, which basically are consistent with LRFD criteria already in place, could be developed for steel structures. The LRFD Specification does not provide specific serviceability criteria; instead, it cautions engineers to give serviceability due consideration in design. Whether additional serviceability criteria should be included in the LRFD Specification is a question that depends on other than purely technical considerations.

## REFERENCES

1. An Hoc Committee on Serviceability (1986). "Structural Serviceability: A Critical Appraisal and Research Needs." *Journal of Structural Engineering*, 112(12):2646-2664 American Society of Civil Engineers.
2. Allen, D. E. and J. H. Rainer, (1980). "Vibration Criteria for Long-Span Floors." *Canadian Journal of Civil Engineering*, 3(2):165-173.

3. Allen, D. E., J. H. Rainer, and G. Pernica, (1985). "Vibration-Criteria for Assembly Occupancies." *Canadian Journal of Civil Engineering*, 12(3):617–623.
4. *American National Standard Minimum Design Loads for Buildings and Other Structures* (ANSI A58.1–1982). American National Standards Institute, New York.
5. *American National Standard Guide to the Evaluation of Human Exposure to Vibration in Buildings* (ANSI S3.29–1983). American National Standard Institute, New York.
6. Ellingwood, B., et al. (1982). "Probability-Based Load Criteria: Load Factors and Load Combinations." *Journal of Structural Division*, 108(5):978–997 American Society of Civil Engineers.
7. Ellingwood, B. and A. Tallin, (1984). "Structural Serviceability: Floor Vibrations." *Journal of Structural Engineering*, ASCE, 110(2):401–418.
8. Galambos, T.V., et al., (1973). "Structural Deflections: A Literature and State-of-the-art Survey." *Building Science Series No. 47*, National Bureau of Standards, Washington, DC.
9. Galambos, T. V., et al., (1982). "Probability-Based Load Criteria: Assessment of Current Design Practice." *Journal of Structural Division*, 108(5):959–977 American Society of Civil Engineers.
10. Galambos, T. V. and B. Ellingwood, (1986). "Serviceability Limit States: Deflection." *Journal of Structural Engineering*, 112(1):67–84 American Society of Civil Engineering.
11. Goto, T. (1983). "Studies on Wind-Induced Motion of Tall Buildings Based on Occupants' Reactions." *Journal of Wind Engineering (Industrial Aerodyn)* 13:241–252).
12. *International Organization for Standardization Bases for the Design of Structures—Deformation of Buildings at the Serviceability Limit States* (ISO Standard 4356), Geneva, Switzerland, 1977.
13. Hansen, R. J., J. Reed, and E. Vanmarcke, (1973). "Human Response to Wind-Induced Motion of Buildings." *Journal of Structural Division*, 99(7):1589–1605 American Society of Civil Engineers.
14. Lin, Y. K. (1967). *Probabilistic Theory of Structural Dynamics*. McGraw-Hill, Inc., New York.
15. *Load and Resistance Factor Design Specification for Structural Steel Buildings* (1986). American Institute of Steel Construction, Chicago.
16. Murray, T. M. (1981). "Acceptability Criterion for Occupant-Induced Floor Vibrations." *Engineering Journal* 18(1):62–70 American Society of Civil Engineers.
17. *Planning and Design of Tall Buildings* (1980). A Monograph in Five Volumes, prepared by the Council on Tall Buildings, American Society of Civil Engineers, New York.
18. *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings*. American Institute of Steel Construction, Chicago, 1978.
19. Tallin, A. and B. Ellingwood, (1984). "Serviceability Limit States: Wind-Induced Vibrations." *Journal of Structural Engineering*, 110(10):2424–2437 American Society of Civil Engineers.