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Floor Vibrations: A New Design Approach

Nouvelle façon d'étudier la vibration des planchers

Eine neue Lösung für Deckenschwingungen

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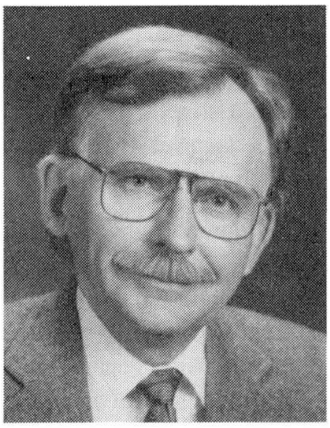

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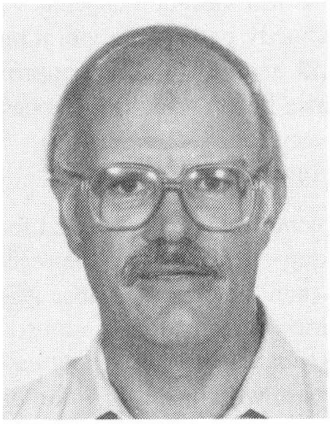

David Allen, born 1934, obtained a Ph.D. from University of Illinois in 1966 and has ben a researcher with the National Research Council Canada ever since. He has authored about sixty papers in diverse subjects related to structural engineering, including safety criteria human error, fire resistance and floor vibration.

SUMMARY

A new criterion for walking vibrations with broader application is discussed. The criterion is based on the dynamic response of steel structures to walking forces. The criterion can be applied to offices, shopping malls and footbridges.

RESUME

Un nouveau critère, ayant un plus vaste domaine d'application, pour l'étude de la vibration des planchers sous l'effet des piétons, est présenté. Le critère est basé sur l'étude de la réaction dynamique des structures métalliques quand elles sont soumises à des forces causées par la marche des piétons. Le critère peut être utilisé pour l'étude de la vibration des passerelles et des planchers dans les bureaux et dans les centres commerciaux.

ZUSAMMENFASSUNG

Ein neues Kriterium für Schrittschwingungen, mit breiter Anwendung, wird vorgelegt. Es basiert auf der dynamischen Reaktion von Stahlkonstruktionen auf Schrittkräfte. Das Kriterium kann auf Geschäftsgebäude, Einkaufszentren und Fussgängerbrücken angewendet werden.

1. INTRODUCTION

Existing North American floor vibration design criteria are usually based on a reference impact, such as a heel-drop, and were calibrated using floors constructed 20-25 years ago. Annoying floors of this vintage generally had natural frequencies between 5 and 8 hz because of the then existing design rules and common construction practice. With the advent of LRFD and the more common use of lightweight concrete, floor systems have become lighter, resulting in higher natural frequencies for the same structural steel layout. Beam and girder spans, however, have increased resulting in frequencies lower than 5 hz. Most existing design criteria do not properly evaluate systems with frequencies below 5 hz and above 8 hz.

A new criterion for walking vibrations with broader applications has recently been proposed [1]. The criterion is based on the dynamic response of steel beam and joist supported floor systems to walking forces. The criterion can be used to evaluate structural systems supporting offices, shopping malls and footbridges. This paper provides an overview of the proposed criterion.

The reaction of people who feel vibration depends very strongly on what they are doing. People in offices or residences do not like distinctly perceptible vibration (about 0.5 percent g), whereas people taking part in a non-stationary activity will accept vibrations approximately 10 times greater (about 5 percent g or more). People dining beside a dance floor, lifting weights beside an aerobics gym, or standing in a shopping mall, will accept something in between (about 2 percent g). Sensitivity within each occupancy, however, varies with duration of vibration and remoteness of source.

Most floor vibration problems are due to repeated forces caused by machinery or by human activities such as dancing, aerobics or walking. In some cases the repeated force is sinusoidal, or nearly so, although walking is a little more complicated than the others because it is not a stationary force. A repeated stationary force can be represented by a Fourier combination of sinusoidal forces with forcing frequencies equal to a multiple (harmonic) of the basic frequency of force repetition (step frequency for human activities). As a general rule, the sinusoidal forces decrease with increasing harmonic, more so if a large number of people are involved. If any of the harmonic forces correspond with the natural frequency of a susceptible vibration mode, then resonance will occur. Walking can excite resonance for more than one harmonic, but experience shows that a consideration of resonance with a lower mode is sufficient for design.

2. OVERVIEW OF PROPOSED CRITERION

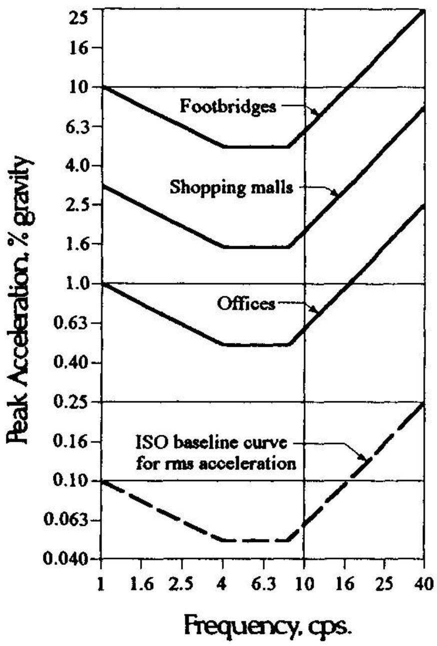

Figure 1 Proposed Acceleration Limits for Walking Vibrations

The proposed criterion was developed using the following:

- Acceleration limits as recommended by the International Standards Association (2) adjusted for intended use. The ISO suggests limits in terms of rms acceleration as a multiple of the baseline line curve shown in Figure 1. The multipliers in the proposed criterion are 10 for offices, 30 for shopping malls, and 100 for footbridges.
- A time dependent loading function repesented by the Fourier series

$$F = P(1 + \Sigma\alpha_i \text{Cos}2\pi ift) \qquad (1)$$

where P is the person's weight, taken as 0.7 kN for design, f the step frequency, i the harmonic multiple, and α_i the dynamic load factor for the harmonic. Proposed values for α_i are given in Table 1.

- A response function of the form:

$$\frac{a}{g} = \frac{R\alpha_i P}{\beta W} \bullet \text{Cos}2\pi ift \qquad (2)$$

Harmonic i	Frequency Range $i \bullet f$	Dynamic Load Factor α_i
1	1.6 to 2.2	0.5
2	3.2 to 4.4	0.2
3	4.8 to 6.6	0.1
4	6.4 to 8.8	0.05

Table 1 Dynamic Load Factors

where W is the total weight supported by the beam, β is the damping ratio, g is the acceleration due to gravity, and R is a reduction factor. The reduction factor R takes into account the fact that full steady-state resonance is not achieved for walking and that the walking person and the person annoyed are not simultaneously at the location of maximum modal displacement. It is proposed that R be taken as 0.7 for footbridges and 0.5 for floor structures having two-way modal configurations.

The proposed criterion is obtained from Eqns. (1) and (2) expressed as a minimum value of damping ratio times equivalent mass weight

$$\beta W \geq \frac{R \alpha_i P}{a_o / g} \tag{3}$$

As shown in Reference [1], Inequality [3] can be approximated as

$$\beta W \geq K \exp(-0.35 f_o) \tag{4}$$

where f_o is the fundamental natural frequency (Hz) and K is a constant which depends on the acceleration limit for the occupancy: 58 kN for offices, 20 kN for shopping malls and 8 kN for footbridges. In terms of minimum fundamental frequency, Inequality [4] is

$$f_o \geq 2.86 \ln\left[\frac{K}{\beta W}\right] \tag{5}$$

Inequality (5) is then the proposed criterion for floor vibrations due to walking. That is, an acceptable floor is one with a natural frequency greater than the right side of Inequality (5).

The following section provides guidance for estimating the required floor properties for application of the proposed criterion.

3. ESTIMATION OF REQUIRED PROPERTIES

Recommended values for the damping ratio, β, are 0.03 for offices, 0.02 for shopping malls, and 0.01 for footbridges. If full height partitions are connected at the top and at the bottom to the structure, the damping ratio for offices can be increased to 0.05. A value of 0.02 should be used if few non-structural components (ceiling, ducts, partitions, etc.) are supported by an office floor. These values are modal damping ratios and are approximately one-half of previously recommended damping values which were based on vibration decay resulting from a heel-drop impact [3,4].

The fundamental natural frequency, f_o, and equivalent mass weight, W, for a critical mode are estimated by first considering the 'joist panel' and 'girder panel' modes separately and then combining them. If the joist span is less than one-half the girder span, both the joist panel mode and the combined mode should be checked separately. (For the purposes of this paper, a "joist" is a structural member supported by a girder; a girder is a structural member supported by a column or wall.)

The first natural frequency for the joist and girder panel modes can be estimated using

$$f_o = 0.18 \sqrt{g / \Delta} \tag{6}$$

where Δ is the maximum deflection of the joist or girder due to the weight supported by the member. Composite action is normally assumed, provided there is sufficient shear connection between the slab/deck and the member. Joists and girders are assumed to be simply supported unless dynamic restraint is verified by a dynamic analysis or experiment. It is recommended that the concrete modulus be taken equal to 1.2 times that assumed in current structural standards to account for the increase in stiffness of concrete under dynamic loads. Also for determining the composite moment of inertia, the width of the concrete slab is equal to the sum of one-half the distances to the adjacent members, but each distance is not to exceed one-eighth of the span.

For the combined mode, the fundamental natural frequency can be approximated by the Dunkerly relationship

$$f_o = 0.18\sqrt{g/(\Delta_j+\Delta_g)} \qquad (7)$$

where Δ_j and Δ_g are the joist and girder deflections, respectively.

The equivalent mass weight for the joist and girder panel modes are estimated from

$$W = w\,B\,L \qquad (8)$$

where w is the supported weight per unit area, L is the member span and B is the effective width determined from

$$B_j = 2\,(D_s/D_j)^{1/4}\,L_j \qquad (9a)$$

for the joist panel mode and from

$$B_g = 1.7\,(D_j/D_g)^{1/4}\,L_g \qquad (9b)$$

for the girder panel mode, where D_s, D_j and D_g are the flexural rigidities per unit width in the slab, joist and girder directions, respectively. The following limitations and requirements apply:

1. The joist effective panel width, B_j, should not be taken greater than two-thirds of the total floor width perpendicular to the joists.
2. If the joist is connected to the girder by a single pin-type connection, the factor 1.7 in Eqn. [9b] should be reduced to 1.4. This requirement does not apply to rolled joists which are shear-connected to girder webs.
3. Where joists or girders are continuous over their supports and an adjacent span is greater than 0.7 times the joist or girder span, respectively, the effective joist or girder weight can be increased by 50%. This requirement applies to rolled sections shear-connected to girder webs.

For the combined mode, the equivalent mass weight can be approximated from

$$W = \frac{\Delta j}{\Delta j + \Delta g} W_j + \frac{\Delta g}{\Delta j + \Delta g} W_g \qquad (10)$$

If the girder span, L_g, is less than the joist panel width, B_j, the combined mode is restricted and the system is effectively stiffened. This can be accounted for by reducing the deflections used in Eqns. (7) and (10) to

$$\Delta_g = \frac{L_g}{B_j}(\Delta_g) \qquad (11)$$

where $0.5 \le L_g/B_j \le 1.0$

4. APPLICATION

Application of the proposed criterion requires careful consideration by the structural engineer. For instance, the acceleration limits for footbridges are meant for situations with many walking pedestrians and not for quiet areas like crossovers in hotel atria. For the later case it is suggested that the office floor acceleration limits be used.

Designers of footbridges are cautioned to pay particular attention to the location of the concrete slab. The first writer is aware of a situation where the designer apparently "eye-balled" his design based on previous experience with floor systems. In this case, the concrete slab was located between the beams at mid-depth (because of clearance considerations) and the footbridge vibrated at a much lower frequency and at a larger amplitude than anticipated because of the reduced transformed moment of inertia. The result was a very unhappy owner and an expensive retrofit.

Unsupported floor edges, as in many mezzanine areas, are also a special consideration because they are often lightly loaded and possess little damping. In this instance, the edge members will be made stiffer by use of the following. Where the edge member is a joist, the equivalent mass weight of the joist panel can be estimated using Eqn. (8) by replacing the coefficient 2 in Eqn. (9a) with 1. Where the edge member is a girder, the equivalent mass weight of the girder panel is the tributary weight supported by the girder. The edge panels are then combined with these orthogonal panels using Eqns. (7) and (10).

A type of light-truss joist used in North America is supported at the ends with a shoe or seat as shown in Fig. 2. This support detail affects the response of both the joist panel and the girder panel. For the joist panel, the coefficient 1.7 in Eqn. (9b) should be reduced to 1.4. For the girder panel where the concrete is separated from the concrete slab, the shoes or seats may act as Vierendal girders causing partial composite action. It is recommended that the moment of inertia of girders supporting joist seats be determined from:

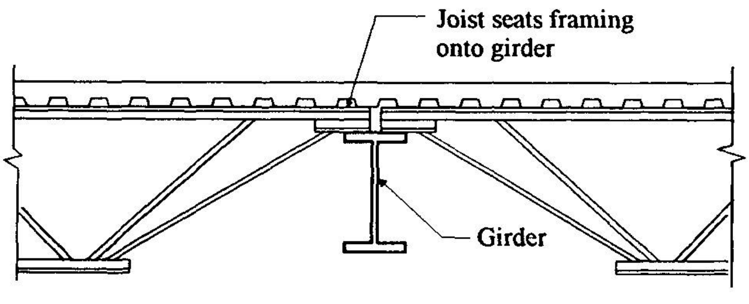

Figure 2 Light Truss Support

$$I_g = I_{nc} + (I_c - I_{nc})/2 \qquad (12a)$$

for joist seat heights 75 mm or less, and

$$I_g = I_{nc} + (I_c - I_{nc})/4 \qquad (12b)$$

for seats heights 100 mm or more, where I_{nc} and I_c are non-composite and fully composite moment of inertia, respectively.

If the bottom chord of a light truss is extended and attached to the girder, the coefficient 1.7 in Eqn. (9b) applies, since continuity is achieved. However, the fundamental frequency, f_o, does not change if the adjacent spans are approximately the same length. From Inequality (5), it is seen that an increase in the mass weight, W, results in a lower required fundamental natural frequency. This fact suggests that, if a seated joist supported floor system (as shown in Figure 2) is not satisfactory, extending the bottom chords may be an effective remedial measure.

When the natural frequency of a panel is greater than 9 hz, harmonic resonance does not occur, but walking vibration can still be annoying. Experience indicates a minimum stiffness of 1 kN per mm is required for office occupancies. To ensure satisfactory performance of office floors with frequencies greater than 9 hz, the design should use this stiffness criterion in addition to Inequality (5) when evaluating a proposed floor system.

Occasionally, a floor system will be judged particularly annoying because of what feels to be motion transverse to the supporting joists. In these situations, when the floor is impacted at one location there is a perception that a "wave" moves from the impact location in a direction transverse to the supporting joists. The first writer has observed this phenomenon and felt the "wave" 15-20 m from the impact location perhaps up to 1 second after the impact. In at least one instance, the "wave" rebounded from the exterior wall and was felt at the impact location. This phenomenon occurred in a rectangular building where the floor was free of partitions and all joists were equally spaced and of the same stiffness, including those at the column lines. The resulting motion is very annoying to occupants because the floor moves without apparent reason as the cause is not within sight or hearing. The proposed criterion does not address this phenomenon but a small change in the structural system will eliminate the problem. If one joist stiffness or spacing is changed periodically, say every third bay, the "wave" is interrupted at that location and floor motion is much less annoying.

5. CONCLUSIONS

A relatively simple criterion for the control of vibration of steel-framed floor systems for walking is proposed. Recommended values for the criterion parameters are suggested, but are expected to be improved with further experience. The proposed criterion can be applied to a number of special situations.

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