Tips for Avoiding Office Building Floor Vibrations

by Thomas M. Murray, Ph.D., P.E.

More and more frequently, engineers who have never encountered a floor vibration problem with their designs are finding themselves searching for fixes. I have been involved with the floor vibration serviceability for over 30 years. For the first 28 years, I heard of perhaps one problem floor a year; now it's one a month and sometimes more. Why?

First, the reason is not LRFD. Yes, LRFD results in lighter floors, especially if they are composite, but nearly identical designs can be obtained with ASD. The reasons are longer beam spans (40 ft today versus 25 ft a decade ago), less actual live loading (11 psf versus perhaps 25 psf), and less damping because of open office areas and less paper, heavy desks, and file cabinets. The bottom line is that floor vibration serviceability can control the design. We may have to accept the fact the may need to add steel to control the problem just like we do to control drift in high-rise buildings.

Following are ten tips to help the designer with the floor vibration problem. I assuming the reader is somewhat familiar with floor vibration analysis, if not, I recommend a study of the AISC/CISC *Design Guide 11 Floor Vibrations due to Human Activity*.

Tip 1: Don't blame vibration problems on LRFD–it's not the cause of serviceability problems.

Sure, LRFD results in lighter floor systems, especially if composite construction is used. Sure, the profession has a hang-up about LRFD. Yes, composite systems rarely satisfy floor vibration criteria, but that's not the fault of LRFD. A stretched-to-the-limit ASD design will result in the same serviceability problems as LRFD. The designer needs to accept that 50 ksi steels, higher strength concrete, optimized computer-based designs, longer spans and much lighter actual live loads result in lively floors (as the British say), and therefore require a little more design time. Better yet, think of it as the need for a little art in your floor designs.

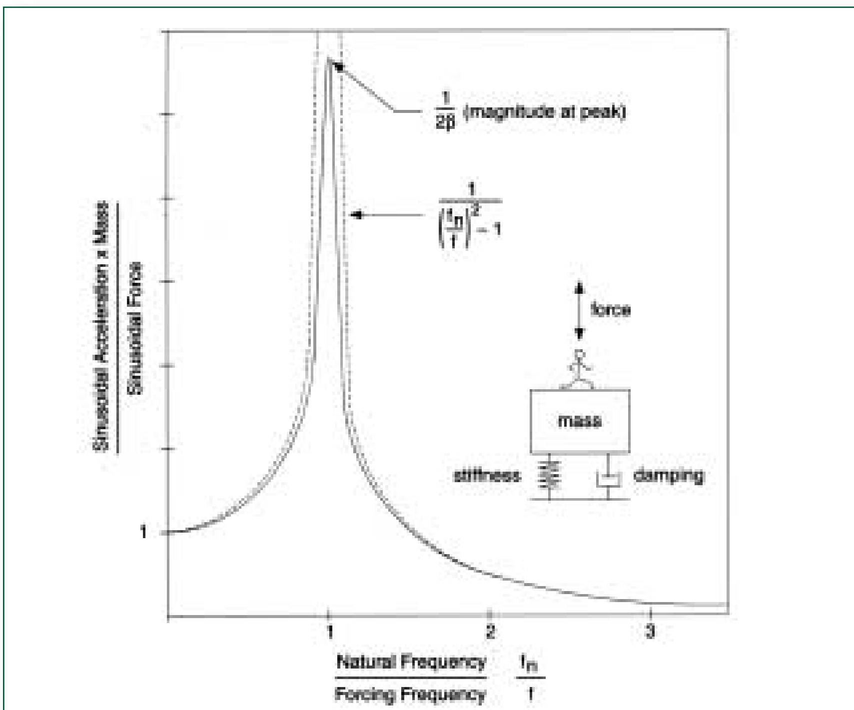

Figure 1. Resonance response (Figure 1.3 of the AISC/CISC design guide).

Tip 2: Use the AISC/CISC design guide criteria.

Study the American Institute of Steel Construction and the Canadian Institute of Steel Construction's *Design Guide 11: Floor Vibrations Due to Human Activity*. Unlike older publications such as *Modified Reiher-Meister Scale*, *Murray Criterion* and the *Canadian Standards Association Rule* that use a heel-drop impact, the new criteria are based on resonance with walking.

Resonance can occur when the exciting frequency (rate of walking) or a multiple of that frequency (harmonic) equals the natural frequency of the floor system. Resonance results in very large amplitudes of displacement, velocity or acceleration, as seen in Figure 1. The criteria ensure that resonance does not occur for the first three harmonics associated with walking. That is, if a person is walking at 2 steps per second (2 Hz), the floor system is checked for resonance at 2, 4 and 6 Hz.

The design guide criteria state that a floor is satisfactory if the following inequality is satisfied:

$$\frac{a_p}{g} = \frac{P_o \exp(-0.35 f_n)}{\beta W} < \frac{a_o}{g}$$

where a_p/g is the predicted peak acceleration of the floor due to walking as a function of gravity, a_o/g is the tolerance acceleration for the environment, P_o is a constant force representing the excitation, f_n is the natural frequency of the floor system, β is the modal damping in the floor system and W is the effective weight that moves because of the excitation. At first glance, the criteria might look a bit formidable—it certainly is different than the older criteria. In reality, only f_n and W require calculations. P_o is 65 lb for office floors, a_o/g is $0.005g$ ($0.5\%g$) for office environments and β is a number between 0.01 and 0.05 (see Tip 3).

But why learn the new criteria? All floor vibration criteria have two parts: a prediction of the floor response and a human tolerance level. Furthermore, all criteria must be calibrated and thus are empirical in nature (the necessary fundamental studies of human response to low frequency/very low amplitude vertical vibration have not been done). The *Modified Reiher-Meister*, *Murray Criterion* and *Canadian Rule* were all calibrated using floors built at least 25 years ago. However, construction and the office environment have changed. Today, we use lighter structural members, thinner concrete decks and longer spans. Actual office live loads are probably less than one-half of what they were 25 years ago, and permanent partitions are more scarce resulting in less damping. The older methods simply do not account for these changes. For instance, the *Modified Reiher-Meister Scale* assumes 5 to 8% log decrement damping, a level very unlikely for today's floors.

Consider the floor framing shown in Figure 2. The structural system is 3¼″ normal-weight concrete on 0.6″ C deck, supported by 24K8 joists at 24″ on center and spanning 38′. The joists are supported by W24×76 girders spanning 30′. Nothing about this system is really unusual except that the live load deflection for the joists is less than $L/480$. The *Modified Reiher-Meister Criterion* predicts a "slightly perceptible" floor. The *Murray Criterion* requires 4.1% damping, which is easily justified. The *Design Guide* predicts a peak acceleration of $0.66\%g$, which is greater than the office environment tolerance acceleration of $0.50\%g$ and is an unacceptable floor. The framing shown is nearly identical to a recently investigated floor where the building occupants had complained quite vigorously and where damping posts were installed to reduce vibration.

Tip 3: Consider the consequences of an electronic office.

The electronic office is virtually paperless; I have been in one where the only papers were a few newspapers (mostly the financial section) scattered around the computer terminals. The result is much less live load and much less damping. Desks, filing cabinets

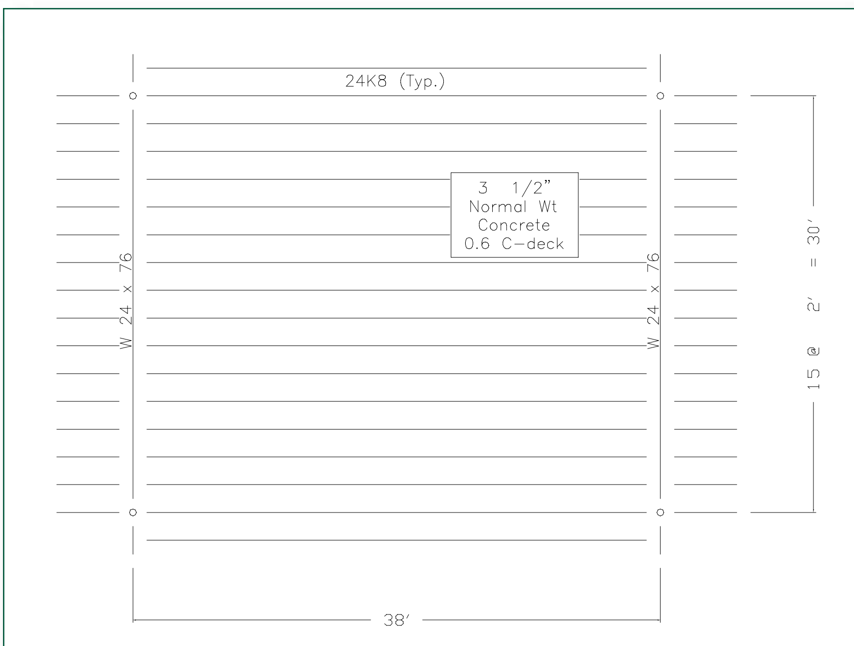

Figure 2. Floor framing.

and bookcases are live load and great sources of damping. In their absence, the potential for annoying floor vibrations mounts. Adding to the problem are modern floor layouts–open, with few fixed partitions, widely spaced demountable partitions or no partitions at all. Atrium-type areas are more common and curtain walls are less stiff. What's the solution? Use the AISC/CISC design guide methods, assume actual live loads in the 6 to 9 psf range, and modal damping of 2 to 2.5% of critical.

Recently, because of an annoying floor, the office contents in one building were actually weighed—the result was an equivalent weight of 8 psf! Throw in the humans, and you may get 9 psf! The floor design live load was 125 psf. Do we need to change our code live loads? Probably, but that's a question for the ASCE-7 Committee.

What about damping? Read on.

Tip 4: Don't mix-up Log Decrement and Modal Damping.

Now for some jargon: log decrement damping was used to develop the older heel-drop-based floor vibration tolerance criteria. Unfortunately, log decrement damping overestimates the damping as it measures not only energy dissipation (true damping) but also the transmission of vibrational energy to other structural components. The design guide criteria use modal damping or "true" damping (it's interesting that we call modal damping "true damping" when we cannot measure it very accurately, at least in floors). What's the difference? Only about 50% to 100%, so be careful! Modal damping is one-half to two-thirds of log decrement damping, so if you are accustomed to estimating damping for heel-drop based criteria, you will need to adjust your design office practices.

What are good modal damping estimates? Damping is usually expressed as a ratio of critical damping. Critical damping is the damping required to bring a system to rest in one-half of a cycle. That is, if you hit something and it has 1.00 or 100% critical damping, it will come to rest without oscillating. For offices with fixed partitions, a good estimate is 0.05 or 5%; for conventional or paper offices, i.e. good old structural engineering offices, with demountable partitions, use 3%; and for the paperless or electronic office, I recommend 2 to 2.5%. Note again that these numbers are much less than those recommended for heel-drop based criteria.

Tip 5: Do not design floors with a natural frequency below 3 Hz

Walking speed in an office can be 1.25 to 1.5 steps per second (or Hz). Resonance at the second harmonic, 2.5 to 3 Hz, is then a real possibility if the floor's natural frequency is below 3 Hz. I have caused a floor to vibrate at its natural frequency by running a shaker (an electrically-powered oscillating mass) at one-half of the floor frequency. The result is quite unsettling; if this happened in an office building, complaints would be loud and clear.

However, a 3 Hz or less floor can be made to work if it is made very heavy, say 100+ psf.

Tip 6: Remember that joists and joist-girders require special consideration.

The stiffness of trusses is affected by shear deformations in the webs. An age-old rule-of-thumb is that the effective moment of inertia of a parallel chord truss is 0.85 times the moment of inertia of the chords. This rule is used to compute the $L/360$ deflection limit live load in the Steel Joist Institute load tables. This rule works well if the span-to-depth ratio of the truss is greater than about 18; if the ratio is less, the deflection will be greater than predicted.

Joists and joist-girders have another problem—they are not fabricated with work points. Panel point eccentricities of up to 2", as shown in Figure 3, are common. Surprisingly, this has no effect on strength although member stiffness is reduced, especially if the span-to-depth ratio is less than about 18. The design guide offers the following expressions that are used to predict the effective moment of inertia of joist and joists girders:

- for angle web members with $6 < L/D < 24$:

$$C_r = 0.90\ (-e^{-0.28(L/D)})^{2.8}$$

- for round rod web members with $10 < L/D < 24$:

$$C_r = 0.721 + 0.00725\ (L/D)$$

where L is the member span and D is the nominal depth; and

$$I_{mod} = C_r\, I_{chords}$$

This moment of inertia is then used to calculate the effective transformed moment of inertia of the composite section. The above expressions were developed using static analysis and tests and apply equally well to static live load deflections.

For many years, I maintained that joist seats provided enough stiffness so that the supporting girder or joist-girder could be considered fully composite for floor vibration analysis. I was very wrong. Using floors constructed in the Virginia Tech Structures and Materials Laboratory, we found that joist seats are not, in fact, very good shear connectors. The design guide recommends that the composite moment of inertia of a girder or joist girder be approximated using:

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

where I_{nc} and I_c are the non-composite and fully composite moments of inertia, respectively. Recent field tests have shown this expression is a bit conservative if the joists are closely spaced, say not more than 30", and unconservative if there are only two or three joists being supported by the girder or joist-girder. Testing is currently being conducted to develop better approximations.

Tip 7: Improve a design that does not satisfy the criterion.

The criteria for heel-drop based methods indicates that increasing the stiffness has very little effect on the floor performance. With these methods, the only way to effectively improve a proposed floor design is to increase the mass. A different result is found when the design guide methods for office floors are used. With this method, the tolerance criterion can be satisfied by either increasing the mass or increasing the stiffness. A stiffer floor is always a better floor so the latter result is logical–no one has ever had a vibration problem with a 10′ span.

If the design guide method is being used and a proposed framing scheme does not satisfy the tolerance criterion, e.g. 0.5% of gravity, there are two approaches to improving the design. First, you can increase the mass by adding concrete or changing from lightweight to normal weight concrete. This approach will result in a slightly lower fundamental frequency but a larger effective weight, *W* in the criteria. The lower frequency will increase the predicted acceleration and the larger effective weight will decrease it, usually more than the frequency-caused increase, resulting in a better floor. Second, you can first stiffen the member (beam or girder) with the lower frequency until both frequencies are approximately the same. If the system is still not satisfactory, member types can be stiffened until a satisfactory design is achieved. My experience has shown that the latter method is more cost effective for most designs.

Tip 8: Don't believe the myth that certain beam spans should be avoided.

In the late 60s or early 70s a paper was written describing a number of joist-supported problem floors where the joist spans were in the 24′ to 28′ range. Somehow this was interpreted to mean that bays with beam or joist spans in this range should be not be designed, and this belief has become part of the folklore (if I may use that term) of the structural engineering community. Even some joist manufacturer engineers will tell you to avoid these spans. The problem floors described in the original paper were typical of the time, meaning that the spans and the problems were connected. But, in fact, there is no correlation between span and occupant complaints. Span alone is not the reason a particular floor is annoying to occupants.

Likewise, long span floors, say spans greater than 40′, are not inherently problem floors. I have made measurements on composite joist supported floors with spans between 40′ and 118′ (that's not a typo, there truly is an office building with a 118′ span). The design guide criteria predicted the floors would not be annoying and they were not.

The bottom line is that floors of any span can be designed such that occupants will not feel annoying vibrations. Just be sure the design satisfies the design guide criteria and the frequency is above 3 Hz.

Tip 9: Be careful when designing crossovers (elevated walks).

Atrium crossovers can be a design challenge. Crossovers typically have long spans; therefore, the frequency is quite low. Further, there is very little damping, generally about 1% modal damping. The result is that deep, stiff supporting members are required.

Also, the location of the slab needs to be considered. I know of two problem crossovers where the structural engineers relied on previous experience with floors of similar framing and did not check the crossover design. In both cases, complaints were received even before the buildings were opened. The major cause of the problems was that the crossover slab was located between the supporting beams at about mid-depth as shown in Figure 4. The result was that the moment of inertia of the crossover was twice the moment of inertia of the supporting beams, which, of course, is much less than

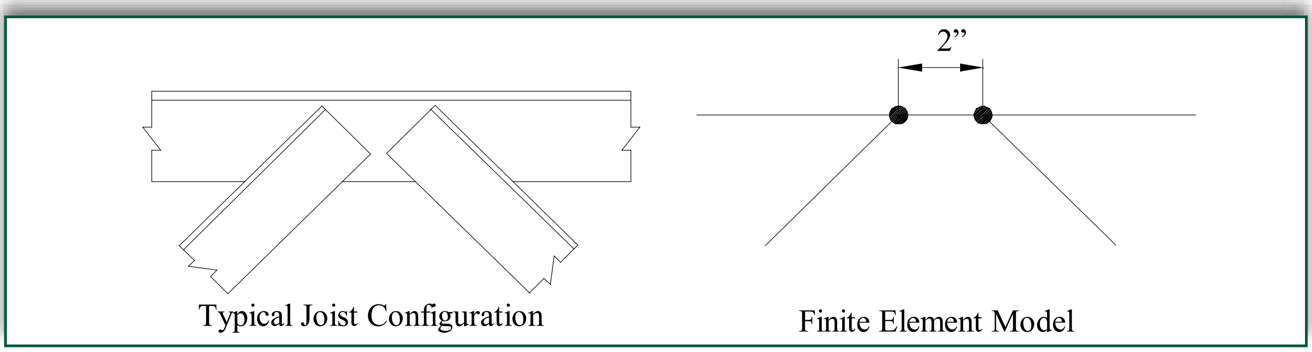

Figure 3. Joist panel point eccentricity.

the composite moment of inertia would had been if the slab was on top of the beams. The result was a much lower frequency than expected and an expensive fix in both cases.

Tip 10: Be even more careful when designing health clubs in office buildings.

Aerobics classes are part of any health club's activities, and an aerobics class is probably the most severe building floor loading for vibration concerns. The energy from aerobics can travel much farther than you might expect. I know of an instance where aerobics on the 60th floor of a building were felt on the 40th floor but not on the floors in between or below the 40th floor! Aerobics in one corner on the second level of a two-story strip mall has been felt several hundred feet away. Solutions are costly: a 400% increase in steel weight over the strength design would have been required in a strip mall to solve the problem (the owner decided to move the health club to the lower level instead).

The design guide has criteria for designing floors supporting rhythmic activities. Basically, the floor frequency must be above a limiting value that depends on an acceleration limit, which is determined considering the activity and what is called the "affected occupancy" and the weight of the floor. The acceleration limits for aerobics alone, aerobics in conjunction with weight-lifting and aerobics near offices are 5 to 10%, 2% and 0.5%, respectively. It turns out that weight-lifters are sensitive folks, thus the lower limit. Also, some of them are big, so you have to be extra careful! The required floor frequencies for the three conditions and a 100 psf floor are 8.8 Hz, 9.2 Hz and 16 Hz. For a 50 psf floor, the corresponding frequencies are 9.2 Hz, 10.6 Hz and 22.1 Hz.

If the spans are less than, say 30', use of lightweight concrete and closely spaced, deep joists will result in a floor frequency in the range of 10 to 12 Hz without too much expense. The floor system would be satisfactory for aerobics alone or in conjunction with weight-lifting but not near offices. Generally, it is cost prohibitive to design a floor system that supports both aerobics and offices.

If the aerobics activity cannot be moved to a slab on grade, then I suggest either a separate framing system for the aerobics floor or the use of a floating floor. Separate framing is an easy solution for two story buildings.

When using this approach, the aerobics floor slab must be completely separated from the surrounding slabs, and the ceiling below cannot be supported from the aerobics floor framing. Separate cold-formed framing connected only to the columns has been used to support the ceiling below.

Floating floors may be the only solution in a tall building. The concept of a floating floor is similar to that used for isolating machinery vibration. A floating floor is simply a separate floor supported by very soft springs attached to the structural floor. The natural frequency of the floating floor should be quite low, less than 2 to 3 Hz, which generally requires a heavy slab, 50 to 100 psf. Also, the space between the two floors must be vented or the change in air pressure due to the movement of the floating floor will cause the structural floor to move.

A Final Thought

A number of structural engineers have told me that they now design for serviceability and then check strength. As Hardy Cross once wrote: Strength is essential but otherwise not important.

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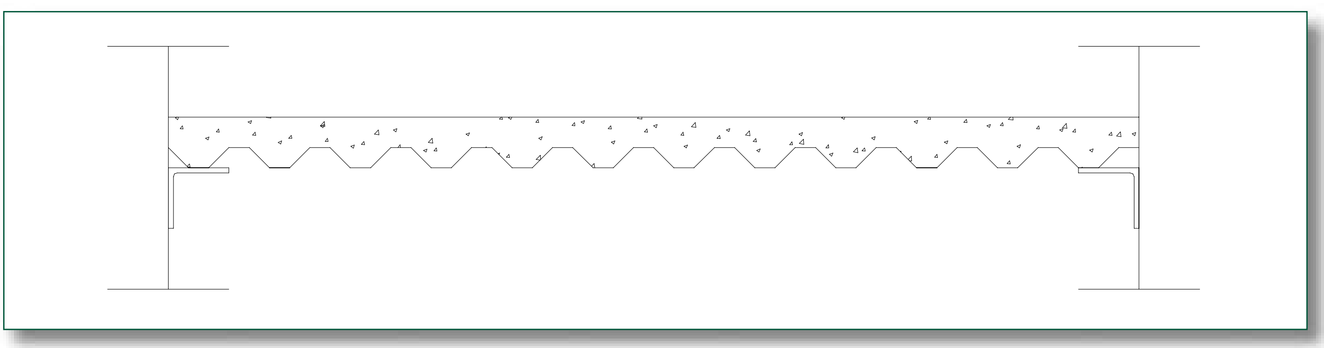

Figure 4. Crossover cross-section. Note: transverse members allow the deck to run parallel to the girders as shown.