Assessing Vibrations

Analysis of a long-span post-tensioned concrete structure

BY JEFFREY S. WEST, MATTHEW J. INNOCENZI, FERNANDO V. ULLOA, AND RANDALL W. POSTON

Live load vibrations of concrete-framed structures are generally not an issue due to their inherent stiffness. On rare occasions, however, human perception of vibrations in a concrete structure is of paramount concern. This was the case for the renovation and expansion of the Campus Recreation Center (CRC) at the Georgia Institute of Technology, located in Atlanta, GA. The structure consisted of a long-span post-tensioned concrete structural system used to span swimming and diving pools constructed for the 1996 Olympics, creating a new fourth floor for the CRC (Fig. 1).

Construction of one of the post-tensioned girders spanning the pool is shown in Fig. 2. The new structure supports various gymnasiums, multipurpose rooms for aerobic activities (Fig. 3), and an elevated jogging track. Due to the rhythmic nature of these types of activities and the overall span of about 150 ft (46 m), a peer review was conducted to assess if activity-induced vibrations would be an issue.

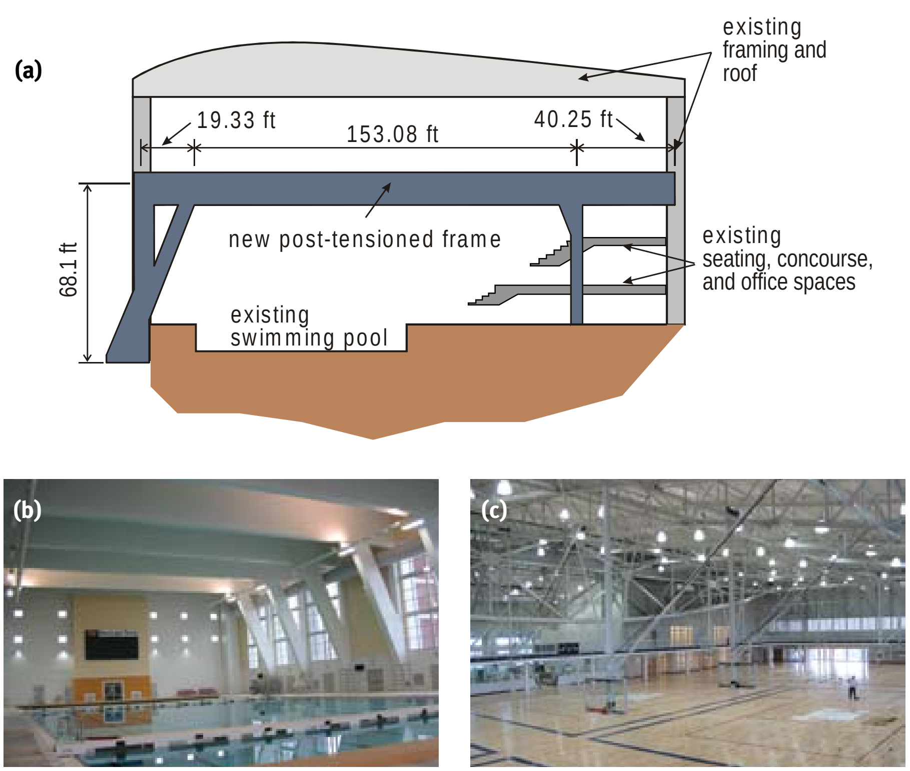

Fig. 1: Long-span post-tensioned concrete frame added within the existing Campus Recreation Center structure at the Georgia Institute of Technology in Atlanta, GA: (a) section through the building showing the new floor and frames; (b) photo of existing pool with new floor above; and (c) photo of six-court gymnasium and elevated jogging track on new floor level (1 ft = 0.3048 m) *(photos courtesy of Lawrence Kahn, Professor, Georgia Tech)*

Fig. 2: Inspection of the tendons and reinforcement for a typical long-span girder *(photo courtesy of Lawrence Kahn, Professor, Georgia Tech)*

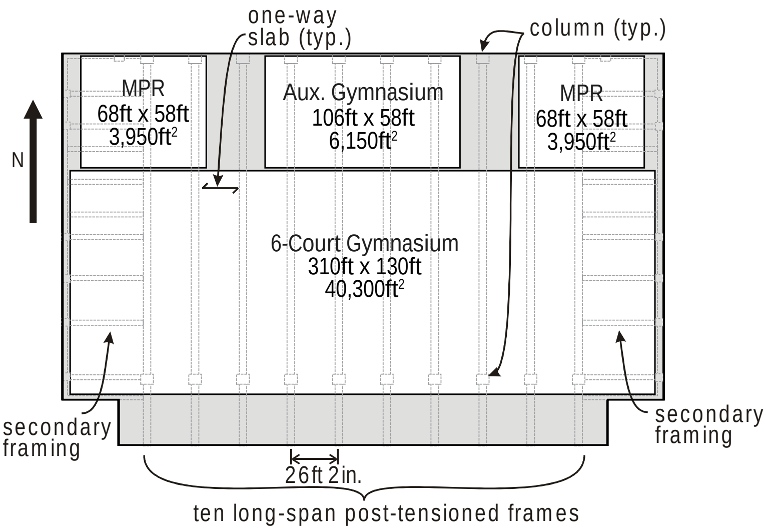

Fig. 3: Plan of the new fourth floor structural framing (1 in. = 25.4 mm; 1 ft = 0.3048 m)

VIBRATION CRITERIA

The vibration specification provided by the owner contained three main criteria related to floor vibrations from human activity. The criteria were adapted from the AISC Steel Design Guide Series 11.[1] The specified damping of 2% of the critical value was used for all computations, and the average weight of an individual was assumed to be 157 lb (70 kg).[1] The vibration criteria were as follows:

- Criterion 1—The maximum vertical acceleration of the new floor due to aerobic exercise could not exceed 5% of gravitational acceleration (*g*). The exercise classes could be attended by up to 78 adults, each occupying 50 ft^2 (4.6 m^2), and could take place in either of the multipurpose rooms (MPR) or the auxiliary gymnasium. The forcing frequencies ranged from 2.0 to 2.75 Hz[1];
- Criterion 2—The maximum vertical acceleration of the new floor due to aerobic exercise could not exceed 5% *g*. The exercise classes could be attended by up to 20 adults, each occupying 50 ft^2 (4.6 m^2), and could be located in any 1000 ft^2 (93 m^2) area in the six-court gymnasium. The forcing frequencies ranged from 2.0 to 2.75 Hz[1]; and
- Criterion 3—The maximum vertical acceleration of the new floor due to dancing could not exceed 2.5% *g*. The dancing classes could be located in any 40 x 40 ft (12 x 12 m) area in the six-court gymnasium. The dancing load was 12 lb/ft^2 (0.57 kPa) with forcing frequencies between 1.5 and 2.5 Hz.

DYNAMIC ANALYSIS MODELS

Dynamic analyses of the post-tensioned floor system were performed using commercial structural analysis software to determine the mode shapes and natural frequencies of the structural system. Time history analyses were used to assess the response of the structure to specific loading conditions and vibration criteria.

Model configuration

Ten large, long-span, post-tensioned concrete beams spaced at 26 ft 2 in. (8 m) on center span the existing swimming pool area, as shown in Fig. 1 and 3. Post-tensioned one-way slabs span between the beams to complete the floor system. An elevation view of the principal post-tensioned frame, including modeling details, is provided in Fig. 4.

The basic space frame model for the floor system is shown in Fig. 5. The structure was modeled using a grillage model where all of the components were beam elements; no plate or slab elements of any kind were used. The post-tensioned beams were modeled as beam elements with T- or L-shaped cross sections, as applicable. As shown in Fig. 6, beam elements transverse to the post-tensioned beams represented the flexural behavior of the one-way concrete slab. The grillage of longitudinal and transverse members in the model is visible in Fig. 5. This approach was selected for the following reasons:

- It provided a reasonable representation of the mass and stiffness distribution in the floor system;
- It permitted modeling the two-way pattern load distribution characteristics of the overall structural system in a frame comprising one-way slabs; and
- It avoided the complications of using beam elements for the post-tensioned beams and plate elements for the slabs, and thus avoided having to account for the eccentricity between the beam and slab centers of gravity.

When performing a dynamic analysis, the program generated joint or nodal translational masses from the member or element loadings, including the self-weight load case. Thus, to more accurately model the distribution of mass in the structure, the long-span post-tensioned frame members were subdivided into a number of shorter elements. This resulted in a space frame model (Fig. 5) with a total of 686 nodes and 946 beam elements. This approach did not have an effect on the static load case, but was very important to obtain reasonable estimates of the mode shapes and frequencies for the structure.

The entire fourth floor framing system was included in the model. It was not possible to take advantage of symmetry to reduce the model size because the columns supporting the secondary framing had different heights at the two ends of the facility and because nonuniform pattern loading on the fourth floor needed to be considered in the time-history analyses.

Member offsets

Due to the large member sizes and configuration of the long-span post-tensioned frames, member offsets (rigid links) were used to more accurately model the frame components. The dashed lines in Fig. 4 and 5 indicate member offsets used to limit the deformable length of the member.

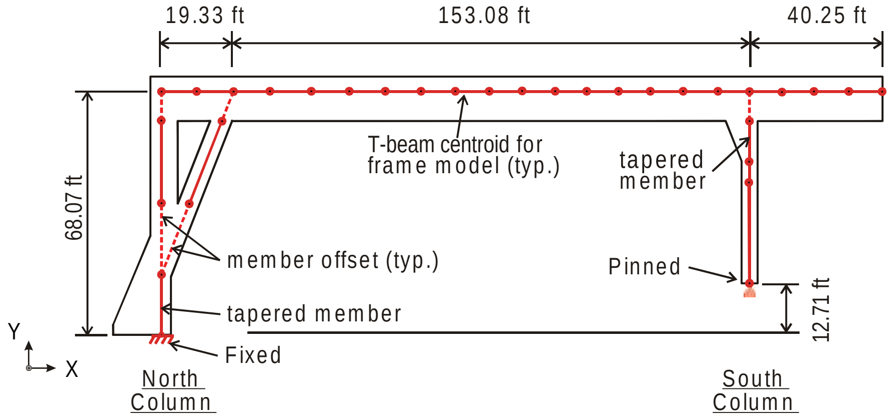

Fig. 4: Details and modeling of a typical long-span frame (1 ft = 0.3048 m)

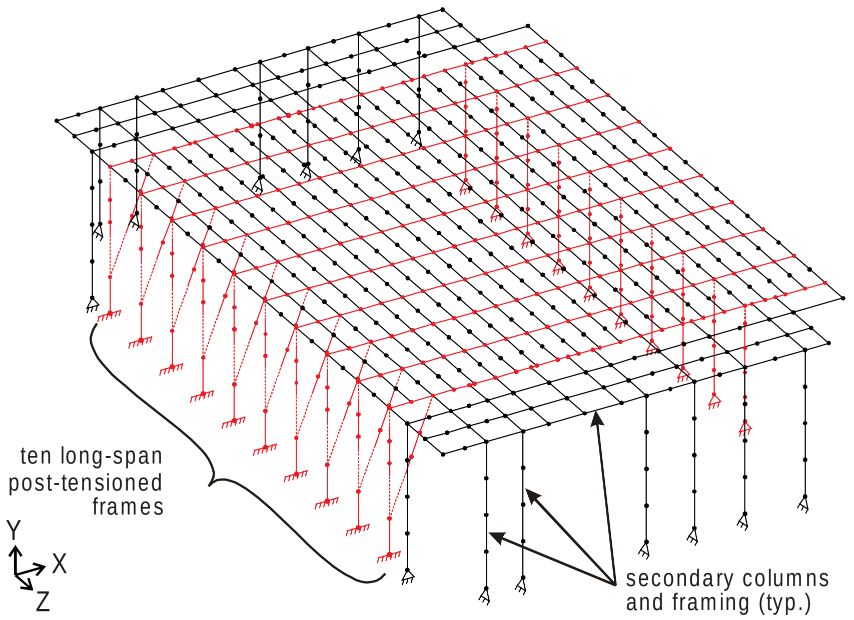

Fig. 5: Isometric view of the structural model used for vibration analyses. Long-span frames are highlighted in red

Boundary conditions

The boundary conditions for the columns were assigned based on the structural detailing provided and are shown in Fig. 4. The bases of the north columns were designed to be moment resisting, and thus were modeled as fixed supports. The south column bases were detailed to minimize rotational restraint, and thus were modeled as pinned supports.

Section properties

The long-span post-tensioned beams were designed to be fully prestressed. As such, the beams were assumed to be uncracked and gross section properties were used. Flange widths for the beams were determined using the effective flange width provisions from ACI 318-02.[2] All section properties were based on the appropriate T- or L-beam dimensions, except for the cross-sectional area. The beam area was taken as the area of the web only so that the mass of the slab would not be counted twice where the beam and slab members overlap in the grillage model, as illustrated in Fig. 6.

In the direction perpendicular to the long-span post-tensioned frames, the concrete slab was divided into strips and modeled using beam elements. These elements were rectangular sections with a width equal to the assumed slab strip width and the full slab depth of 8 in. (200 mm).

The columns of the primary structural frames were designed as reinforced (nonprestressed) concrete. Consistent with ACI 318-02, the column moments of inertia were taken as 70% of the gross section moments of inertia to account for possible cracking. The bases of the north columns and the tops of the south columns were tapered, as shown in Fig. 4. This geometry was accounted for in the model using beam elements with tapered member profiles.

Material properties

The design concrete compressive strength was 8000 psi (55 MPa) and the density was 145 lb/ft^3 (2300 kg/m^3) for all floor slabs, beams, and columns. Based on these values, an elastic modulus of 5100 ksi (35,200 MPa) was computed in accordance with ACI 318-02. Following the AISC Steel Design Guide Series 11 recommendations, the elastic modulus was increased by a factor of 1.35 to 6900 ksi (47,600 MPa) to account for increased concrete stiffness under dynamic loading.

LOADING

Static loads

The primary static loading on the structure considered in the vibration analysis was the self-weight of the concrete structure, including the structural frame, secondary framing, and slab. In addition, the superimposed dead loads due to flooring and cladding and the weight of the elevated structural steel jogging track were included. The self-weight of the concrete structure and slab components were automatically calculated based on the member cross-sectional areas and material densities.

Dynamic loads

The dynamic analysis consisted of two components. The first was an eigenvalue solution to determine the free vibration mode shapes and frequencies of the structure. The mass distribution in the structure used in the eigenvalue solution was based on the static dead load case (self-weight plus super-imposed dead load). The second component was a forced vibration time history analysis that was performed using the mode superposition method. This analysis was used to evaluate the project-specific vibration criteria. A cyclic load history was developed based on the project vibration criteria and on information contained in the AISC Steel Design Guide Series 11.[1] The shape of the loading history, and in particular the period during which load was applied, sustained, and removed during a given load cycle, was based on the AISC Document.

A sample load history based on a measured loading function for rhythmic activity[1] and used in the time history analyses is shown in Fig. 7. The load history consisted of 10 cycles of loading, followed by an equivalent duration of no loading. The load history was varied in magnitude and forcing frequency to represent the desired vibration criteria given in the project specifications. The dynamic load was applied as a series of nodal loads over specific regions of the structure according to the project vibration criteria. Although not required by the project specifications, the effects of loading frequencies in the range of the first six natural frequencies of the structure were also evaluated for Criterion 3.

TABLE 1:
FLOOR SYSTEM NATURAL FREQUENCIES

Mode number	Frequency, Hz
1	4.47
2	4.70
3	5.15
4	5.31
5	5.58
6	5.85

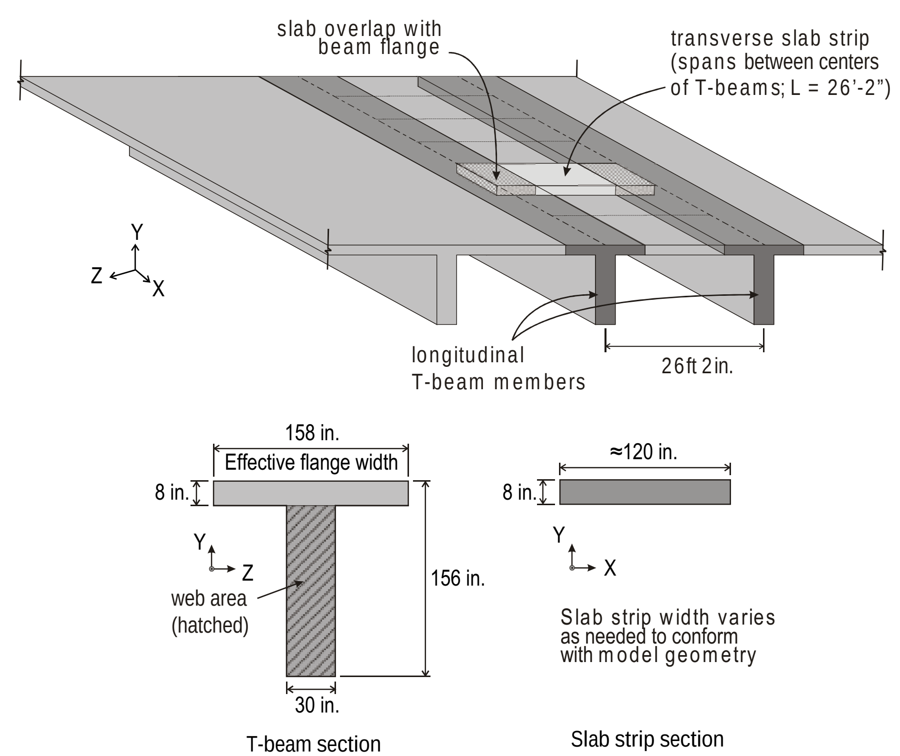

Fig. 6: Sections used for analysis of T-beams and transverse slabs. To avoid double-counting the weight of the slab where it overlapped the flange of the T-beam, the cross-sectional area of each T-beam included only the web area (1 in. = 25.4 mm; 1 ft = 0.3048 m)

MODE SHAPES AND FREQUENCIES

The fundamental frequencies for the first six modes of vibration are presented in Table 1. Amplified deflected shapes for Mode 1 and Mode 4 are presented in Fig. 8 as examples of the computed modes of vibration.

TIME HISTORY ANALYSIS RESULTS

Criterion 1

Vibration Criterion 1 involved aerobic activities in a multipurpose room or the auxiliary gymnasium. Three different load areas or configurations were considered to evaluate this criterion: one in the multipurpose room, and two in the auxiliary gymnasium, as shown in Fig. 9(a). Forcing frequencies of 2.0, 2.25, 2.5, and 2.75 Hz were considered, as specified in the Criterion 1 requirements for aerobic activities.

The maximum computed vertical accelerations for each combination of load area and forcing frequency are listed in Table 2 and are well below 5% g, indicating that the structural system satisfied Criterion 1. The maximum computed vertical acceleration of the floor due to aerobic activity in the multipurpose room was 2.88 in./s^2 (0.073 m/s^2) or 0.75% g. Intuitively, the maximum acceleration might be expected to occur under the loaded area. In this case, however, the location of the peak acceleration was not immediately under the loaded area but instead was out towards the middle of the long span. It occurred in the six-court gymnasium at a forcing frequency of 2.75 Hz. Vertical accelerations within the multipurpose room were significantly less than the peak value.

The maximum computed vertical acceleration of the floor due to aerobic exercise in the auxiliary gymnasium was 4.27 in./s^2 (0.108 m/s^2) or 1.11% g. Again, the peak acceleration did not occur immediately under the loaded area but instead was out towards the middle of the long span. It occurred in the six-court gymnasium at a forcing frequency of 2.25 Hz. Again, vertical accelerations within the auxiliary gymnasium were significantly less than the peak value.

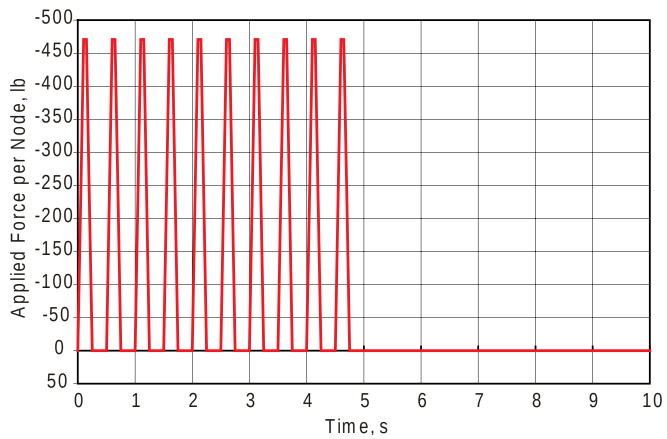

Fig. 7: Sample loading function, based on Fig. 2.3 of Reference 1, used for a time history analysis. Here, 10 load cycles at 2 Hz were applied, followed by an equivalent duration of zero live load (1 lb = 4.45 N)

TABLE 2:
TIME HISTORY ANALYSIS RESULTS FOR CRITERION 1

Loaded area*	Forcing frequency, Hz	Maximum computed vertical acceleration, g†
Area 1: multipurpose room	2.00	0.32%
	2.25	0.69%
	2.50	0.69%
	2.75	0.75%
Area 2: auxiliary gymnasium	2.00	0.32%
	2.25	0.83%
	2.50	0.68%
	2.75	0.65%
Area 3: auxiliary gymnasium	2.00	0.41%
	2.25	1.11%
	2.50	0.77%
	2.75	0.74%

*See Fig. 9 for loaded areas.

†g = acceleration due to gravity = 386 in./s^2 (9.81 m/s^2).

Criterion 2

Vibration Criterion 2 involved aerobic activity in the six-court gymnasium. A total of six different load areas

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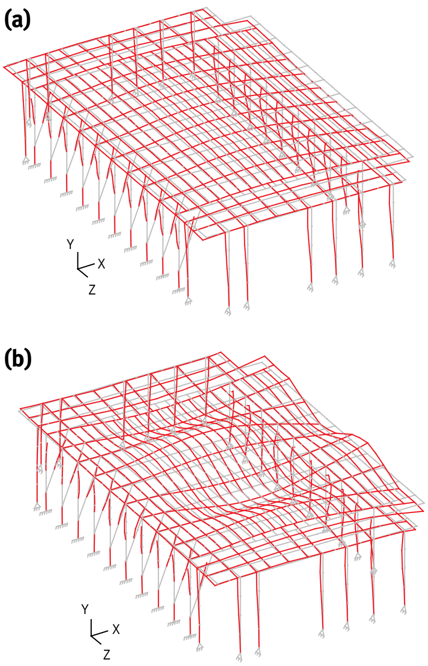


Fig. 8: Mode shapes for: (a) Mode 1; and (b) Mode 4. The corresponding natural frequencies were 4.47 and 5.31 Hz, respectively

were considered. The proportions and location of the rectangular load area were varied to evaluate the response of the structure to a range of possible conditions. Forcing frequencies of 2.0, 2.25, 2.5, and 2.75 Hz were considered, as specified in the Criterion 2 requirements. The maximum computed vertical acceleration was 3.14 in./s^2 (0.080 m/s^2) or 0.81% *g*. It occurred in the six-court gymnasium, directly under the centrally located loaded area shown in Fig. 9(b), at a frequency of 2.5 Hz, and was well below the criterion of 5% *g*. For brevity, the complete results (loaded areas and computed accelerations) for Criterion 2 are not shown.

Criterion 3

Vibration Criterion 3 involved dancing in the six-court gymnasium. Three different load areas were considered to evaluate the response of the structure to dancing loads in different areas of the gymnasium. The uniformly distributed dancing load was converted to nodal forces within the 1600 ft^2 (149 m^2) loaded area. For dancing, forcing frequencies of 1.5, 1.75, 2.0, 2.25, and 2.5 Hz were considered, as specified in the Criterion 3 requirements. Note that the range of forcing frequencies for dancing activities is different than that used for aerobics exercises in Criteria 1 and 2. The computed maximum vertical acceleration was 6.44 in./s^2 (0.164 m/s^2) or 1.67% *g*. Similar to the results for Criterion 2, this peak acceleration occurred in the six-court gymnasium, directly under the centrally located loaded area shown in Fig. 9(c), at a forcing frequency of 2.5 Hz. Again, for brevity, the complete results for Criterion 3 are not shown. The peak vertical acceleration for dancing activities was below the criterion of

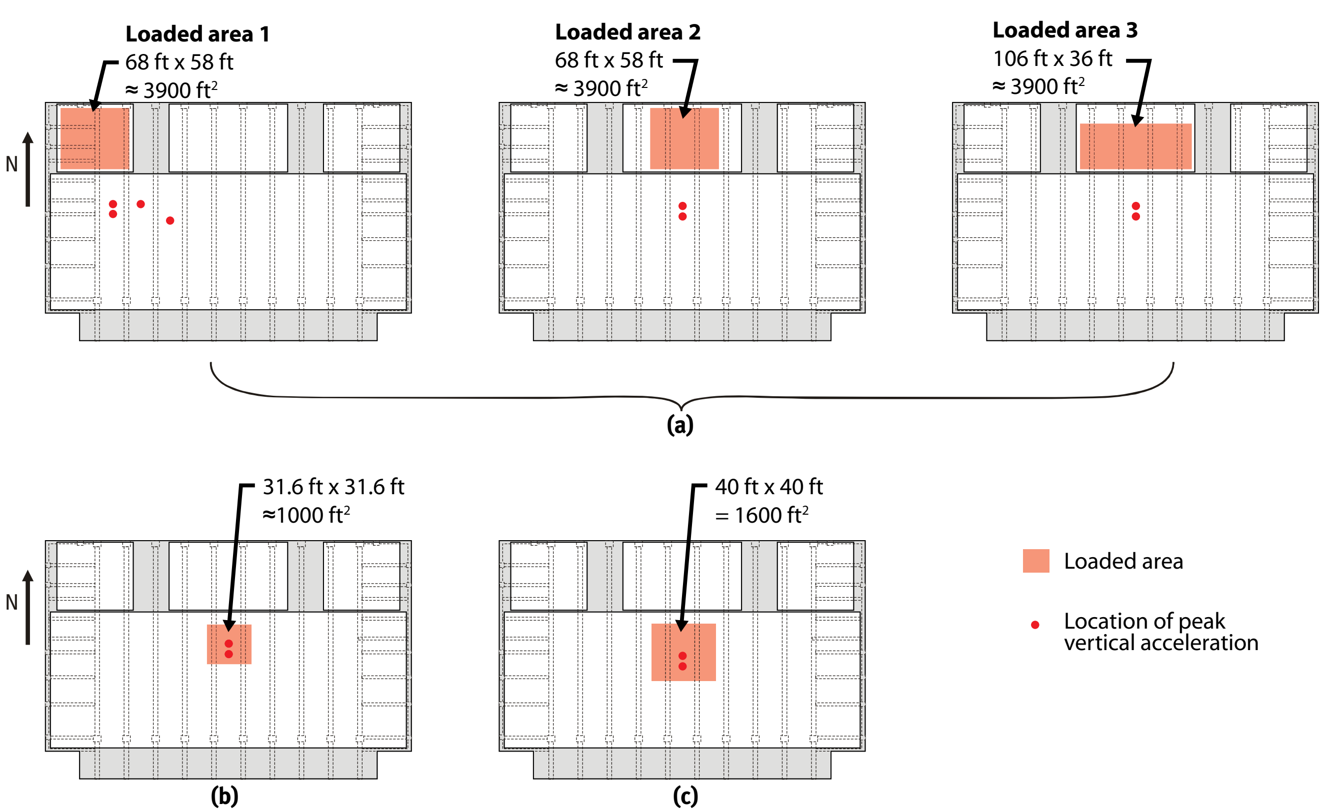


Fig. 9: Areas loaded for evaluation of vibration criteria: (a) Criterion 1; (b) Criterion 2; and (c) Criterion 3 (1 ft = 0.3048 m, 1 ft^2 = 0.093 m^2)

2.5% *g*, indicating that the system satisfied this aspect of the vibration criteria.

Although not required by the project specifications, the effect of loading frequencies between 4.25 and 5.75 Hz were also evaluated for Criterion 3. These values are in the range of the first six natural frequencies (Table 1) and should produce maximum accelerations. Under this very extreme and unlikely loading, the peak acceleration was 10.3 in./s^2 (0.262 m/s^2) or 2.67% *g*. This only slightly exceeds the maximum acceptable value of 2.5% *g*.

ASSESSING VIBRATION CHARACTERISTICS

Although human perception of vibration is not normally an issue for concrete structures, the new CRC floor had a relatively long span and its function as an activity and recreational facility predisposed the floor system to vibrations. Detailed analyses showed that the vibration characteristics of the post-tensioned structure met the project-specific criteria. Since opening in 2004, the new floor level has been used for cheerleader and ROTC training as well as various other rhythmic exercise activities. There have been no complaints about floor vibrations, and the owners are very happy with the facility.

References

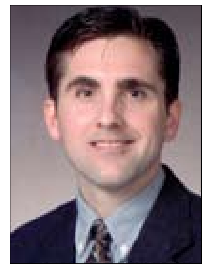
1. Murray, T.M.; Allen, D.E.; and Ungar, E.E., "Floor Vibrations Due To Human Activity," AISC Steel Design Guide Series 11, American Institute of Steel Construction, Chicago, IL, 1997, 69 pp.

2. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (318R-02)," American Concrete Institute, Farmington Hills, MI, 2002, 443 pp.

Selected for reader interest by the editors after independent expert evaluation and recommendation.

PROJECT CREDITS

Owner: Facilities, Georgia Institute of Technology, Atlanta, GA
Architect: Hastings & Chivetta Architects, Inc., St. Louis, MO
Structural Engineer: ABS Consulting, St. Louis, MO, and Continental Concrete Structures, Inc., Alpharetta, GA
Structural Peer Reviewer: WDP & Associates, P.C., Austin, TX
Contractor: Beers Skanska (Skanska USA Building), Atlanta, GA
Post-Tensioning Supplier: Continental Concrete Structures, Inc., Alpharetta, GA
Major Awards: 2002 Award of Excellence—Public Works Category from the ACI Georgia Chapter and 2004 Stuctures Award—Industrial/Special Applications from the Post-Tensioning Institute



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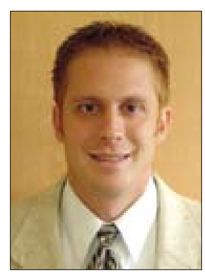

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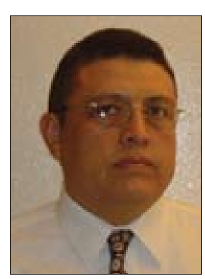

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