10(2013) 601 – 630

Latin American Journal of
Solids and Structures
www.lajss.org

Finite element analysis of high modal dynamic responses of a composite floor subjected to human motion under passive live load

Arash Behnia[*, a],
Ahmad Kueh Beng Hong[b],
Mohammad Mehdi Shabazi[c],
Navid Ranjbar[a],
Nima Behnia[d],
Mohamad Reza Vafaei[b]

[a]University Malaya- Department of Civil Engineering- Structure and Material- Kuala Lumpur- Malaysia

[b]Universiti Teknologi Malaysia, Faculty of Civil Engineering, Department of Structure and Materials Steel Technology Centre, 81310 Skudai, Johor, Malaysia

[c]University of Tehran, Engineering Optimization Research Group, Iran

[d]Islamic Azad University- Ghazvin Branch, Department of Civil Engineering, Iran

Received 23 Apr 2012
In revised form 13 Aug 2012

*Author email: behnia.arash@gmail.com
barash@siswa.edu.um.my

Abstract

Light weight and long span composite floors are common place in modern construction. A critical consequence of this application is undesired vibration which may cause excessive discomfort to occupants. This work investigates the composite floor vibration behavior of an existing building based on a comprehensive study of high modal dynamic responses, the range of which has been absent in previous studies and major analytical templates, of different panels under the influence of loads induced by human motion. The resulting fundamental natural frequency and vibration modes are first validated with respect to experimental and numerical evidences from literature. Departing from close correlation established in comparison, this study explores in detail the effects of intensity of passive live load as additional stationary mass due to crowd jumping as well as considering human structure interaction. From observation, a new approach in the simulation of passive live load through the consideration of human structure interaction and human body characteristics is proposed. It is concluded that higher vibration modes are essential to determine the minimum required modes and mass participation ratio in the case of vertical vibration. The results indicate the need to consider 30 modes of vibration to obtain all possible important excitations and thereby making third harmonic of load frequency available to excite the critical modes. In addition, presence of different intensities of passive live load on the composite floor showed completely different behavior in each particular panel associated with load location of panel and passive live load intensity. Furthermore, implementing human body characteristics in simulation causes an obvious increase in modal damping and hence better practicality and economical presentation can be achieved in structural dynamic behavior.

Keywords

Composite floor vibration; finite element analysis; human structure interaction; human motion; dynamic response; modal damping

1 INTRODUCTION

Competitive industrial world has compelled structural designers to provide low cost and prompt construction as well as light weight and low labor cost structures for modern global market. Composite floors, both steel-deck and dovetailed profile, are one of the most popular constructional methods used in high rise buildings. Long span and slenderness are the main characteristics of these structures and thereby serviceability problems, particularly floor vibration, have been found to be the chief concern attributable to structural stiffness and low frequency, which is susceptible to walking and jumping frequencies due to human motion. Moreover, static method and one-way spanning assumption which are common in the design of these floors have neglected vibrational issues. In practice, the rhythmic activities such as sporting events in stadium, dancing and jumping in aerobic classes or in gymnasium are the main load sources in floor vibration problems. To date, there exists a great number of studies in regard to the vibration of structural components in literature. Tredgold (1828) dealt with vibration problems and published one of the first papers on the stiffness criteria in order to avoid vibration troubles. By specifying depth limit for girders, a simple approach was proposed to provide satisfactory serviceability in preventing the shaking of items in the room. Lenzen [2] carried out a research in Steel Joist Institute to examine the vibrational performance of steel joist concrete slab. Also, a series of study on the behavior of composite floor, particularly steel deck composite floor had been conducted by Allen (1990), Williams and Waldron (1994), and Da Silva et al. (2003). With the aid of finite element method, they investigate the structural behavior subjected to vibration due to human motion. Studies conducted by Murray et al. (1997) were the main framework used by the American Institute of Steel Construction to publish AISC Design Guide 11, Floor Vibration Due to Human Activity, in providing serviceability design guidelines for designers. In addition to AISC Design Guide 11, “Design Guide on the Vibration of Floors” by steel construction institute SCI) (Wyatt,1989) and “Evaluation of Human Exposure to Whole Body Vibration (1997)” by International Standards Organization (ISO 2631-1) have presented limitations based on the acceptable acceleration regarding to human perceptibility in various types of occupancies. Nevertheless, presented formulae in these guidelines are an approximated consideration of simplified model which are not able to simulate precisely the behavior of multi-panel and multi-modal floors, a primary concern in practice.

Loads which are generated by human are hard to formulate in precise detail. Considering the type of activity, the magnitude and frequency of domain will vary significantly and are difficult to extract. In order to illustrate amplitude of vibration from a structure, there are several key factors that influence the response of the structure such as load intensity, type of activity, load location, activity rate in conjunction with system properties and so on. Copious investigation had been made intending to establish proper parameters to explain such vibrating loads (Allen et al., 1985; Bachmann and Ammann, 1987). A study was conducted by Ji and Ellis (1994) to obtain response of structure applied to different activities such as walking, jumping, and aerobics. Different frequencies, resulting displacements, and accelerations were captured. In addition, Da Silva et al. (2003) worked on the vibrational responses of the structures in active panels.

Thus far, several studies have been carried out to improve the procedure of finite element modeling of vibration serviceability problems. Da Silva et al. (2006) modeled a real composite floor structure to investigate its dynamic behavior when submitted to different types of activity such as walking and jumping. With the aid of Fourier series as loading formulation, Da Silva et al. (2006) conducted numerical modeling for different frequencies and the structural responses were compared with AISC Design Guide 11 limit criteria. From the study, they pointed out the effect of consideration of column and its height in finite element results. Thambiratnam et al. (2009), in order to investigate multi-panel loading and to find the multi-modal response of the floor, implemented half-sinusoidal curve for different contact ratio of jumping and aerobic activities as well as various damping values. However, they ignored the effect of crowd loading and reduction factors due to crowd dynamic loading.

Although there are several available studies on composite floor vibration problems, there is still lack of sufficient information in some details. To existing literature, higher vibration modes and their contribution in the response of the structure has been neglected. In addition, the effect of passive occupancy (passive live load) in terms of intensity and human-structure interaction in real composite floor dimension considering realistic jumping load were not investigated sufficiently. Recently, Behnia et al. (2013) explored the effect of coordinated passive live load and intensity of active occupancy (Behnia, 2012), considering synchronization effects of crowd.

This study addresses and investigates all the above mentioned aspects in which a comprehensive examination is carried out through the application of a realistic jumping load pattern on the composite floor based upon individual and crowd jumping loads. Besides the impact of higher vibration modes and the mass participation ratio in dynamic response of the composite floor, the effect of passive live load in two methods, with and without considering human-structure interactions, are investigated.

The studied structural model is based on an existing structure, the Cardington eight-storey steel framed building with profiled sheet composite floor, located in the United Kingdom. The floor dimension is 21 m by 45 m. in the present study, dynamic numerical simulations are considered. The experimental results are available in El-Dardiry et al. (2006) and Ellis et al. (2010). Dynamic characteristics of this structure were captured in two scenarios:

(i) One 76 kg person walking across the floor at a defined frequency to generate resonance.

(ii) 32 evenly spaced people (of average mass of 67.6 kg) jumping at the critical frequency on the floor

2 DYNAMIC OF HUMAN LOADS

In order to achieve occupancy comfort and a safe condition for structures, it is necessary to find the response of the structure for different types of dynamic loading that may be applied to the floor. It is reasonable that humans are capable of generating forces much greater than their self-weight through energetic activities such as jumping. As mentioned before, loads generated by human being are difficult to predict in detail and amplitude and frequency will vary in different types of induction. As a result, it is necessary to categorize different induction caused by human based on frequency domain and amplitude. The most common human loadings can be listed as below.

2.1 Walking and running loads

In general, walking may be categorized as a regular horizontal human body motion considering at least one foot in contact with the ground at all times during motion. Generally frequency domain is in the range of 1.4-2.2Hz (Sahnaci and Kasperski, 2005). Raising and lowering of the body centre of the mass give rise to vertical loads while horizontal loads are arisen from friction weight shifting. Rarely in individual cases do horizontal loads cause significant effect except in large scale loading e.g. crowds or in an unbraced structure.

2.2 Jumping load

Jumping is one of the physical that exerts substantial vertical load due to a sudden impact of human feet onto the contact body, in the current case, the floor. The action can be described as a transferring of load with magnitude many times that of the human static mass corresponding to spiking impulsive force the feet made once the jump ends with a landing. Jumping load is difficult to quantify and characterize in an exact manner since load at high frequencies cannot be repeated and is not generally periodic. Much effort is required to make correlations of the load under the influence of factors such as airborne duration, repetition speed, height of jump, and landing type. Nonetheless, a great account of attempts has been made to obtain load models for iterated jumping of either single persons or small groups by many researchers (Wyatt, 1985; Ellis and Ji, 1994; Moreland, 1905; Hansen and Sørensen, 2002), the formulations of which are based on equivalent static loads (Ellis and Ji, 1994; Kasperski, 2002) and Fourier series representation of load as well as application of equivalent human structure loads (Tilden, 1913). It should be stated here that from these studies, load frequencies for individuals, small groups with coordinated action such as aerobic activities and large crowds at concerts are respectively characterized within the range of 1.2–2.8 Hz, 1.5–2.5 Hz, and 1.8–2.3 Hz. In regard to these activities, Littler (2003) has proposed a load excitation frequency range of 1.0-3.5 Hz with a mean value of 1.8 Hz for applications.

2.3 Rhythmic exercise load

Rhythmic exercises are classified most often as unique types of load case and as a subset of jumping cases occasionally, as shown by BRE Digest 426 (Ellis and Ji, 1997). The classification emerged and formed based on investigations of many structures which experienced vibration problems.

3 LOAD MODELS

Having previously discussed types of human induced loads, it is essential to note that their model descriptions are not intuitively straightforward. Several works have been devoted in literature to provide the representation of aforementioned actions, the complexity of which varies from equivalent models to those consider underlying interactions. Since resonance is the chief concern when dealing with vibrational responses, it is a normal practice to consider model that captures action resultants, which are sufficiently small to not surpass the natural frequency of structures and to not cause distress to the occupants.

3.1 Half-Sine pulse

Half-sine pulse is the load model that expresses jumping load history as a series of half-sine pulses as proposed in Bachmann and Ammann (1987). The model is characterized by a specified width and amplitude, which take the form of contact ratio and impact factor, respectively. The contact ratio is the time fraction of the jumper during contact with the ground in each jumping cycles whereas the impact factor is the ratio of maximum force to jumper's body weight. For convenience, measured pulse of jumping obtained from experiment has been fitted using the half-sine model given as:

$$F_t = \begin{cases} k_p G \, sin\left(\dfrac{\pi t}{t_p}\right), & 0 \le t \le t_p \\ 0, & t_p \le t \le T_p \end{cases} \qquad (1)$$

Here, G is the static weight of the person, $k_p = F_{max}/G = (\pi/2\alpha)$ is the impact factor and F_{max} is the peak dynamic load. The contact duration is denoted as t_p, the jumping period is denoted as T_p and $\alpha = t_p/T_p$ is the contact ratio. It should be noted that equation 1 is applicable only to time dependent problems. When dealing with steady state responses, frequency domain model is a better option.

3.2 Use of Fourier series

Widespread use of Fourier series, a frequency domain model, can be tracked in works in regard to dynamic loadings owing to the fact that it deals with steady state problems without paying the cost of significant computational time in comparison to the half-sine variant. The working template is based on the division of signal into a summation of sets of sine waves at different frequencies, magnitudes, and phase lags/angles (Figure1). Fourier transform is then utilized to derive meaningful parameters from these sine waves for further post-processing (Figure2).

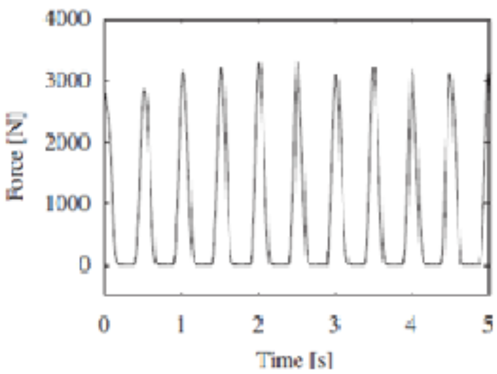

Figure 1 A measured history for jumping (Rainer and Swallow, 1986)

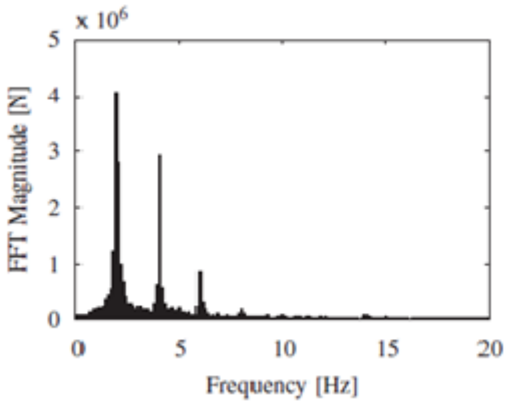

Figure 2 Frequency domain versus magnitude (Rainer and Swallow, 1986)

The typical Fourier series used to represent periodic human loading, takes the simplified form in equation 2:

$$F_t = \begin{cases} k_p G \; sin\left(\frac{\pi t}{t_p}\right) & , \quad 0 \le t \le t_p \\ 0 \quad , \quad t_p \le t \le T_p \end{cases} \tag{1}$$

$$F(t) = G\{1 + \sum_{n=1}^{\infty} r_n \sin[(2n\pi t / T_p) + \phi_n]\} \tag{2}$$

Here, F(t) represents the total force at time t, with G representing the weight of the occupant in the same unit. Using Fourier transform, the peaks of the frequency domain at integers multiplies of forcing frequency that equals to $1/T_p$ can be obtained by considering a sum of n harmonics. In equation 2, r_n is the dynamic load factors (DLFs) given by the Fourier amplitude coefficients and $\phi_n$is the phase angle in radians. Note that it is also feasible to get dynamic load factors and phase angles from measurements or from the contact ratio of the half-sine pulse model using procedure proposed by Ji and Wang (2001) (see equations 2.1 and 2.2):

$$r_n = \sqrt{a_n^{\;2} + b_n^{\;2}} \tag{2.1}$$

$$\phi_n = \tan^{-1}\left(a_n / b_n\right) \tag{2.2}$$

Where

$$a_n = \begin{cases} 0, & \alpha = 0.5 \\ \left(\frac{1+\cos(2n\pi\alpha)}{1-4n^2\alpha^2}\right), & n\alpha \ne 0.5 \end{cases} \tag{2.3}$$

$$b_n = \begin{cases} \pi / 2, & n\alpha = 0.5 \\ \left(\frac{\sin(2n\pi\alpha)}{1-4n^2\alpha^2}\right), & n\alpha \ne 0.5 \end{cases} \tag{2.4}$$

The dynamic load factors (DLFs) and phase lags may also be derived from works of other papers, the function can be modified with $\phi_n = \pi / 2$. By changing sine to cosine, it gives a time series with a maximum at t = 0, which is corrected by presuming $t = t - t_p$ throughout. Human activities can be characterized by different DLFs and phase angles.

4 EXPANSION TO CROWDS

The expansion of single imposed load to a realistic crowd model is intended for a wide variety of structures where crowds may gather at locations, such as gymnasium, dance floors, and grandstands. Previously presented load models are mostly reasonable for the individual or a small group jumping or dynamic loading but for cases in which floor is encountered with considerable crowd load, equation 1 and equation 2 are not only conservative but also impractical. In some recent guidelines it can be seen that coordination and crowd size of dynamic loading are being considered. ISO guidelines make explicit mention through the introduction of modified load equation in cases prone to large crowd size. Equation 3 is represented considering all above aspects.

$$Fv(t)N = C(N)Q[1 + \sum\nolimits_{n=1}^{k} \alpha_{\mathrm{n,v}} \sin(2\pi fnt + \phi_{nv})] \tag{3}$$

Here, $F_v(t)$ is the magnitude of the applied load at time t for a crowd of N people, C(N) is a coordination factor for a crowd of size N (see Table 1). Q is the weight of the associated crowd, k the number of DLF, α_n is the n-th DLF coefficient which is represented by different people in different values for different activities. For example in ISO guidance for jumping and dancing and exercise activities these formulae are represented as:

$$\alpha_1 = 2.1 - 0.15(f) \tag{3.1}$$

$$\alpha_2 = 1.9 - 0.17(2f) \tag{3.2}$$

$$\alpha_3 = 1.25 - 0.11(3f) \tag{3.3}$$

Table 1 Various crowd coordination factor

harmonic	NBC - 2005, 2006	ISO 10137 (coordination/Rhythmic ability) (high),2007	ISO 10137 (coordination/Rhythmic ability) (medium),2007	ISO 10137 (coordination/Rhythmic ability) (low),2007	IstructE, 2008	Ellis and Ji, 1997	BS 6399-1, 1996	Bachmann and Ammann, 1987
1	Included in load amplitudes	0.8	0.67	0.5	Included in load amplitudes	$m^{-0.082}$	0.67	0.75
2		0.68	0.5	0.4		$m^{-0.24}$		
3		0.5	0.4	0.3		$m^{-0.31}$		

As a first approximation, the values of α_1, α_2, and α_3 can be taken to be 1.7, 1, and 0.4, respectively. Again, f is the activity frequency and ϕ_n is the phase lag associated of n-th harmonic of the load. The subscript v implies that the vertical direction is being considered. In addition, α_n and the phase lag are the parameters. It is noticeable that values of the dynamic coefficient for the i-th harmonic, α_i , are based on a commonly encountered event involving a minimum of 20 persons. It implies that in cases where the source of dynamic load is from less than 20 persons, load pattern will be taken in the form as shown by equation 2.

5 HUMAN-STRUCTURE INTERACTION

Human-structure interaction is defined as the dynamic relationship that is formed from the contribution of attributes of dynamic properties of humans and structure in contact. Similar to other load models mentioned previously, the interaction model is particularly difficult to describe owing to underlying complexity since human dynamic properties customarily vary in accordance with different biomechanics found in terms of independent jumping postures and structure-human contact and release conditions which include also the irregular cycles. To date, there are two primary techniques for the human-structure interaction description:

- Indirect description – Increase damping parameter in addition to execution of load reduction
- Direct description – Prescribe extra degree of freedom with various parameters on structure to characterize human contribution

It should be stated here that the significantly simplified indirect method is currently provided in the User's Guide-NBC 2005, Canadian Commission on Building and Fire Codes (2006). In the method, no treatment is addressed for occupancy resonant frequencies. The direct technique is relatively recent (Sachse et al., 2004; Sachse, 2002) and hence not well established more investigations are warranted. There is a significant difference between the properties of passive crowd loads and those active crowd loads in activities such as dancing or jumping. Dynamic mechanical models with multiple degrees of freedoms have been considered to model human body as spring-mass system (see Figure 3). The works by (Brownjohn, 1999; Sim et al., 2007; Wei and Griffin, 1998) can be referred for modeling details. Although biomechanical properties of human body are well documented and model descriptions are developed in detail as a result, these models are habitually too complicated to be utilized for widespread engineering use. For instance, simulating individuals as a model with multiple degrees of freedoms for a crowd that consists of a thousand individuals is highly impractical. However, mean values for load frequency, damping ratio, and mass of human body have been pro-posed in a relatively recent work and given as 3.7 Hz, 37 %, and (70-80)kg for passive crowd loads in standing position, respectively (Brownjohn, 1999). It is essential to state that these values are suggested for a single person. So far, detailed dynamic properties for crowds are still ill-defined and most model descriptions are extended directly from those of individual or small groups.

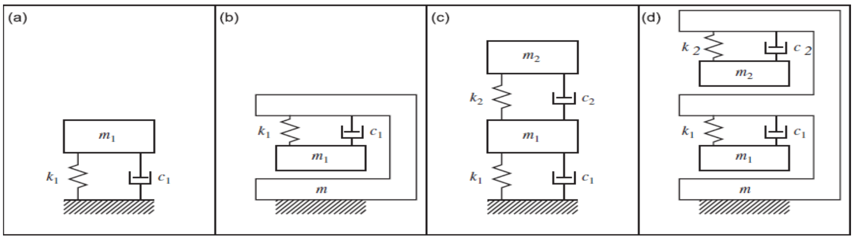

Figure 3 Simplified dynamic representations of the human body: (a) SDOF model, (b) SDOF model with rigid support, (c) 2DOF model, (d) 2DOF model with rigid support (Wei and Griffin, 1998).

6 STRUCTURAL DESCRIPTION AND FINITE ELEMENT MODEL

6.1 The structural model description

The typical plan of a floor of the Cardington steel frame building with a floor slab, constructed using a trapezoidal steel deck (PMF CF70), light weight concrete, and anti-crack meshes (A142 steel mesh), in the third storey of the structural model of the composite flooring system used in the present investigation is shown in Figure 4. The overall depth of the slab is 130 mm, with the mesh placed 15 mm above the steel deck. The investigated building is designed as a non-sway frame with a central lift shaft and two staircases which had been braced to provide necessary resistance to lateral loads such as construction and wind loads. The aforementioned composite floor is supported by beams and columns. The details and the location of different sections used for beams and columns are provided in Table 2.

Table 2 Sections used for beams and columns (El-Dardiry and Ji, 2006)

Floor element	Section	Dimensions (mm)
Beams	Beam 1	610x229x10 UB
Beams	Beam 2	356x171x51 UB
Beams	Beam 3	305x167x40 UB
Beams	Beam 4	254x146x31 UB
Columns	C1	254x245x89 UC
Columns	C1	305x305x137 UC

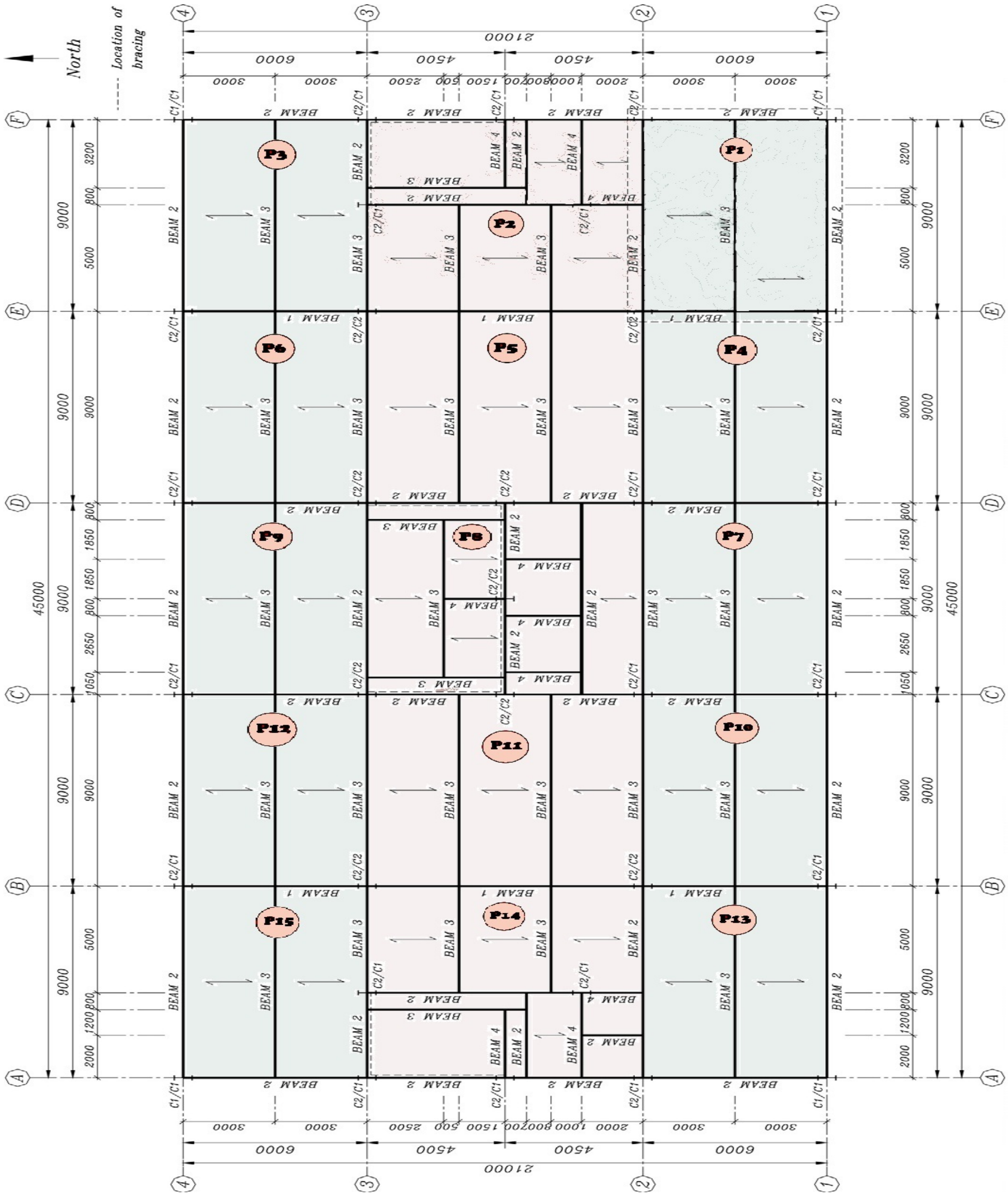

Figure 4 Typical plan of Cardington building (El-Dardiry and Ji, 2006)

6.2 Finite Element Model

For numerical investigations, a finite element (FE) model of the above mentioned composite floor was developed using commercial software package, SAP2000/Standard version 14.2. An eight-node solid element was allocated for concrete slab. It is based upon an isoparametric formulation that includes nine optional incompatible bending modes. For profile sheet, four node quadrilateral shell element was employed considering Kirchhoff theory (thin plate). The primary and secondary steel beams and columns have all been modeled utilizing linear beam elements 3DOF. In addition, according to Mello et al., (2008), it has been shown that the consideration of orthotrophy and its accuracy depends on the geometry of model and composite slab. Hence, in order to eliminate possible errors, the present composite floor is modeled without any material description simplification in the definition of shell and solids elements. Overall, the finite element mesh density contains 129461 joints, 4359 frame elements (beams and column with linear elements 3DOF which are provided in all intersections of solid and plate elements), 48422 shell elements and 87190 solid elements. With respect to dynamic response, all significant vertical vibration modes are aimed to be captured by restricting lateral translation in required connections. All connections are considered simply supported. Material properties of the floor are given in Table 3.

In order to reduce additional stiffness provided by either pinned or fixed supports, a floor-column model was considered (De Silva and Thambiratnam, 2009; El-Dardiry and Ji, 2006). The FE model of this steel-deck composite floor has been validated by comparing the numerical results for the first few fundamental natural frequencies with those of experimental and numerical works from literature (El-Dardiry and Ji, 2006). The comparison of the results between the present model, experiment and numerical works (El-Dardiry and Ji, 2006) can be seen in Figure 5 and Table 4. Additionally, the first four fundamental natural frequencies and associated mode shapes as simulated by current model are illustrated in Figure 5.

Table 3 Material properties of the floor (El-Dardiry and Ji, 2006)

Material	Young's modulus [GPa]	Poisson's ratio	Material density [kg/m^3]
Steel sheet	210	0.3	7800
Concrete	33.5	0.2	2000

Table 4 Comparison between obtained natural frequencies and literature results

Mode	Present study	Numerical model (El-Dardiry and Ji, 2006)	Experiment (El-Dardiry and Ji, 2006)
1	7.22	7.35	-
2	7.78	7.67	-
3	7.91	7.76	-
4	8.11	8.17	8.5
5	8.29	8.25	-
6	8.47	8.70	-

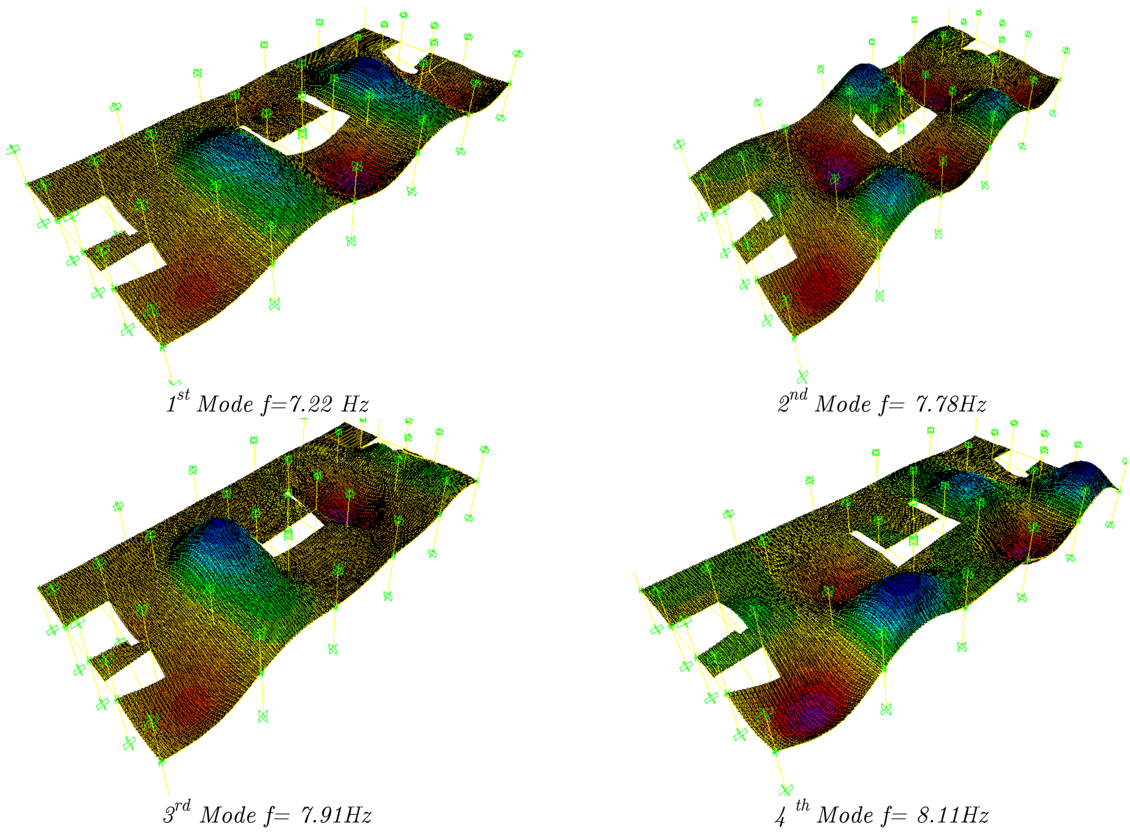

Figure 5 Fundamental natural frequency

7 PATTERN LOADS AND DAMPING RATIO

Based on equations 2 and 3, the dynamic load for individual and group jumping can be calculated. In this study live load comprises of two different types of dynamic and static loads. The first type is associated with jumping and is called "active live load" (ALL) and the second type is stationary live load and is called "passive live load" (PLL), the examples of which include human standing or sitting. For active live loads, two types of forces are considered: PL1 due to an individual jumping and PL2 due to 20 people jumping, both load cases are applied in panel 1. In all cases, normal jumping is applied and the frequency of jumping is considered in a range of 1.8Hz to 3.5Hz. The average value of person's weight is taken as 70 kg. Damping ratio is estimated to be 2 % based on the suggestion of Eliss et al. (2010) by considering live load.

8 DYNAMIC ANALYSIS

For the sake of comparison, two types of panels are presented herein. Activity Panel (AP) which is subjected to jumping loads and Non-Activity Panels (NAP) for those do not experience any jumping load.

In order to find out the number of essential modes for dynamic analysis, a comprehensive vibrational analysis was done by considering individual jumping load at the mid location of panel 1 which gives the maximum deflection values. Dynamic amplification factor (DAF) and the response of the composite floor in the loaded panel through different modal analysis were gained. The dynamic analysis of the structure has been performed in six different circumstances considering 6, 12, 18, 24, 30, and 36 numbers of modes. From investigation of the response of the structure in each model and comparison of results, the effect of higher modes, the minimum acceptable mass participation ratio, and sufficient mode numbers in vertical vibration analysis for this structure which may be applicable for same composite floors and further analysis, can be determined.

To order to find out the number of essential modes for dynamic analysis, a comprehensive vibrational analysis was done by considering individual jumping load at the mid location of panel 1 which gives the maximum deflection values. Dynamic amplification factor (DAF) and the response of the composite floor in the loaded panel through different modal analysis were gained. The dynamic analysis of the structure has been performed in six different circumstances considering 6, 12, 18, 24, 30, and 36 numbers of modes. From investigation of the response of the structure in each model and comparison of results, the effect of higher modes, the minimum acceptable mass participation ratio, and sufficient mode numbers in vertical vibration analysis for this structure which may be applicable for same composite floors and further analysis, can be determined:

$$\text{Allowable DAF} = 30/\ \text{static deflection}$$

In addition to aforementioned parameters, human-structure interaction is investigated in the present work. Considering Brownjohn (1999), parameters of human body (f = 4.9Hz, damping ratio = 37 %, and weight of a normal human = 80 kg) are defined as a SDF equivalent system. Passive live load (PLL) with 80 kg/m^2 intensity is subjected to above mentioned panels (10, 11, 12, 13, 14, and 15) while other parameters, jumping load and etc, have been preserved as constant. In this model, which will be called model three, passive live load is modeled as a spring or a link in which human body is considered as a SDF spring system with its mass, in contrast to preceding models. In order to find out the influence of this simulation method, the results of dynamic analysis (DAF, displacement, and acceleration) are compared with preceding obtained results from model of the intensity of 80 kg/m^2 that are simply considered as an additional mass. Note passive live load was just considered as additional mass in previous models.

The structural responses in terms acceleration and DAF are obtained in three different groups of panels: The responses of the activity panel (AP), responses related to the non-activity panel (NAP) adjacent to activity panel (panel 5), and responses in the non-activity panel in which the composite floor experiences the presence of the passive live loads (panel 11). All the maximum acceleration responses in above mentioned panels are used to determine the appropriate occupancies of the studied panels that complied with human perceptibility criteria mentioned in AISC Design Guide11.

9 RESULTS AND DISCUSSION

9.1 Effect of considering higher modes regarding Pattern Load 1 (PL 1)

Figure 6 illustrates all possible Dynamic Amplification Factors (DAFs) of dynamic analysis for different numbers of modes in panel 1. Results for Dynamic Amplification Factors (DAFs) obtained from different numbers of modes indicate that they have high dependency on the number of considered modes. It is worth mentioning that neither 12 modes nor 6 modes are adequate for this composite floor dynamic analysis. It should be noted that both above stated cases cannot capture all possible resonant peaks in panel 1. In other models, composite floor is excited due to jumping load with three resonant peaks while the models with 6 and 12 modes are excited only in the first two frequencies, 2.4Hz and 2.7Hz respectively. In these cases 3.3Hz is the frequency which is discounted. The present composite floor is excited in the third harmonic of jumping load that is because models with lower numbers of modes (6 and 12 modes) are not excited with load frequency of 3.3Hz. Because the fundamental natural frequency of the composite floor in the first mode of vibration is started from 7.22 Hz hence no frequency of jumping load in the first and the second harmonic could excite the composite floor. As a result, 3.3Hz load frequency of jumping in the third harmonics of jumping load could excite the mode of the composite floor whose natural frequency is close to 9.9Hz. This can be properly addressed and seen in the FFT curve plotted in Figure 7. On the other hand, models with 6 and 12 numbers of modes do not include this frequency (9.9Hz) in which the composite floor could be excited. In addition, numerical values of DAF in these two models do not reach the possible value of DAF which could be exerted. This is because of low mass participation ratio in these models (see Table 5).

Table 5 Properties of models with different numbers of vibration modes

Number of modes	Mass participation ratio	DLF’s amplitude at 2.7Hz
6	9.56%	1.07
12	18.67%	1.09
18	45.87%	1.1
24	65.19%	1.1
30	72%	1.12
36	88%	1.12

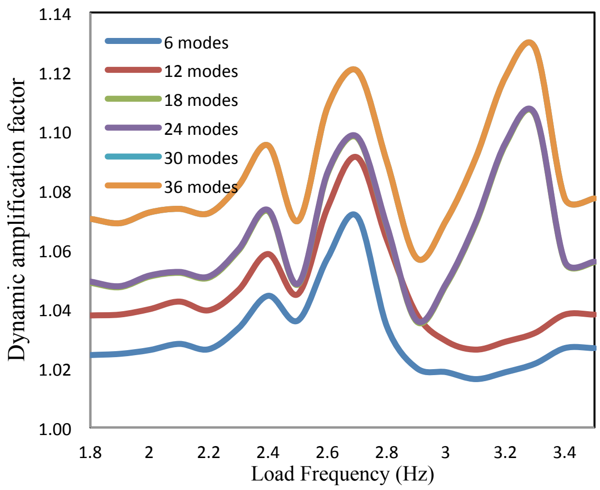

Figure 6 Dynamic amplification factors in panel 1 due to PL1

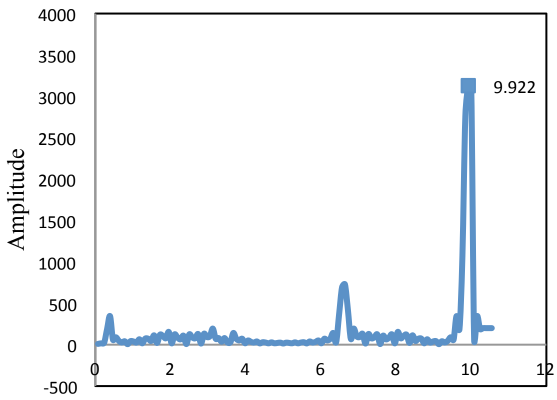

Figure 7 Fourier Amplitude Spectrum Analysis due to 3.3Hz load frequency of PL1

Although models with 18 and 24 modes could excite the composite floor in all possible critical frequencies and reach the three possible main peaks of vibration (2.4Hz, 2.7Hz, and 3.2Hz), because of inadequate mass participation ratio could not reach the ultimate values of displacements in these peaks. Despite an increase in the mass participation ratio from model with 18 numbers of modes to model with 24 numbers of modes, there is no important change in the displacement values. It implies that though there is high increase in the mass contribution from model with 18 modes to model with 24, this contribution due to behavior of mode shape does not have important effect on the panel 1 displacements and responses. Noticeably, 30 modes seem to be sufficient consideration of modes numbers to be used. It can be seen that there are no considerable changes in DAF amplitude and displacements of the model with 30 modes compared to the model with 36 modes. From these observations, 30 modes with 72% mass participation ratio and 1.12 DAF's amplitude are considered having reliable mode numbers for this composite floor and the next sets of analysis.

9.2 Response of Activity Panel (AP, i.e. P 1) due to different intensities of Passive Live Load (PLL1 to PLL7, subjected to panels 10 to 15) and Pattern Load 2 (PL 2)

From DAF curves shown in Figure 8, it is quite clear that in all cases of PLL, without presence of PLL (PLL1) and with different intensities of PLL (PLL2 - PLL7), three distinctive peaks can be observed. It is remarkable to state that in all different values of PLL, first and third peaks turned out to produce the same amplitude of DAF at similar load frequencies which are 2.4Hz and 3.2Hz, respectively. It is interesting that, load frequency of 2.4Hz could cause resonance at modes 1, 2, 3, 4, 4, 5, and 6 for PLL1 to PLL7, respectively. Figure 9 illustrates this phenomenon through an FFT analysis. FFT analysis showed that load frequency of 2.4Hz in its third harmonic excites the composite floor with natural frequency of 7.188Hz.

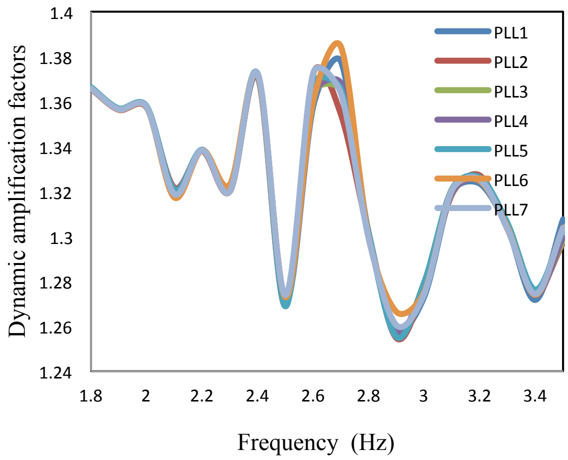

Figure 8 Dynamic amplification factors in panel 1 due to different PLLs

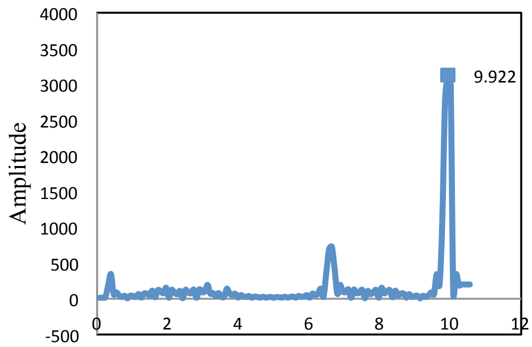

Figure 9 Fourier amplitude spectrum analysis due to 2.4Hz load frequency of PL2

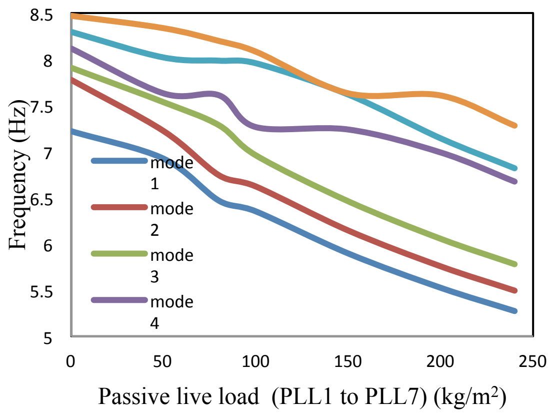

Figure 10 Natural frequency variations due to PLLs in first six modes

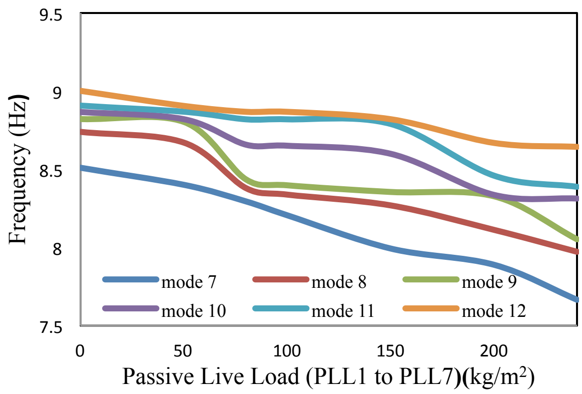

Figure 11 Natural frequency variations due to PLLs from 7th mode to 12th mode

From 10 and Figure 11 (the variations of natural frequencies associated with variation of intensity of passive live loads in the 12 first modes of vibration), it can be realized that the range of variations of natural frequencies due to variations of intensity of PLL in the first six modes of vibration are quite noticeable (see Figure 10). That is why load frequency of 2.4Hz could excite the composite floor in a wide variety of different modes in the first six modes (1^{th} to 6^{th}). As aforementioned, this load frequency could cause a resonance at 7.2 Hz of the composite floor natural frequency. Therefore, this frequency (7.2Hz) occurs in different modes of each case of PLL and the range of variation is from first to sixth mode corresponding to the intensity of PLL. On the contrary, load frequency of 3.2Hz excites the composite floor only at two different modes, 15^{th} or 16^{th} modes, in all ranges of intensity of PLL. This phenomenon indicates that in higher modes the effect of intensity of PLL is insignificant and the rate of increase in the natural frequency in higher vibration modes reduces substantially. This implies that higher stiffness is found for the composite floor when submitted to the different intensities of PLL in higher modes of vibration.

It should be noted that the most important peak was the second one which is associated with load frequencies of 2.7Hz or 2.6Hz. While there is no PLL (PLL1) applied to the composite floor, this peak happens at 2.7Hz. In other cases of PLL (PLL2 to PLL7) this frequency of peak occurs somewhat alternately at 2.6Hz or 2.7Hz. A regular decline in the DAF amplitude is expected, corresponding with a rise in intensity of PLL. But, there is an observable exception in the case of 200 kg/m^2 of intensity of PLL with the introduction of 1.384 as DAF amplitude at 2.7Hz. And the second higher peak, as is expected, occurs in absence of PLL (PLL1) at 2.7Hz frequency as mentioned previously. The other DAF amplitude, related to the rest of intensities of PLL, show a normal compliance with expected trends. Generally, the variations in DAF amplitude in the activity panel are not that high because of considerable distance from panels subjected to PLL.

Table 6 Maximum Accelerations in the panel 1 (m/s^2)

Resonant point	PLL1	PLL2	PLL3	PLL4	PLL5	PLL6	PLL7
(2.4Hz)	1.56	1.56	1.56	1.56	1.56	1.56	1.56
(2.7Hz and 2.6 HZ)	1.99	1.71	1.8	1.61	1.66	2.15	1.71
(3.2Hz)	1.77	1.77	1.77	1.77	1.77	1.77	1.77

Table 6 shows the maximum accelerations in panel 1 for the three evident peaks of resonance happened due to group jumping load (PL 2). Similar trends to the DAF curve are quite obvious here. Maximum accelerations due to 2.4Hz and 3.3Hz frequencies of dynamic load are constant in all cases of PLL with 1.56 m/s^2 and 1.7 m/s^2, respectively. On the other hand, the accelerations associated with the load frequency of 2.7Hz which possess almost the highest acceleration, vary in different intensities of PLL. DAFs trend (Figure 8) does not match thoroughly the trend of accelerations variations. However, the highest acceleration values are related to the load frequency of 2.7Hz, in some few cases of PLL the maximum accelerations emerge at the load frequency of 3.2Hz which had the lowest DAF's amplitude among the three main peaks of DAF.

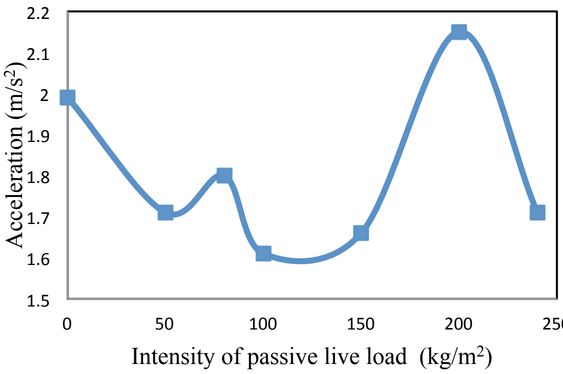

Figure 12 Maximum accelerations for second resonant peak due to different PLLs in panel 1

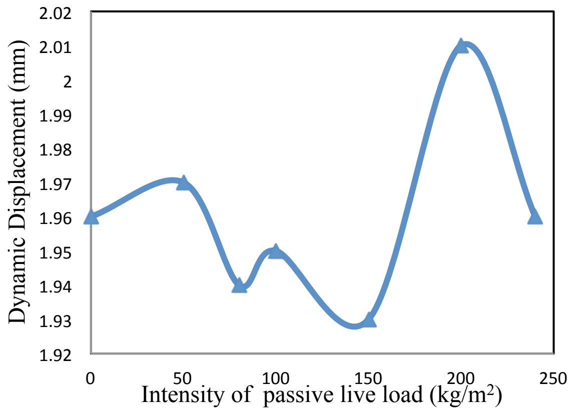

Figure 13 Maximum dynamic displacements for second resonant peak due to different PLLs in panel 1

For the sake of comparison between acceleration and dynamic displacement trends (i.e. displacements due to jumping load without considering effects of self-weight and PLL) considering the second peak of resonance (i.e. 2.7 Hz and in some cases 2.6Hz), Figure 12 and Figure 13 can be referred. In general, to some extent both trends have depicted similar trends with some differences. A descending trend with the presence of PLL (PLL2 to PLL7) for the acceleration trend is obvious, except a significant rise in PLL6 which presents the highest value of acceleration. Apart from a gentle increase of dynamic displacement in PLL2, the rest of dynamic displacement curve match the acceleration graph. Moreover, both graphs (i.e. acceleration and displacement graphs) experience a slight peak during their descending behavior in PLL3 and PLL4, respectively.

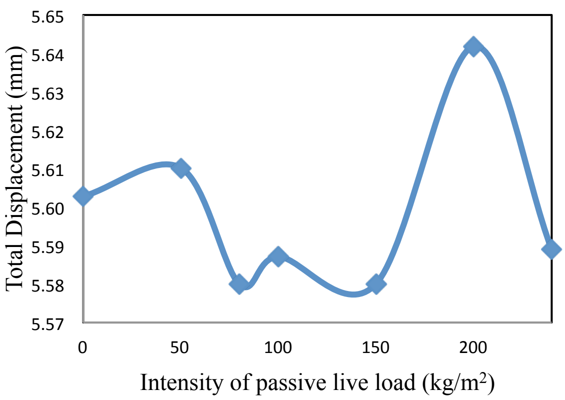

Figure 14 Maximum total displacements for second resonant peak due to different PLLs in panel 1

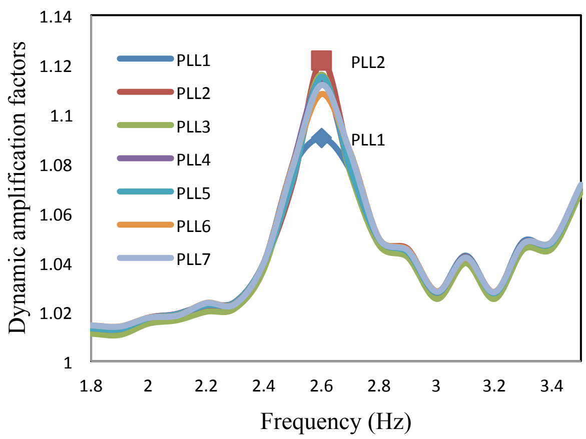

Figure 15 Dynamic amplification factors in panel 5 due to different PLLs

In view of comparison between total displacement (Figure 14) and dynamic displacement, it can be summarized that although the presence of PLL (PLL2 to PLL7) causes upward deflections in this activity panel, generally both trends are similar.

Table 6 summarizes that although in all cases of PLL maximum DAF amplitude are related to the second peak of DAF curve, maximum acceleration values at the second peak do not show the highest value in the all cases of PLL. But it can be mentioned that the most severe one which happens at the load case of 200 kg/m^2 belongs to the second peak of DAF.

9.3 Response of Non-Activity Panel (NAP, i.e. P 5) due to different intensities of Passive Live Load (PLL1 to PLL7, subjected to panels 10 to 15) and Pattern Load 2 (PL 2)

As can be observed in Figure 15, panel 5 in the all cases of PLL is excited only at one peak of resonance which is observed at the unique frequency of load, 2.6 Hz. On the contrary with activity panel (P 1), the minimum DAF amplitude of the present panel occurs in the PLL1 (i.e. zero passive live load). Immediately after presence of the passive live load (PLL2), the DAF amplitude reaches its maximum value. The rest of the other load cases in terms of DAF amplitude are between the above mentioned load cases. The presence of passive live load (PLL2 to PLL7) could affect mode shape of the composite floor in a way that causes severe displacement in PLL2 load case.

The variation of the maximum accelerations in the present panel corresponding to the different PLL is depicted in Figure 16. It is obvious that the behavior of the acceleration trend of the present panel is quite in contrast to panel 1. When the composite floor is subjected to the passive live load (PLL2), a drastic increase in acceleration value can be seen. From 1.4 m/s^2 at PLL2, values of acceleration drop gradually to 1.29 m/s^2 at PLL7. In general, submission of passive live load (PLL2 to PLL7) to the composite floor in panel 5 causes a significant increase in acceleration. Total displacement curve shown in Figure 17 presents a similar manner to the trend of the acceleration variation in the present panel

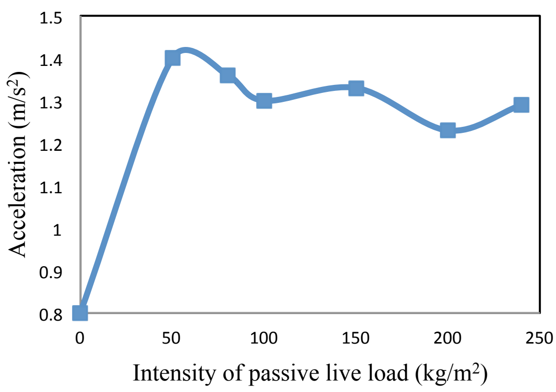

Figure 16 Maximum accelerations for resonant peaks to different PLLs in panel 5

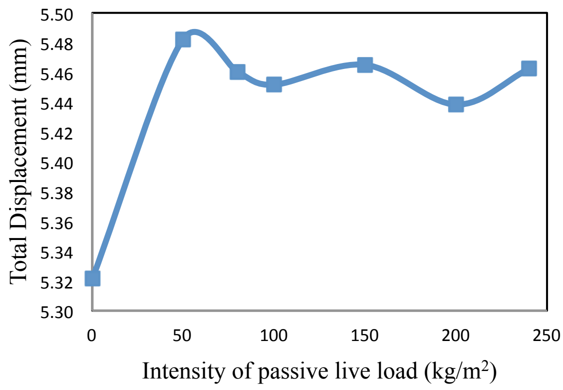

Figure 17 Maximum total displacements for resonant peaks due to different PLLs in panel 5

9.4 Response of Non-Activity Panel submitted to Passive Live Load (NAP, i.e. P 11) due to different intensities of Passive Live Load (PLL1 to PLL7, subjected to panels 10 to 15) and Pattern Load 2 (PL 2)

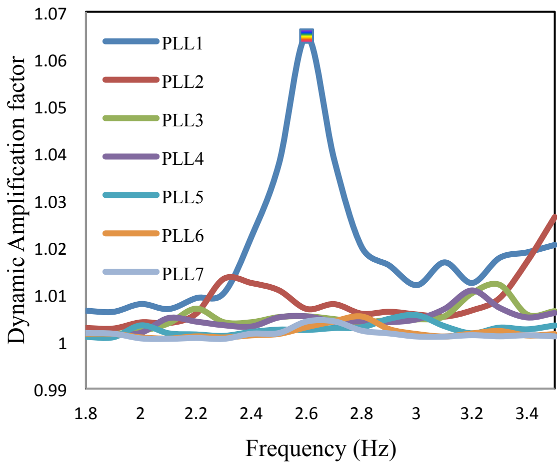

Figure 18 Dynamic amplification factors in panel 11 due to different PLLs

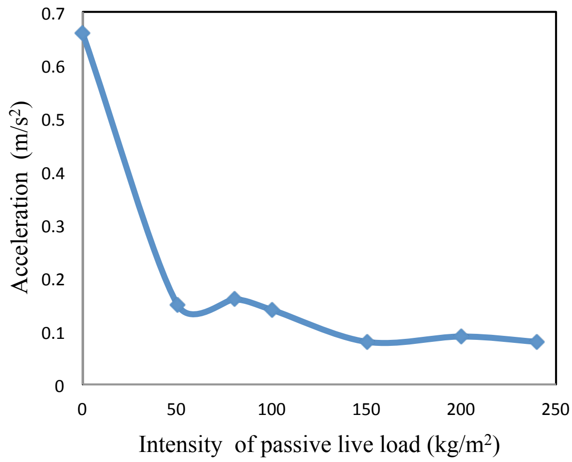

Figure 19 Maximum accelerations values due to different PLLs in panel 11

In Figure 18, it is realized that while there is no passive live load submitted on the composite floor, the present panel is excited at 2.6 Hz frequency of jumping load with 1.065 amplitude of DAF, which is quite lower than preceding investigated panels. That is due to the notable distance of the present panel to the activity panel. In addition, there is no considerable peak in the presence of the passive live loads (PLL2 to PLL7) in comparison with PLL1. Passive live loads (PLL2 to PLL7) are submitted to the present composite floor directly and it gives rise to higher flexibility for this panel. This phenomenon causes a reduction of DAF amplitude in the present panel. However, there are irregular small peaks at different load frequencies and the range of these frequencies is from 2Hz to 3Hz.

Table 7 Maximum accelerations in panel 11 (m/s^2)

resonant point	PLL1	PLL2	PLL3	PLL4	PLL5	PLL6	PLL7
2 Hz	-	-	-	-	0.04	-	-
2.1 Hz	-	-	-	0.06	-	-	-
2.3 Hz	-	0.15	-	-	-	-	-
2.6 Hz	0.66	-	-	0.1	-	-	0.08
2.8 Hz	-	-	-	-	-	0.09	-
3 Hz	-	-	-	-	0.08	-	-
3.1 Hz	-	-	-	-	-	-	-
3.2 Hz	-	-	-	0.14	-	-	-
3.3 Hz	-	-	0.16	-	-	-	-
maximum	0.66	0.15	0.16	0.14	0.08	0.09	0.08

With respect to the existence of the multi peaks for each load case in DAF curve, maximum accelerations are taken in each case of loading which are shown in Table 7. Maximum acceleration curve associated with the highest value of Table 7 is displayed in the Figure 19. Submission of PLL2 to the composite floor gives rise to a dramatic drop in the value of maximum acceleration with some fluctuations thereafter.

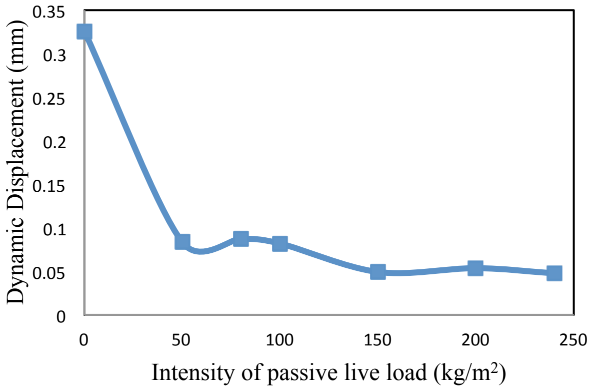

Figure 20 Maximum dynamic displacements values due to different PLLs in panel 11

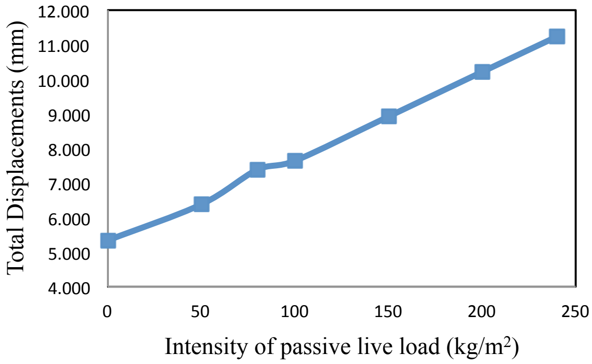

Figure 21 Maximum total displacements values due to different PLLs in panel

Maximum dynamic displacements curve in the present panel, which is illustrated in the Figure 20, show a similar trend to the maximum acceleration curve of this panel. On the other hand, total displacements curve in the Figure 21 shows an evident contrast to the maximum dynamic displacements curve. The total displacement curve increases gradually with the presence of PLL2 to PLL7 whereas an opposite behavior for maximum dynamic displacements curve can be seen from PLL2 to PLL7. This observation indicates the direct effect of the distance between the present investigated panel and activity panel. In the present panel, due to unimportant effects of the dynamic loading compared to the effect of the intensity of passive live loads, the total displacements are just affected by the intensity of passive live load. In other words, increase of intensity of live load causes a direct rise in the displacements values.

Based on the fundamental theory ($F = ma$), a gradual reduction in the values of the maximum accelerations is expected when the mass of the composite floor is increased. As mentioned earlier, in all cases of the present analysis for different intensities of PLL, the jumping load (PL2) is constant. As a result, produced energy due to the dynamic loading in all cases of different PLL is kept unchanged. In general, as expected, with rise in mass, acceleration values drop. As can be seen in the Figure 19, apparently in some cases of PLL for the present panel, trend of the acceleration variations does not comply with the theory particularly in the present panel at PLL3 which has greater value of acceleration than PLL2. Moreover, in panel 1 and panel 5, unexpected significant increases in acceleration values are obvious at PLL6 and PLL2, respectively.

For the case of panel 11, it can be seen that selected accelerations for Figure 19 are chosen from a wide variety of different frequencies of maximum accelerations. Then, the produced energies vary from one case to another. Panel 11 of the present composite floor in PLL2 is excited at 2.3Hz frequency and with 80 kg/m^2 it is excited at 3.3Hz. These load frequencies excite the composite floor in the third harmonic of jumping load. So, regarding to natural frequencies of these cases it is found that the 50 kg/m^2 load case (PLL2) is excited in the first mode while the 80 kg/m^2 is excited at 17th mode with higher value mass participation compared to PLL2. In this case, the most important effect of additional mass is on the variation of mode shapes and the frequency of possible peaks. These two abovementioned cases are close in terms of the mass of the structure rather than frequency of excitation. As a result, with a small difference in mass, a considerable difference in excitation frequency is found.

But in the case of panel 1 and panel 5, although produced energies for selected higher accelerations in each particular panel are resulted from similar frequency (i.e. load frequency of 2.7Hz for Figure 8 in panel 1 and load frequency of 2.6Hz for Figure 15 in panel 5), the input energy related to each of passive live load varies from each other. For example, input energies for PLL1, PLL2, and PLL6 in the frequency of 2.7Hz are 135625 N-mm, 128892 N-mm, and 145687 N-mm. Meanwhile, in the case of maximum acceleration, resulted kinetic energy is higher than other cases while the modal damping has the lowest value. As a result, considering modal damping and the input energy associated with modal mass, the observed trend seems to be rational.

10 EFFECT OF HUMAN-STRUCTURE INTERACTION ON THE RESPONSE OF THE COMPOSITE FLOOR

10.1 Response of Activity Panel (AP, i.e. P 1) due to 80 kg/m^2 intensity of Passive Live Load (PLL1 to PLL7, subjected to panels 10 to 15) and Pattern Load 2 (PL 2)

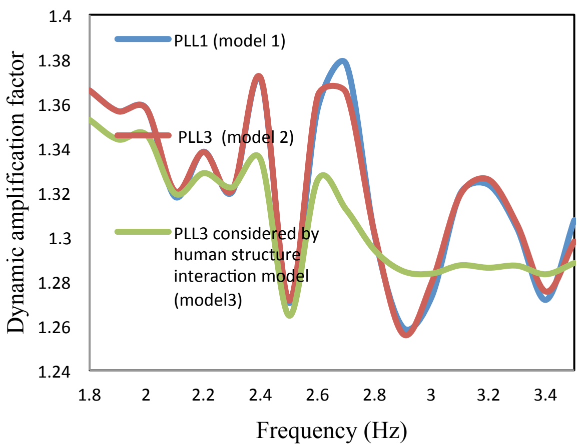

Figure 22 Dynamic amplification factor in panel 1 considering three different models (1, 2, 3)

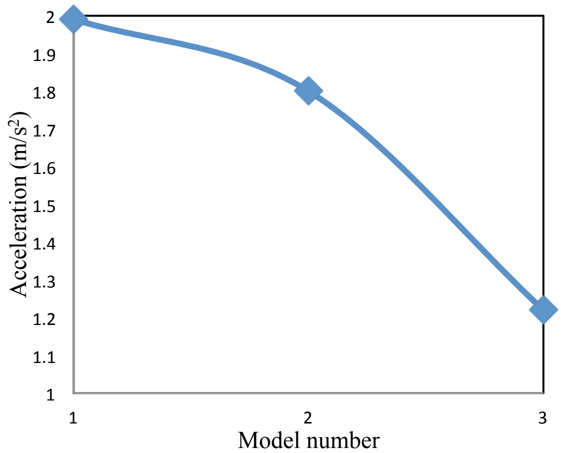

Figure 23 Maximum accelerations in panel 1 considering models 1, 2, and 3

Considering human body properties on the non-activity panels (panel 10 to panel 15) in the modeling procedure of the passive live load of 80 kg/m^2 submitted to the composite floor resulted in interesting findings. Figure 22 and Figure 23 depict the irrefutable effect of the human body interaction with structure on the obtained responses for DAFs and accelerations. By including the human body and structure interaction in the simulation procedure, it is found that damping ratio of the structure has a noticeable increase compared to the ordinary model with same condition of passive live load (PLL2). In contrast to the other models, third peak of the resonance vanishes herein. First point and second point of DAF curve are observed at 2.4Hz and 2.6Hz, respectively. The peak resulted from 2.6Hz has lower DAF amplitude compared to the first peak related to 2.4Hz load frequency. In general, a drop in DAF amplitude can be seen. It should be noted that in the present panel, all other cases with different intensities of passive live load at the first peak at 2.4Hz have unique amplitude but in this model the amplitude of the load frequency of 2.4Hz drops.

In the case of maximum accelerations, as can be seen in the Figure 23, models of PLL1(model 1) and PLL3 considering passive live load as an additional mass (ordinary model) are compared to the model of PLL3 in which human body interaction with structure had been included (model 3). Complying with expectations, results show a descending trend from 1.99 m/s^2 for model 1 to 1.22 m/s^2 for model 3.

Table 8 Acceleration in panels 1 considering human-structure interaction (m/s^2)

Resonant frequency	PLL1	PLL3	PLL3 considering human structure interaction
2.4Hz	1.56	1.56	1.2
2.6Hz or 2.7Hz	1.99	1.8	1.22
3.2Hz	1.77	1.77	-

Table 9 Displacements in panel 1 considering human-structure interaction

Resonant frequency	PLL1	PLL3	PLL3 considering human structure interaction
2.4Hz	1.96	1.96	1.81
2.6Hz or 2.7Hz	1.96	1.94	1.78
3.2Hz	1.77	1.77	-
maximum	1.96	1.96	1.81

From Table 8 and Table 9, it can be summarized that the trend of variation of displacements is similar to the acceleration graph. It should be noted that both frequencies (2.4Hz and 2.6Hz) in the third model have noticeable reductions in the acceleration values whereas model 2 does not show any change in the acceleration and displacement values compared to model 1 at frequencies 2.4Hz and 3.2Hz.

10.2 Response of Non-Activity Panel (NAP, i.e. P 5) due to 80 kg/m^2 intensity of Passive Live Load (PLL1 to PLL7, subjected to panels 10 to 15) and Pattern Load 2 (PL 2)

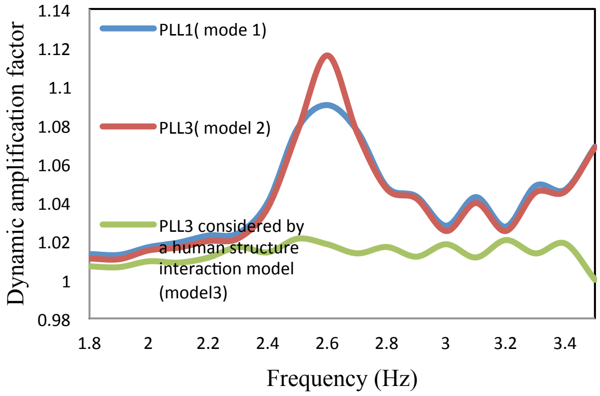

Figure 24 Dynamic amplification factor in panel 5 considering three different models (1, 2, 3)

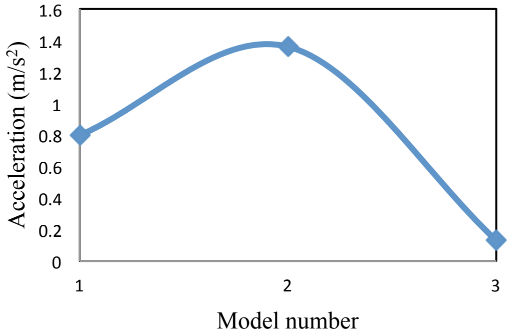

Figure 25 Maximum accelerations in panel 5 considering models 1, 2, and 3

By investigating Figure 24, DAF curves, it is found that human-structure interaction model could affect notably on the DAF curve. Considering this interaction, there is no longer important peaks. In this case, just peaks remain with a fluctuation in the range of 2Hz to 3.4Hz whose maximum peaks occur at 2.5Hz of the jumping load frequency with 1.02 amplitude of DAF. In addition, maximum accelerations of consideration of human-structure interaction, model 3, significantly drop to 0.1 m/s^2 from a maximum point of 1.36m/s^2 in model 2 which is displayed in Figure 25.

10.3 Response of Non Activity Panel submitted to Passive Live Load (NAP, i.e. P 11) due to different intensity of Passive Live Load (PLL1 to PLL7, subjected to panels 10 to 15) and Pattern Load 2 (PL 2)

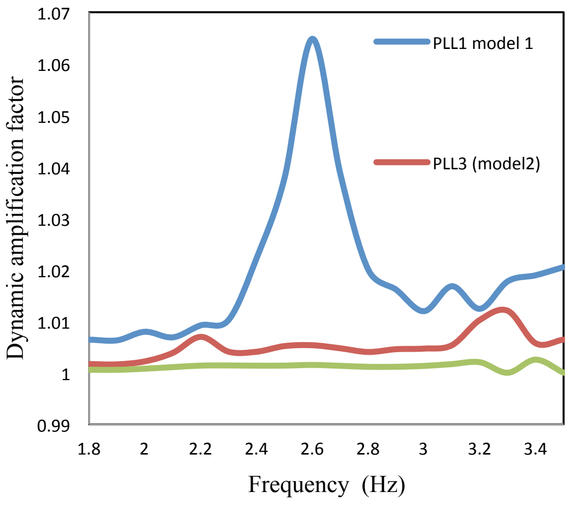

Figure 26 Dynamic amplification factors in panel 11 considering three different models (1, 2, 3)

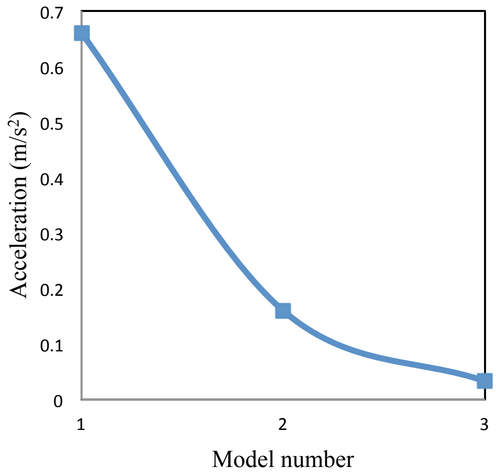

Figure 27 Maximum acceleration in panel 11 considering model 1, 2, 3

Figure 26 shows DAF curves in the present panel that illustrated a dramatic plunge in the amplitude of DAF. In fact, the DAF curve changes to a complete steady line and there is no peak in the model when human-structure interaction is included. Consequently, maximum accelerations values diminish dramatically and reduce to 0.033 m/s^2 for model 3. The variations of the accelerations trend can be seen in Figure 27.

10.4 Modal damping

In general, total energy component of the structure can be written as below:

$$\text{Input energy} = \text{kinetic energy} + \text{potential energy} + \text{modal damping}$$

In order to figure out the effect of the link model of human body on the composite floor, it is possible to compare the modal damping of cases, additional mass of passive live loads, and mass as a spring link.

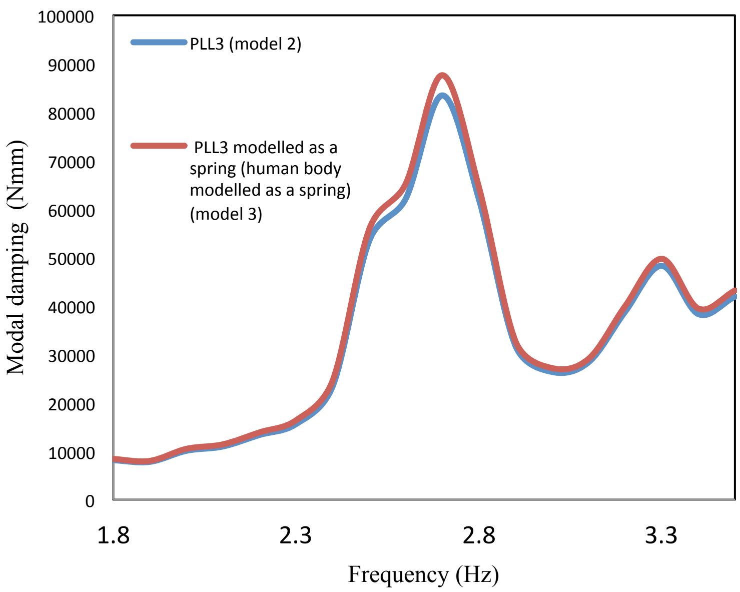

Figure 28 Modal damping in two different models of passive live load

Figure 28 depicts the differences of presented models (ordinary model of human mass, model 2, and spring model of human mass, model 3) which can be seen in Table 10 in numerical form. Generally, human-structure model could increase modal damping in the range of 2.5 to 5 %.

Table 10 Modal Damping for mode1s 2 and 3 (N-mm)

Frequency (Hz)	Model 2(PLL3)	Model 3 (PLL3 through considering human structure interaction)	rate of increase in modal damping
1.8	8144.96	8389.3088	1.03
1.9	7811.49	8006.77725	1.025
2	10088.83	10492.3832	1.04
2.1	10983.94	11423.2976	1.04
2.2	13357.77	13892.0808	1.04
2.3	15612.46	16393.083	1.05
2.4	23405.84	24576.132	1.05
2.5	53426.39	55830.57755	1.045
2.6	62149	65256.45	1.05
2.7	83460.71	87633.7455	1.05
2.8	62000	64480	1.04
2.9	31733.56	32685.5668	1.03
3	26370.48	27161.5944	1.03
3.1	28381.16	29090.689	1.025
3.2	38816.35	39980.8405	1.03
3.3	48278.54	49726.8962	1.03
3.4	38304.79	39453.9337	1.03
3.5	41920.98	43178.6094	1.03

11 REMARKS

In order to evaluate different investigated panels from the aspect of DAF's amplitude, limits are defined and have been compared to the DAF amplitude obtained from dynamic analysis using the present model. Referring to the preceding texts of the present study, acceptable values of DAF amplitude can be taken as presented values in the following for the different panels:

- DAF amplitude limit for panel 1 in all cases = conservatively about 6-7.
- DAF amplitude limit for panel 5 in all cases of loading = approximately 5-6.
- DAF amplitude limit for panel 11 = 2.5 to 6 corresponding to different PLLs

As a result, in all presented cases, no DAFs' amplitude found from the numerical models has exceeded the limit values of DAF.

We have given extensive treatment to the computation of several dynamic characteristics and responses of the composite floor. We now wish to present in Table 11 a summary of acceptable types of occupancy in each investigated panel, based on Murray (1975) and AISC Design Guide 11 criteria. Each discussed panel is investigated for all possible passive live load cases. Table also includes the model with the consideration of human-structure interaction.

Table 11 Summary of occupancy fit-outs for various load consideration

Panel	PLL1 (zero Passive Live Load)	50 kg/m^2 (Passive Live Load)	80 kg/m^2 (Passive Live Load)	100 kg/m^2 (Passive Live Load)	150 kg/m^2 (Passive Live Load)	200 kg/m^2 (Passive Live Load)	240 kg/m^2 (Passive Live Load)	80 kg/m^2 Human-structure interaction model 0f (Passive Live Load)
Panel 1	Occupancy (U)	Occupancy (U)	Occupancy (U)	Occupancy (U)	Occupancy (U)	Occupancy (U)	Occupancy (U)	Occupancy (U)
Panel 5	Occupancy (1)	Occupancy (U)	Occupancy (U)	Occupancy (U)	Occupancy (U)	Occupancy (U)	Occupancy (U)	Occupancy (2)
Panel 11	Occupancy (1)	Occupancy (2)	Occupancy (2)	Occupancy (2)	Occupancy (3)	Occupancy (3)	Occupancy (3)	Occupancy (3)

[a]Occupancy (U) Uncomfortable
[b]Occupancy (1) Rhythmic activities / aerobics / dance- type loads
[c]Occupancy (2) Shopping malls (centres) / weightlifting / Stores / manufacturing / warehouse /walkways/ stairs
[d]Occupancy (3) Office / residencies / hotels / multi - family apartments / school rooms / libraries
[e]Occupancy (4) Hospitals / laboratories / critical working areas (e.g. operating theatres, precision laboratories)

In general, there is no comfortable condition in panel 1 due to all different passive live load cases. Furthermore, submitting any intensity of passive live load cases on the composite floor brings uncomfortable occupancies as well. If there is no PLL on the composite floor, panels 5 and 11 could obtain occupancy (1) as a comfort condition. It is interesting to note that in spite of uncomfortable description for panel 5 in presence of all PLL cases, when human-structure interaction is considered, a level of comfort with occupancy (2) is achieved for composite floor.

Contrary to panel 5, by introducing passive live load (PLL) cases to the composite floor, the level of comfort in panel 11 has been increased from occupancy 1 in NLL load case to higher levels of comfort, 2 and 3. Also, consideration of human-structure interaction has mitigated the level of comfort from occupancy (2) to occupancy (3) for PLL3 (80 kg/m^2) load case.

12 CONCLUSIONS

Dynamic characteristics of the composite floor system of Cardington steel frame building subjected to the individual and group jumping loads have been examined using finite element modeling technique. Different intensities of passive live load are applied and the resulting responses of the structure in the activity panel and non-activity panels have been investigated. In one of the cases, effect of human-structure interaction has been incorporated and explored. Higher modes of vibration are found to be significant and they have been taken into account in the dynamic analysis. Obtained results have been used to evaluate the suitability of different panels, activity panel (AP) and non-activity panel (NAP), for different occupancies. The main findings of this study can be listed as followings:

- In addition to the fundamental modes, it is essential to consider higher modes of vibration for composite floor particularly in real composite floor with notable dimensions as they can participate in all possible cases of excitation by the higher harmonic of human activity resulting in multi modal vibration. It is found that, in contrast to previous studies, current investigation covers all possible vibration modes through the consideration of 30 modes.
- Acceptable mass participation ratio, in order to capture all serious vibration modes, is 72 % which can be applicable to the other similar composite floors with dimensions that are significantly large.
- First and second harmonics do not cause vibration problems in all considered cases for this work. Possible vibration problems only occur at the third harmonic of jumping load. Consequently, for composite floor with large dimension and relative high natural frequencies, the significant vibration problems and resonance may occur for third harmonic of dynamic loading.
- Although the DAFs amplitude and deflection may found to be in the acceptable range, they cannot guarantee acceptable responses for the acceleration and level of comfort. Both deflection and acceleration criteria must be assessed.
- Presence of passive live load alone always does not always have the same effect on the behavior of activity panel. Depending on different intensities and locations of passive live loads, increase or decline in accelerations of composite floor are observed.
- Regarding to notable distance between activity panel and non-activity panels subjected to passive live loads, the variations of DAF amplitude in activity panel are not significant. For particular case of intensity and coordination of passive live load, the response of the structure may vary and must be investigated in each specific case. Generally, in activity panel, DAF amplitude is affected by intensity of jumping load rather than the intensity of passive live load.
- The responses of the non-activity panels vary from panel to panel depending on the distance from activity panel and intensity of passive live load as well as the type of non-activity panel in terms of being subjected to passive live load or not. For example, panel 5 in presence of passive live load in panel 11 experiences remarkable increase in acceleration whereas panel 11 in presence of passive live load undergoes significantly lower acceleration compared to PLL1 load case. So, it can be deduced that the most suitable panel to comply with requirement can be found by dynamic analysis through considering type of activity and intensity of passive live load.
- In general, human-structure interaction model, as a new approach in composite floor response simulation, contributes a rise in damping coefficient from 2.5% to 5% for 80 kg/m2 passive live load case. Consideration of this method of simulation may produce more realistically economical result for dynamic responses.

References

[1] A. Behnia, The finite element analysis of high modal dynamic responses of composite floor due to human motion induction, Master Thesis, University Technology Malaysia, 2012.

[2] A. Behnia, H.K. Chai, N. Ranjbar, N. Behnia, A. Fateh, N. Merabi, Effects of coordinated crowd motion on dynamic responses of composite floors in buildings, IES Journal Part A: Civil and Structural Engineering, Taylor & Francis, 6 (1) (2013) 1-17. DOI:10.1080/19373260.2013.738522.

[3] A. V. de A. Mello, J. G. S. da Silva, P. C. G. da S. Vellasco, S. A. L. de Andrade, L. R. O. de Lima, Modal analysis of orthotropic composite floors slabs with profiled steel decks, Latin American Journal of Solids and Structures, 5 (2008) 47–73.

[4] B. R. Ellis, T. Ji, E. El-Dardiry, T. Zheng, Determining the dynamic characteristics of multi-panel floors, The Structural Engineer 88 (18) 21 September 2010.

[5] B.R. Ellis, T. Ji, The response of structures to dynamic crowd loads, BRE Digest 426 (1997).

[6] B.R.Ellis, T.Ji, Floor vibration induced by dance-type loads: verification, The Structural Engineer 72 (3) (1994)45–50.

[7] British Standards Institution, BS 6399-1:1996, Loading for buildings, British Standard Institution, UK, 1996.

[8] C. Sahnaci, M. Kasperski, Random loads induced by walking, Structural DynamicsEURODYN2005, Vol.1, Millpress, Rotterdam2005, pp.441–446.

[9] C.J. Tilden, Kinetic effects of crowds, Proceedings ASCE 39 (3)(1913)325–340.

[10] Canadian Commission on Building and Fire Codes, User's Guide—NBC 2005: Structural Commentaries (Part 4ofDivisionB), National Research Council of Canada, Institute for Research in Construction, Ottawa, 2006.

[11] Canadian Commission on Building and Fire Codes, User's Guide—NBC 2005: Structural Commentaries (Part 4ofDivisionB), National Research Council of Canada, Institute for Research in Construction , Ottawa, 2006.

[12] D.E. Allen, Floor vibration from aerobics, Canadian Journal of Civil Engineering, 1990. 17: p.771-779.

[13] D.E. Allen, J.H. Rainer, G. Pernica, Vibration criteria for assembly occupancies, Canadian Journal of Civil engineering, (1985). Vol. 12, pp. 617-623.

[14] E. El-Dardiry, T. Ji, Modelling of the dynamic behaviour of profiled composite floors, Engineering Structures. 28 (2006) 567–579.

[15] H. Bachman, W. Ammann, Vibrations in structures induced by man and machines, Structural Engineering Document 3e, International Association for Bridges and Structural Engineering. (1987).

[16] H.Bachmann, W. Ammann, Vibrations in Structures Induced by Man and Machines, IABSE—AIPC—IVBH, Zurich, Switzerland,1987.

[17] International Organization for Standardization, ISO10137:2007, Bases for design of structures—serviceability of buildings and walk ways against vibration, 2007.

[18] ISO 2631 - 1, Evaluation of Human Exposure to Whole Body Vibration, 1997.

[19] IStructE/DCLG/DCMS Joint Working Group, Dynamic Performance Requirements for Permanent Grandstands: Recommendations for Management Design and Assessment, Institution of Structural Engineers, London, 2008.

[20] J. G. S. da Silva, P. C. G. da S. Vellasco, S. A. L. de Andrade, and L. R. O. de Lima, Dynamical response of composite steel deck floors ,Latin American Journal of Solids and structures, 3 (2006) 163-178.

[21] J.D. Littler, Frequencies of synchronized human loading from jumping and stamping, The Structural Engineer 2003 (22)(2003)27–35.

[22] J.G.S. Da Silva, P.C.G. da S Vellasco, S.A.L. de Andrade, F.J. da CP Soeiro, R.N. Werneck, An evaluation of the dynamical performance of composite slabs, Computers and Structures, 2003. 81: p. 1905-1913.

[23] J.H. Rainer, J.C. Swallow, Dynamic behaviour of a gymnasium floor, Canadian Journal of Civil Engineering 13 (3)(1986)270–277.

[24] J.H.H.Sim, A. Blakeborough, M. Williams, Modelling of joint crowd-structure system using equivalent reduced-DOF system, Shock and Vibration 14 (4) (2007) 261–270.

[25] J.M.W. Brownjohn, Energy dissipation in one-way slabs with human participation, Asia Pacific Vibration Confer-ence'99,Vol.1, Nanyang Technological University, 2nd SPIE International Conference on Experimental Mechan-ics,1999,pp.155–160.

[26] K. Lenzen, Vibration of steel joist concrete slab system, AISC Engineering Journal. 3 (3). 133-163.

[27] L. Wei, M.J. Griffin, Mathematical models for the apparent mass of the seated human body exposed to vertical vibration, Journal of Sound and Vibration 212 (5)(1998)855–874.

[28] M. Kasperski, Safety and reliability of stadia under men-induced vibrations, H.Grundmann,G.I. Schueller (Eds.), Structural Dynamics EURODYN'02 ,Vol.2, A.A. Balkema Publishers, Lisse/ Abingdon/Exton(PA)/Tokyo2002,pp.1223–1228.

[29] M.S. Williams, P. Waldron, Evaluation of methods for predicting occupant-induced vibrations in concrete floors. The Structural Engineer, 1994. 72(20): p. 334-340.

[30] R. Moreland, The weight of a crowd, Engineering. 79 (1905) 551.

[31] R. Sachse, A.Pavic, P.Reynolds, Parametric study of modal properties of damped two-degree-of-freedom crowd-structure dynamic systems, Journal of Sound and Vibration 274 (3–5)(2004)461–480.

[32] R. Sachse, The Influence of Human Occupants on the Dynamic Properties of Slender Structures, PhD Thesis, University of Sheffield, 2002.

[33] S. Sandun De Silva, P. David. Thambiratnam, Dynamic characteristics of steel–deck composite floors under human-induced loads, Computers and Structures.87 (2009) 1067–1076.

[34] S.O.Hansen, J.D.Sørensen, Dynamic loads due to synchronized movements of people, H.Grundmann,G.I. Schuellr(Eds.) Structural Dynamics EURODYN'02, Vol.2, A.A.Balkema Publishers, Lisse/Abingdon/Exton(PA)/Tokyo 2002, pp.1217–1222.

[35] SAP2000/14.2. Static and dynamic finite element analysis of structures, version 14.2.Berkeley (CA): Computers and Structures Inc.

[36] T. Ji, and B.R. Ellis, Floor vibration: floor vibration by dance type loads: theory. The Structural Engineer, 1994. 72(3): p. 37-44.

[37] T. Ji, D. Wang, A supplementary condition for calculating periodical vibration, Journal of Sound and Vibration 241 (5)(2001)920–924.

[38] T. Tredgold, Elementary Principles of Carpentry, 2nd Ed., Publisher unknown, 1828.

[39] T.A. Wyatt, Design guide on the vibration of floors. SCI Publication 076. 1989: Steel Institute. Berkshire. UK. , Construction Industry Research and Information Association. London UK.

[40] T.A.Wyatt, Floor excitation by rhythmic vertical jumping, Engineering Structures 7 (1985)208–210.

[41] T.M. Murray, D.E. Allen, and E.E. Ungar, Steel design guide series 11: floor vibration due to human activity. 1997, Chicago, USA: American Institute of Steel Construction, Inc.