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Dynamic response of low frequency Profiled Steel Sheet Dry Board with Concrete infill (PSSDBC) floor system under human walking load

Farhad Abbas Gandomkar*, Wan Hamidon Wan Badaruzzaman and Siti Aminah Osman

Department of Civil & Structural Engineering, Universiti Kebangsaan Malaysia, Bangi, Selangor – Malaysia

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* Author email:
farhad.abbas.gandomkar@gmail.com

Abstract

This paper investigates the dynamic response of a composite structural system known as Profiled Steel Sheet Dry Board with Concrete infill (PSSDBC) to evaluate its vibration serviceability under human walking load. For this point, thirteen (13) PSSDBC panels in the category of Low Frequency Floor (LFF) were developed using Finite Element Method (FEM). The natural frequencies and mode shapes of the studied panels were determined based on the developed finite element models. For more realistic evaluation on dynamic response of the panels, dynamic load models representing human walking load were considered based on their Fundamental Natural Frequency (FNF), and also time and space descriptions. The peak accelerations of the panels were determined and compared to the limiting value proposed by the standard code ISO 2631-2. Effects of changing thickness of the Profiled Steel Sheet (PSS), Dry Board (DB), screw spacing, grade of concrete, damping ratio, type of support, and floor span on the dynamic responses of the PSSDBC panels were assessed. Results demonstrated that although some factors reduced dynamic response of the PSSDBC system under human walking load, low frequency PSSDBC floor system could reach high vibration levels resulting in lack of comfortableness for users.

Keywords

structural composite floor system, profiled steel sheet dry board, vibration serviceability, human walking load, dynamic response, human comfort.

1 INTRODUCTION

Serviceability in modern structures constructed by high strength and lightweight materials is the most important issue and should be considered in addition to the strength/safety criteria [6, 12]. In case of evaluation of vibration serviceability, generally codes and standards present two approaches. First is static deflection caused by nominal live load which is commonly limited to SPAN/360 (A58, 1982) or between SPAN/180 and SPAN/480 in different specifications (ACI 318-77, 1977 and AISC, 1978), and second is the minimum of DEPTH/SPAN for flexural members depending on the end restrains (ACI 318-77, 1977) [14]. Ellingwood and Tallin [14] stated that control of the static deflection is not sufficient to evaluate the vibration serviceability of floors.

On the other hand, Al-Foqaha et al. [2] reported a number of researchers (Onysko 1970, 1985, 1988; Polensek 1970, 1971, 1975, 1988; Polensek et al. 1976; Allen 1974, 1990; Allen and Rainer 1976; Allen and Murray 1993; Murray 1979; Chui 1986; Smith and Chui 1988; Ebrahimpour and Sack 1989; Ohlsson 1988, 1991; Kalkert et al. 1993; Dolan et al. 1995, Lenzen 1966; Wiss and Parmelee 1974, Filiatrault et al. 1990; Foschi et al. 1995; Kalkert et al. 1995) have declared that evaluation of the floor vibration serviceability may not be performed by control of the static deflection such as SPAN/360 [2]. Wood floor systems were studied based on the finite element method under dynamic loads induced by human activities. A series of design curves related to Root-Mean-Square (RMS) acceleration, mass, and FNF were proposed and compared with the experimental study which concluded in a close agreement. It was demonstrated that the vibration criteria based on static properties or FNF are not enough to prevent unwanted vibration of floors [2].

The International Standards Organization (ISO 2631-2) [22] recommended an acceleration limit as a baseline in terms of RMS for various applications of floors, as illustrated in Fig. 1. This Standard proposed a criterion on the basis of the peak acceleration by multiplication of baseline with 10 for offices, 30 for shopping malls and indoor footbridges, and 100 for outdoor footbridges.

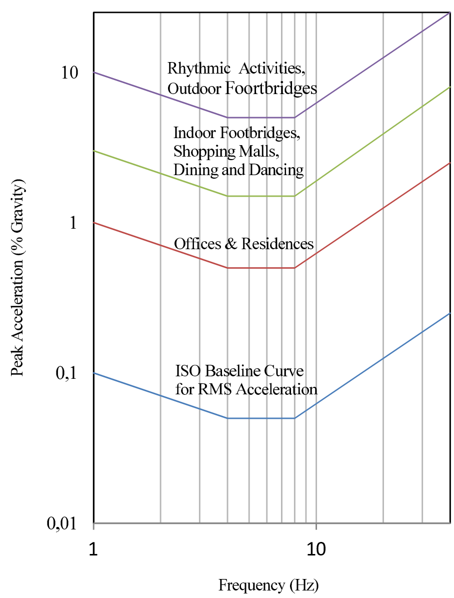


Figure 1 Recommended peak acceleration for human comfort for vibrations due to human activities [22].

The American Institute of Steel and Construction (AISC) [26] has proposed criteria for human comfort which are the same as the ISO Standard [22], as shown in Fig. 1.

Various authors have evaluated the vibration serviceability of floors under human activities through determination of their dynamic responses, analysis, and experiment from few years ago [24]. Sandun de Silva and Thambiratnam [10], da Silve et al. [8], da Silve et al. [9], El-Dardiry and Ji [13], Williams and Waldron [37], Chen [6] determined dynamic responses of composite floors under human activities to assess their vibration serviceability. Ellingwood and Tallin [14] mathematically studied the dynamic response of floors under a pragmatic model instead of the pedestrian dynamic load. Experimental serviceability criteria were also reported to minimise the vibration of the floors. Smith and Chui [31] presented a usable method for designers based on a flow chart to evaluate the dynamic response of lightweight wood-joist floor by determination of natural frequency and RMS acceleration of the system under the heel-drop impact load. Howard and Hansen [21] studied the vibration analysis of waffle floors based on a mathematical method for several manufacturing buildings which was also verified by finite element and experimental results. Foschi et al. [16] carried out an experimental and analytical study on the vibration response of wood floors as a lightweight panel useful in residential and commercial buildings under impact load induced by users. Occupants were modelled by two simple oscillators, one degree of freedom and two degrees of freedom. Osborne and Ellis [28] presented a study on a long-span lightweight LFF (FNF lower than 10 Hz [25]) system by the analysis of various methods to evaluate the vibration acceptability of the system through obtaining FNF, damping ratio, and acceleration. Willford et al. [36] reviewed five methods to predict the response of structures under the footfall load. The study was performed in two parts; floor and bridge with the FNF lower than 10 Hz, and also floor with the FNF above 10 Hz (HFF). Mello et al. [24] also studied dynamic analysis of a composite system made of concrete slab and steel beams. The research on acceptability of studied models was conducted under four types of dynamic loads which were represented by human walking load, measurement of peak acceleration of panels, and comparison with limit of codes. The dynamic response of the mentioned floors was investigated by using FEM as a modern computational tool for structural analysis.

The PSSDB is a lightweight composite structural system consisting of the PSS and DB attached together by self-drilling and self-sapping screws, as shown in Fig. 2.

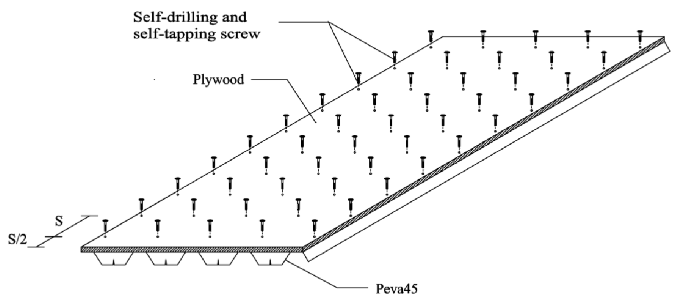


Figure 2 Profiled steel sheet dry board system.

Study on using concrete as an infill material in the PSSDB system uncovered the performance of concrete on increasing the stiffness of the system [34]. The performance of using concrete in trough of the PSS of the PSSDB system was revealed experimentally to reduce its response where the panel is applied under human walking load [17].

It was reported that since the PSSDB system is slender and flexible in its nature, the natural frequency of the floor system may be low and becomes perceivable to users [39]. In addition, Gandomkar et al. [18] showed experimentally and numerically that in practical dimensions of floors, the FNF of the PSSDBC system is lower than the PSSDB system and is in the category of LFF. Therefore, the PSSDBC floor panels with usual spans are exposed to vibrations of human activities, because the FNF of a LFF is close to frequency range of human activities. Accordingly, a consistent dynamic analysis of the PSSDBC system with the practical span is advisable to evaluate the vibration serviceability of the system under human walking activities.

This paper deals with the dynamic response of the low frequency PSSDBC floor system used as offices and residences under human walking load. Thirteen PSSDBC panels were considered to reveal effects of different parameters such as boundary conditions, damping ratio, thicknesses of the PSS and DB, screw spacing, grade of concrete, and floor span on the dynamic response of the system. Firstly, natural frequencies and vibration modes of all panels were obtained. Secondly, dynamic responses of the studied panels were determined in terms of peak acceleration and also compared to limiting values proposed by the ISO 2631-2 [22] to show their vibration acceptability.

2 HUMAN-INDUCED DYNAMIC LOADS

Vibration of floors under human rhythmic activities is a very complex problem with respect to mathematical or physical characterisation of this phenomenon because the properties of dynamic vibration of these activities are interconnected to the individual body adversities and the ways which human performs a certain rhythmic activity [24]. A number of studies tried to evaluate the dynamic loads representing human activities. According to Mello et al. [24], the first pioneer in determination of the forces induced by human motion was Otto Fischer, a German mathematician, who presented his study in 1895. Also, Ohmart presented walking motion forces graphically in 1968. Folz and Foschi [15] idealised the occupants on the floor as lumped parameter models which are components of discrete masses, springs, and viscous dashpots with two and eleven degrees of freedom. Racic et al. [29] reviewed 271 references which deal with various experimental and analytical characterisations of human walking forces and their application in vibration serviceability design of civil engineering structures when subjected to pedestrian movement such as footbridges, floors, and staircases. Mello et al. [24] reported that experimental studies were performed by Alves (1997) and Faisca (2003) on two kinds of concrete platforms; rigid and flexible, when a group of volunteers acted on them. The aim of their studies was a description of forces induced by human activities such as soccer and rock, aerobics, and jumps.

In the current study, dynamic responses of the studied panels were determined under following four dynamic human walking loads [24] to evaluate their vibration acceptability.

2.1 First load model

First load model which represents people walking is shown in Eq. (1).

$$F(t) = P\alpha_i \cos(2\pi i f_s\ t) \qquad (1)$$

Where:

P: individual's weight, taken as 700-800 N;
α_i: dynamic coefficient for the ith harmonic force component;
i: harmonic multiple of the step frequency;
f_s: step frequency;
t: time in seconds.

In the first load model, only one resonant harmonic of the load was considered. The harmonic multiple of the step frequency was adopted from Tab. 1 which depends on the FNF of the panel. For example, if calculated FNF of a panel is equal to 7.326 Hz (Panel Number (PN) of 9 = PN9), according to Tab. 1, only fourth harmonic of the walking loads with step frequency of f_s =1.8315 Hz (4 × 1.8315 Hz = 7.326 Hz) should be used in Eq. (1) to determine the first applied load on the panel. Fig. 3 illustrates the first dynamic load model for the panel with FNF equal to 7.326 Hz.

Table 1 Loading frequencies, dynamic coefficients, and harmonic phase angles.

Harmonic i	Person walking			
	if_s (Hz)	α_i	Φ	
			Second and third load model	Fourth load model
1	1.6-2.2	0.5	0	0
2	3.2-4.4	0.2	$\pi/2$	$\pi/2$
3	4.8-6.6	0.1	$\pi/2$	π
4	6.4-8.8	0.05	$\pi/2$	$3\pi/2$

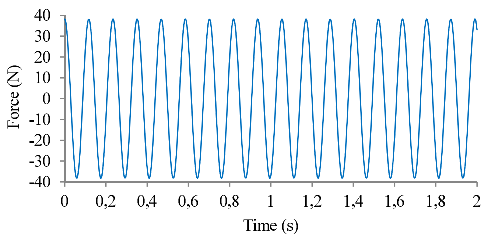


Figure 3 First load model for PN9.

2.2 Second load model

The second load model that represents human walking load is presented in Eq. (2).

$$F\,(t) = P\left[1 + \sum \alpha_i \cos(2\pi i f_s\ t + \Phi_{\mathrm{i}})\right] \tag{2}$$

Where:

P: person' weight;
α_i: dynamic coefficient for the harmonic force;
i: harmonic multiple ($i = 1,\ 2,\ 3, \ldots,\ n$);
f_s: activity step frequency (dancing, jumping, aerobics or walking);
t: time;
Φ_i: harmonic phase angle.

Unlike the previous load model, this load was composed of a static parcel and a combination of four time-dependent repeated loads presented by Fourier series. Four harmonics (see Tab. 1) were adopted to produce the second dynamic load model. Considering a panel the same as the discussed panel in the previous load model with the FNF equal to 7.326 Hz, the fourth harmonic with a step frequency of 1.8315 Hz (4 × 1.8315 Hz = 7.326 Hz) was the walking load resonant harmonic. Tab. 1 shows the dynamic coefficients and phase angles for each harmonic which were used to produce second dynamic load model, as depicted in Fig. 4.

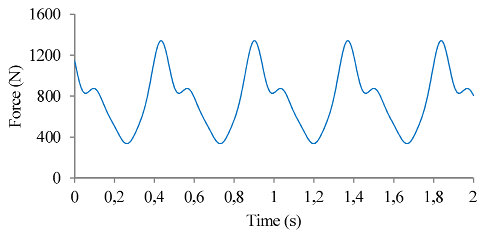


Figure 4 Second and third load models for PN9.

2.3 Third load model

The mathematical function of the third load model which represents the human walking load is similar to the second one, presented in Eq. (2). Similar to the previous load model, the fourth harmonic with a step frequency of 1.8315 Hz was the resonant harmonic of human walking load (see Tab. 1). The third load model is more pragmatic than the last two kinds of the load models, as the position of this load is changed across the singular location of the floor system (see Fig. 5).

Study of some other parameters related to the step frequency such as step distance and speed of walking, presented in Tab. 2, is necessary in this kind of load. Also, finite element mesh should be very refined in the third dynamic load model. The contact time of application

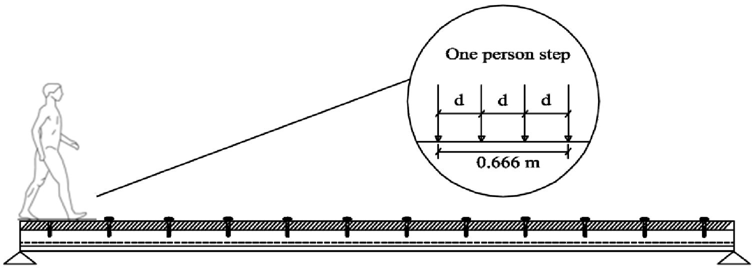


Figure 5 Person walking on the PSSDBC floor panel.

of the dynamic load with floor was calculated from the step distance and step frequency (see Tab. 2).

In this load model, the subsequent scheme was followed: In a panel identical to the panel in the previous load models, and according to Tab. 1, the step frequency was equal to 1.8315 Hz when the fourth harmonic was as the resonant harmonic. Therefore, in accordance with Tab. 2, the step distance was equal to 0.666 m (see Fig. 5).

Table 2 Person walking characteristics [24].

Activity	Velocity (m/s)	Step distance (m)	Step frequency (Hz)
Slow walking	1.1	0.6	1.7
Normal walking	1.5	0.75	2.0
Fast walking	2.2	1.0	2.3

The step period which corresponds with the step distance of 0.666 m is equal to $1/f = 1/1.8315 = 0.546$ s (see Tab. 2). As it is shown in Fig. 5, four forces were considered representing one human step, which each of the forces as P1, P2, P3, and P4 was applied on the floor during 0.546(contact time)/3 = 0.18 s. The dynamic forces of P1, P2, P3, and P4 were not applied together at the same time. First, the load of P1 was applied on the floor according to Eq. (2) for 0.18s. At the end of this time period, the load of P1 became zero and the load of P2 was applied for 0.18 s. The other loads of the first person step, P3 and P4, were applied in the same procedure described previously. After 0.546 s, the first person step finished and the second person step started and the load of P1 of the second step was equal to the load of P4 in the first step. According to the mentioned method, the process continued repeatedly until all the dynamic forces applied along the considered path (see Fig. 12) of the floor.

2.4 Fourth load model

The fourth dynamic load model representing human walking load is investigated with the same procedure considered in the third one. The principal difference between the third and fourth loads was consideration of the human heel effect in the fourth load which was ignored in the third load model. The human heel effect was uncovered to be an effective parameter on the increase of the load by comparing the third and fourth load models. According to Mello et al. [24], Varelo (2004) proposed the mathematical functions of the fourth load model as Eqs. (3)-(6).

$$F(t) = \begin{cases} \left(\frac{f_{mi}F_m - P}{0.04T_p}\right) t + P & \text{if } 0 \le t < 0.04T_p \\ f_{mi}F_m\left[\frac{C_1(t-0.04T_p)}{0.02T_p} + 1\right] & \text{if } 0.04T_p \le t < 0.06T_p \\ F_m & \text{if } 0.06T_p \le t < 0.15T_p \\ P\left[1 + \sum_{i=1}^{nh} \alpha_i \sin(2\pi i f_s(t + 0.1T_p) + \Phi_i)\right] & \text{if } 0.15T_p \le t < 0.90T_p \\ 10\,(P - C_2)\,.\left(\frac{t}{T_P} - 1\right) + P & \text{if } 0.90T_p \le t < T_p \end{cases} \tag{3}$$

F_m: maximum Fourier series value, given by Eq. (4);
f_{mi}: heel-impact factor;
T_p: step period;
C_1: coefficients given by Eq. (5);
C_2: coefficients given by Eq. (6).

$$F_m = P\left(1 + \sum_{i=1}^{nh} \alpha_i\right) \tag{4}$$

$$C_1 = \left(\frac{1}{f_{mi}} - - 1\right) \tag{5}$$

$$C_2 = \begin{cases} P\,(1 - \alpha_2)\,, & \text{if } nh = 3 \\ P\,(1 - \alpha_2 + \alpha_4)\,, & \text{if } nh = 4 \end{cases} \tag{6}$$

Mello et al. [24] reported that Varela (2004) and Ohlsson (1982) declared the impact factor varies person-to-person. In this study, impact factor was adopted equal to 1.12 (f_{mi} = 1.12 [24]). Fig. 6 shows the dynamic load model of a panel which presented in the previous load models with the FNF of 7.326 Hz based on Eqs. (3)-(6).

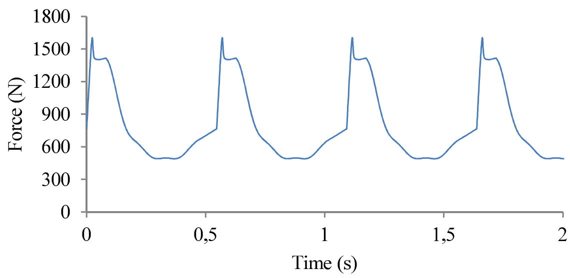


Figure 6 Fourth load model for PN9.

3 DAMPING

Damping of structures used in the dynamic analysis is the most difficult item to find. Damping is not an exclusive physical phenomenon in structures dissimilar to mass and stiffness characteristics of a structural system. Therefore, modelling of damping in structures is not accurate like mass and stiffness and its determination is entirely possible based on full-scale measurements [19, 23]. The damping ratios are usually measured from experience or adopted through suggested values by design guides and cannot be determined through analytical methods [25]. Sandun de Silva and Thambiratnam [10] had a wide discussion on damping ratios suggested by some authors such as Osborne and Ellis (1990), Wyatt (1989-SCI-076), Hewitt et al. (2004), Murray (2000), Elnimeiri and Lyengar (1989), Brownjohn (2001), and Sachse (2002) which were dependent on application of floors and kind of partitions built on them. Damping ratios of 1.6%, 3%, 6%, and 12% were used for a steel-deck composite floor system in their studies.

In order to obtain damping ratios for floors, applicable design guides present simple guidance which has been summarised in Tab. 3 [25]. It can be seen that the SCI-P076, SCI-P331, and the AISC [26] have almost the same values for damping, but according to Middleton and Brownjohn [25], the Canadian Standards Association (CSA) clearly presents overestimated damping.

Table 3 Proposed damping ratios by standards [25].

Name of Standard	Bare floors or very few non-structural components (%)	Non-structural elements, i.e. furniture, fixtures and fittings and cantilever partitions (%)	Heavily partitioned floor with full height partitions (%)
SCI P076	1.5	3	4.5
SCI P331	1.1	3	4.5
AISC*	2	3	5
CSA	3	6	12

* for offices, residences and churches

A new edition of SCI-P354 [30] presents critical damping ratios for various floor types which are very similar to those of SCI-P075 and SCI-P331 but with small differences in content, as summarised in Tab. 4.

Usually damping of the completely bare floors is not used because the bare floors are mostly utilised during the construction time and would not be used after the floors are occupied. Therefore, damping of bare floors is useful to reveal their dynamic response and to evaluate their vibration acceptability before the building is equipped [30].

Gandomkar et al. [18] estimated damping ratios of bare PSSDBC panels with such characteristics as: width and length of 795 mm and 2400 mm respectively, and thicknesses of 0.8 mm and 18 mm respectively for PSS and DB, 200 mm screw spacing, and concrete grade of 30 (C30) where the panels were located on pin-roller end supports. Damping ratios of 2.90%,

Table 4 Proposed damping ratios by SCI-P354 [30].

ζ (%)	Floor finishes
0.5	For fully welded steel structures, e.g. staircases
1.1	For completely bare floors or floors where only a small amount of furnishings are present
3.0	For fully fitted out and furnished floors in normal use
4.5	For a floor where the designer is confident that partitions will be appropriately located to interrupt the relevant mode (s) of vibration (i.e. the partition lines are perpendicular to the main vibrating elements of the critical mode shape)

1.52%, 0.83%, and 2.54% were found for the first four vibration modes, respectively.

Osborne and Ellis (1990) stated that although damping in a floor system can be measured by simple heel impact tests, various limitations make mostly unknown an exact value for damping of a steel-deck composite floor. Similarly, according to Dolan et al. [11], Smith and Chui (1988) reported a wood floor as a component of sheathing and joist which were connected to each other by glue, had more different damping characteristics than the same floor where nails are used instead of glue. Therefore, used construction techniques and workmanship in a floor are effective on its damping ratio.

In this study and in accordance with the above mentioned literature, the damping ratios were adopted as 1.1%, 3%, and 4.5% for the studied panels in order to consider different situations of the floor during the lifetime of its service (see Tab. 4).

4 STRUCTURAL MODEL

Peva45 is available in the local market by the width of 795 mm and maximum length of 15 m. Also, maximum length and width of plywood are 2400 mm and 1200 mm, respectively. Therefore, to prepare the PSSDB panels with practical dimensions with sizes greater than the size of Peva45 or plywood, some pieces of Peva45 and plywood should be used together. In the current study, the panels were consisted of four (4) pieces of Peva45 and eight (8) pieces of plywood. Also, C30 was used in trough of Peva45 as an infill material. Fig. 7 shows the section of the studied panels. The connection between the two adjacent Peva45 side by side panels (detail A) was represented by a typical lap joint idea as shown in Fig. 8. Wright and Evans [38] presented the connectivity characteristics of such joint. As can be seen in Fig. 8, nodes I(2) and J(2) are connected to nodes I(3) and J(3) respectively, assuming complete freedom in the longitudinal and rotational directions whilst assumed to have complete connection in the vertical and lateral directions [38].

In this study, the PSSDBC control panel (PN1(see Tab. 5)) adopted 0.8 mm thick Peva45

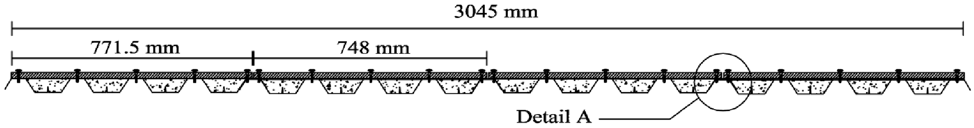
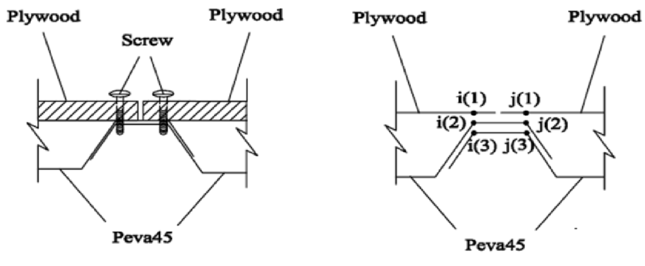


Figure 7 The section of studied panels.



(a) (b)

Figure 8 Detail A: (a) Constructional model (b) Analytical model

as PSS, 18 mm thick plywood as DB, DS-FH 432 self-drilling and self-tapping screws at 200 mm screw spacing as the connectors, and C30 as an infill material in trough of Peva45. Thirteen (13) PSSDBC panels have been developed with various supports as shown in Fig. 9. The characteristics of these panels are summarised in Tab. 5.

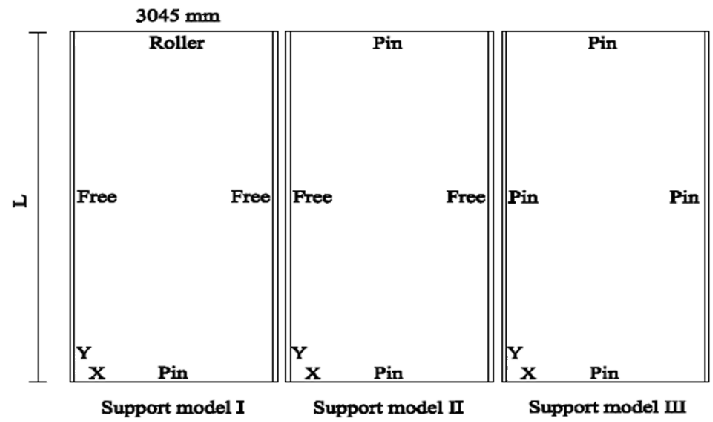


Figure 9 Considered supports in the study.

The dynamic Young's modulus of materials was used herein. According to the AISC [26], the dynamic Young's modulus for steel can be chosen similar to its static value (BS 5950-Part4 [4]), i.e. $2.10{\times}10^5$ MPa for Peva45. Stalnaker and Harris [32] stated that the property of plywood is mostly isotropic because of its manufacturing process. Also, Ahmed [1] declared that although dry boards may be found as isotropic or orthotropic in the nature, they can

Table 5 Characteristics of the studied panels.

Panel No. (PN)	L (mm)	Support type (see Fig. 9)	Thickness of PSS (mm)	Thickness of DB (mm)	Screw spacing (mm)	Grade of concrete	Damping ratio (%)
1	4200	I	0.8	18	200	C30	1.1
2	4200	I	0.8	18	200	C30	3.0
3	4200	I	0.8	18	200	C30	4.5
4	4200	I	0.8	25	200	C30	1.1
5	4200	I	1.0	18	200	C30	1.1
6	4200	I	0.8	18	100	C30	1.1
7	4200	I	0.8	18	200	C35	1.1
8	4200	II	0.8	18	200	C30	1.1
9	4200	III	0.8	18	200	C30	1.1
10	4200	III	0.8	18	200	C30	3.0
11	4200	III	0.8	18	200	C30	4.5
12	3600	I	0.8	18	200	C30	1.1
13	4800	I	0.8	18	200	C30	1.1

be modelled as isotropic plates without any difficulties. Considering isotropic sheeting, the static Young's modulus of plywood which is available in the local market adopted as 7164 MPa [40] in this study. However, the dynamic value was chosen 10% greater than the static value according to Bos and Bos Casagrande [3].

In accordance with BS 8110 [5], the static Young's modulus of concrete was determined as 24597 MPa and 26567 MPa for concrete grades of 30 and 35, respectively. da Silva et al. [8] discussed that according to the AISC [26] in situations where the composite slab is subjected to dynamic excitations concrete becomes stiffer than the case when is subjected to pure static loads. This issue [26] suggests a 35% increase in the dynamic Young's modulus of the conventional concrete in comparison with the static Young's modulus. Therefore, dynamic modulus of elasticity as 33206 MPa and 35865 MPa were adopted for concrete grades of 30 and 35, respectively.

The stiffness of screws which are connections between Peva45 and plywood and also between Peva45 and concrete was obtained by push-out tests. The stiffness of shear connectors is needed in the finite element analysis. Nordin et al. [27] performed a study to identify the stiffness of screws between Peva45-Cemboard, Cemboard-Timber, and Peva45-Plywood. It was found that the shear connection stiffness between Peva45 and plywood was 610 N/mm [27]. This value was represented instead of the connections between I(1) and J(1) respectively to I(2) and J(2) (Fig. 8). The connection between Peva45 and concrete as an infill in trough of Peva45 is also a partial interaction problem. Gandomkar et al. [18] focused on finding the connection stiffness between Peva45 and concrete with different grades of C25, C30, and C35 by push-out tests where covering was chosen as plywood. The shear stiffness of this connection was obtained as 386 kN/mm and 417.60 kN/mm respectively for C30 and C35 where spacing between the springs was considered as 200 mm. The density of Peva45 and plywood was adopted as 7850 kg/m^3 and 600 kg/m^3, respectively. Also, the density of concrete was determined as 2273 kg/m^3 in the laboratory [18].

5 COMPUTATIONAL MODEL

The FEM presents a more accurate dynamic response especially for structures with involved geometry. Using this method is increased because it can reduce the cost of computing functions [25]. The logical estimation on vibration of composite floors under walking load is a complicated work because of complexity in geometry of structures, variety of material properties in different structural components, and the nature of walking load as a continuous and transient load. It has been known that using FEM can cover and solve the mentioned tasks foremost [6]. Therefore, the FEM was used in this study to evaluate the dynamic response of the PSSDBC panels under the dynamic human walking load.

Developed finite element models were simulated by the use of refined mesh in the ANSYS program [33]. In the studied system, the PSS and DB were made of SHELL281 element as a suitable element for analysing thin to moderately-thick shell structures. In addition, the self-drilling and self-tapping screws were represented by COMBIN14 element as connection between Peva45 and plywood and also between concrete and Peva45. Moreover, SOLID65 element was adopted for modelling of concrete in the computational models.

For more clarification, Fig. 10 shows the procedure of modelling Peva45, concrete, and plywood in the simulation for one bay of the studied system. The connection between elements of Peva45 and concrete [20], plywood and concrete, and also Peva45 and plywood in the simulation is performed by using spring element (COMBIN14) in three directions (X, Y, and Z). In this case and according to Fig. 10, nodes of D2 and D10 were respectively connected to nodes of P2 and P10 which stiffness of springs was adopted as 610 N/mm [27] in X and Y directions (see section 4) and 10^5 N/mm in Z (vertical) direction (see Fig. 10). Nodes of CT3, CB4, CB5, CB7, CB8, CT9, and D6 were connected to nodes of P3, P4, P5, P7, P8, P9, and CT6 respectively in the same procedure as mentioned above with 1 N/mm [20] stiffness of springs for all directions (X, Y, and Z). Node of CM6 was connected to node of P6 by 386 KN/mm stiffness of springs in X and Y directions (see section 5) and 10^6 N/mm in Z direction, when C30 is used in the PSSDBC system.

6 DYNAMIC ANALYSIS OF THE STUDIED PANELS

To determine the dynamic responses of the PSSDBC composite panels, a linear time-domain analysis was performed [7]. The dynamic responses of the studied panels were obtained from a vast parametric analysis performed using finite element ANSYS program [33]. The results were natural frequencies, displacements, velocities, and accelerations.

The main goal of this study was to evaluate vibration serviceability of the PSSDBC composite panels. For this purpose, the maximum acceleration of the panels was determined

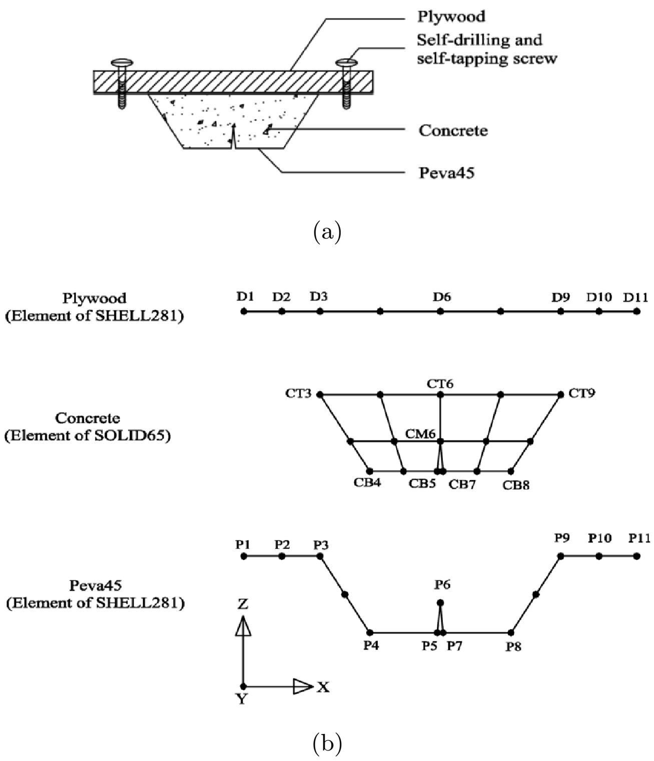


Figure 10 (a) One bay PSSDBC structural system (b) Situation of nodes in elements of one bay PSSDBC system.

under the four dynamic load models described previously. Then, the obtained accelerations were compared to the proposed peak acceleration limit by the ISO [22].

6.1 Natural frequencies and mode vibrations of studied panels

Gandomkar et al. [18] conducted a wide experimental and numerical investigation on the natural frequencies of the PSSDBC system. The numerical study performed by FEM presented accurate results compared to the corresponding experimental results. In the current study, developed finite element models which were verified in the literature [18] were used to obtain the natural frequencies of the studied panels, as tabulated in Tab. 6.

The results of Tab. 6 illustrate that increasing the thickness of plywood and Peva45 decreased and increased the FNF of the system, respectively. Therefore, it can be seen that the obtained results reveal the effect of the mass and stiffness of Peva45 and plywood on the FNF of the system.

It was also shown that the decrease of the screw spacing enhances the FNF of the panel, because the panel will be stiffer [35].

The increase of the concrete grade can improve the FNF of the PSSDBC panel, but not significantly. Control of sliding parallel with the strong direction of the PSS and using four-

Table 6 First six natural frequencies of the studied panels.

Panel No. (PN)	f_{01} (Hz)	f_{02} (Hz)	f_{03} (Hz)	f_{04} (Hz)	f_{05} (Hz)	f_{06} (Hz)
1, 2, 3	4.742	5.514	7.670	10.696	13.852	15.926
4	4.675	5.587	8.048	11.477	15.035	15.661
5	4.891	5.801	8.259	11.625	14.858	16.895
6	4.82	5.598	7.863	11.001	14.214	18.819
7	4.849	5.616	7.768	10.797	13.982	16.013
8	6.693	7.341	9.221	12.171	15.504	18.871
9, 10, 11	7.326	9.208	12.385	18.236	19.81	22.299
12	6.447	7.098	9.076	12.063	15.426	19.596
13	3.648	4.515	6.767	9.748	12.688	13.226

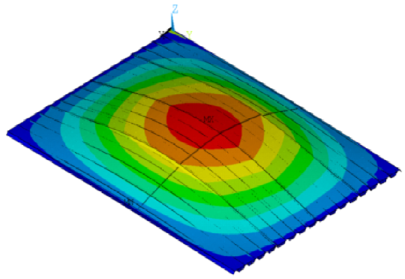
sided support instead of two-sided support (perpendicular to the strong direction of the PSS) can considerably increase the FNF of the system. It is clear that change in the panel span can also change the FNF of the panel. The FNF of the panels had two properties. First, all were smaller than 10 Hz; therefore, the category of the studied panels was LFF. Second, all were greater than 3 Hz, as a minimum limitation of FNF for floors proposed by the SCI – P354 [30].

First six vibration modes of the PN9 which were obtained by the finite element model are illustrated in Fig. 11.

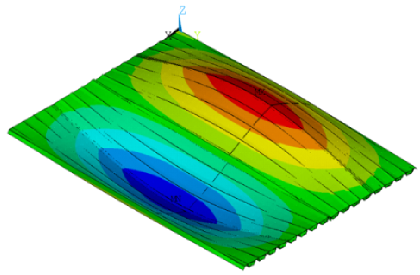
6.2 Peak acceleration of studied panels

The peak accelerations of the panels were determined by the dynamic analysis of the developed finite element models. Person walking across the panel was considered in the third and fourth dynamic load models. Therefore, the paths of walking should be defined. The peak accelerations of all models were determined for path 1 (see Fig. 12) under three different support models (see Fig. 9). Also, the peak accelerations of PN2, PN7, and PN10 were obtained where person used path 2 (see Fig. 12) for walking. Tab. 7 indicates the peak accelerations of the PSSDBC panels under the four previously described loads. This table also presents the limit of peak acceleration recommended by the ISO 2631-2 [22] for residences and offices (see Fig. 1).

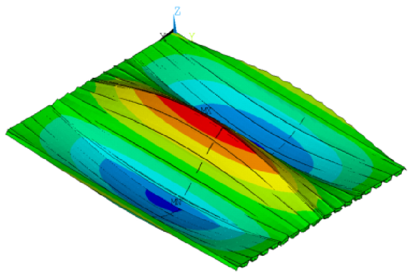
According to Tab. 7, the peak accelerations of the studied panels were evaluated under the second load model which uncovered to be greater than those corresponding evaluated peak accelerations under the first load model. This point revealed that considering four harmonics in the dynamic load is a very important issue in the dynamic responses of the floor and showed a significant effect on the increase of the peak acceleration. As it is obvious from Tab. 7, when the third and fourth load models were applied on the studied panels, the peak accelerations were higher than those of the applied first and second load models. This fact was highlighted when the position of the dynamic load changed across the individual direction, the dynamic



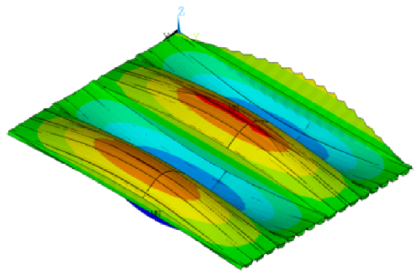
(a) Mode shape associated to the first natural frequency: f_{01}=7.326 Hz



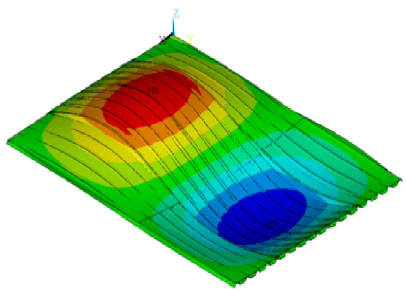
(b) Mode shape associated to the second natural frequency: f_{02}=9.208 Hz



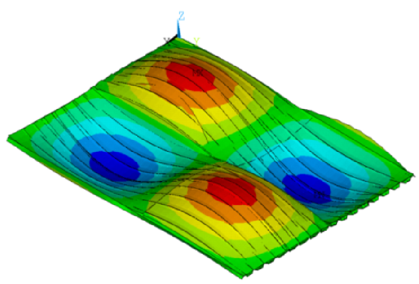
(c) Mode shape associated to the third natural frequency: f_{03}=12.385 Hz



(d) Mode shape associated to the fourth natural frequency: f_{04}=18.236 Hz



(e) Mode shape associated to the fifth natural frequency: f_{05}=19.810 Hz



(f) Mode shape associated to the sixth natural frequency: f_{06}=22.299 Hz

Figure 11 Floor vibration modes of model number 8.

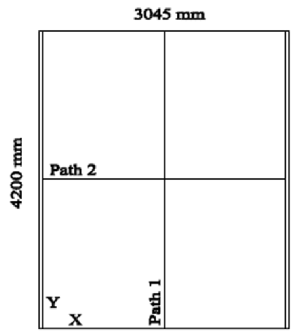


Figure 12 Layout of paths.

response of the panels increased. Mello et al. [24] also focused on this point and stated that this is a substantial increase in the structure. The peak acceleration of the panels under the fourth load model was assessed higher than those under the third load model. On the other hand, the scheme of loading on the panels in the third and fourth load models was the same as each other. Therefore, this increase should be caused by the heel impact factor (f_{mi} = 1.12) used in the fourth load model.

In accordance with the design criteria proposed by the ISO 2631-2 [22] and also based on the peak accelerations of the panels produced by the first load model, all the studied panels did not have any problems regarding the human comfort. By comparing the results of PN1, PN8, and PN9, an interaction was revealed between the dynamic load model with different parameters such as support conditions and dynamic characteristics of the panels which made an unknown phenomenon in the dynamic response of the panels. It should be noted that the harmonic of the resonant in PN1 was the third harmonic; therefore, the dynamic coefficient of the load is 0.1. However, the fourth harmonic is the resonant harmonic in PN8 and PN9, accordingly, the dynamic coefficient is 0.05. By investigation of the results of the second load model, all studied panels did not show any problems related to the human comfort. On the other hand, by comparing the peak accelerations of the dynamic analysis of the panels under the third and fourth load models and recommendations of the ISO 2631-2 [22], it was uncovered that all panels were not comfortable for human under these loads.

The results on the path 1 also demonstrated that change of the characteristics of the PSSDBC system changes its dynamic response. By the increase of the thickness of plywood from 18 mm to 25 mm and Peva45 from 0.8 mm to 1.0 mm, peak accelerations of the four load models were decreased by an average value of 11.52% and 17.06%, respectively. Furthermore, the reduction of the screw spacing from 200 mm to 100 mm decreased the peak acceleration of the four load models by an average value of 7.35%. The grade of concrete did not have a considerable effect on the dynamic response of the PSSDBC panel, as by changing C30 to C35 the peak acceleration of the panel only reduced by 2.98%. The results also indicated that change of the damping ratios from 1.1% to 3% and 4.5% can respectively decrease the peak accelerations of the PSSDBC panels by 14.94% and 26.38% for the panel with the support type

Table 7 Peak accelerations of the studied panels at resonance.

Panel No. (PN)	Load model I (m/s^2)	Load model II (m/s^2)	Load model III (m/s^2)		Load model IV (m/s^2)		ISO 2631-2 [22]
			Path 1	Path 2	Path 1	Path 2	
1	0.03367	0.03917	0.10397	-	0.12594	-	0.04903 (m/s^2)
2	0.02508	0.03161	0.09671	0.08946	0.11591	0.10261	
3	0.01796	0.02679	0.08993	-	0.10862	-	
4	0.02691	0.03552	0.09948	-	0.11043	-	
5	0.02373	0.03284	0.09463	-	0.10884	-	
6	0.02828	0.03614	0.09957	-	0.12414	-	
7	0.03187	0.03833	0.10245	-	0.12219	-	
8	0.02939	0.04287	0.11907	0.10745	0.13006	0.11214	
9	0.03439	0.04647	0.13851	-	0.15313	-	
10	0.02936	0.03785	0.11324	0.13061	0.14838	0.16353	
11	0.02566	0.03021	0.10572	-	0.13264	-	
12	0.02439	0.03135	0.09184	-	0.10023	-	
13	0.03948	0.04886	0.14952	-	0.17823	-	

of I (see Tab. 5 and Fig. 9) and respectively 13.63% and 24.36% for that with the support type of III (see Tab. 5 and Fig. 9). By comparing PN1 with PN12 and PN13 it was shown that the length of the panels had a direct effect on the peak acceleration of the panels, where the response enhanced and reduced respectively for the increase and decrease of the length of panels for all four load models. However, these results are not addressable, as according to results of Ref. 24, the peak accelerations are not attributed to the length of the panels.

By comparing results of PN1 and PN8, it is obvious that control of sliding in support decreased the peak acceleration of the panel for the first load model. It may be due to the significant increase of the FNF of the panel, therefore, the dynamic coefficient changed from 0.1 to 0.05. On the other hand, as stated in the first load model only one harmonic considered depending on the FNF, the dynamic load applied on PN1 was higher than that on PN8. According to the results illustrated in Tab. 7, the mentioned issue was not shown to be effective when four harmonics considered in the dynamic load models (load models of II, III, and IV), even for PN9. The reason was about the complexity in the dynamic responses of the panels under interaction between supports and other substantial characteristics of structures.

Comparing peak accelerations of path 1 and path 2 in the panels shows different phenomena. The response of path 1 was greater than that of path 2 when only two sides of panels were supported. On the other hand, the response of path 2 was greater than that of path 1 when all four sides of panels were supported. The panels were not also comfortable for users, where walking performed across path 2.

7 FINAL REMARKS

This paper investigated the dynamic response of the PSSDBC low frequency floor panels under human walking load to evaluate the vibration serviceability of the system. Four dynamic load models were used while the third and fourth load models were more pragmatic having two properties; changing load according to the individual position, and generating time function corresponding to the nature of human walking load. The effect of human heel impact was also considered in the fourth load model.

The dynamic responses of the studied PSSDBC panels were obtained in terms of the peak acceleration and compared to the proposed limiting value by the ISO 2631-2 [22] where the panels used as residences and offices. The studied panels were showed to be comfortable when the first and second dynamic load models applied on them. The position of loads was changed across the individual directions when the third and fourth dynamic load models applied on the panels. For these two types of loads, two paths were selected to show the effect of direction of walking on the response of the panels. The peak accelerations of the studied panels under the third and fourth dynamic load models were determined higher than those of the first and second loads and also limiting value of the ISO 2631-2 [22]. Therefore, all panels were uncomfortable for users when the third and fourth load models applied on them. These results uncovered the fact that changing the position of the load is an effective item in the increase of the response of the panels.

Increasing the thickness of the PSS and DB and decreasing screw spacing significantly reduced the peak acceleration of the system. However, change of the concrete grade did not show a pronounced effect on changing the response of the PSSDBC system. Enhancement of the damping ratio of the PSSDBC system can considerably reduce the peak acceleration of the system. These results can be useful to help designers reduce the response of the floor by furniture and types of partitions (see Tab. 4).

The increase and decrease of the length of the studied panels enhanced and reduced the peak acceleration of the panels, respectively.

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