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EXPERIMENTAL AND NUMERICAL ASSESSMENTS OF A LONG-SPAN MASS TIMBER FLOOR SYSTEMS SUBJECTED TO FOOT-FALL INDUCED VIBRATION

Christian Slotboom[1], Carla Dickof[2], Tobias Fast[3]

ABSTRACT: The following paper presents a series of experimental and numerical vibration assessments completed on the long-span floor system of Fast+Epp's home office. Measurements were taken on the building throughout different stages of construction to understand the impact of non-structural elements on floor vibration. Factors considered are partitions, concrete toppings, and screws between the glulam beam and CLT topping. Finite element models are also created for the structure, and predictions from these numerical models are compared to experimental data. The model used simplified models, similar to those practicing designer would make, and several parameters in these are varied to understand their impact on vibration. It was found that non-structural elements had a significant effect on the vibration performance of the system, changing both the fundamental frequency and reducing the overall accelerations.

KEYWORDS: Vibration, Mass Timber, Long-span floor, Footfall loads.

1 INTRODUCTION

Long-span floor systems are preferred for many types of buildings such as office and commercial buildings because they allow for flexible open floor plans suitable to any tenant. However, long-span floors are generally more susceptible to serviceability constraints such as footfall vibration. Human-induced vibration on a floor is the result of periodic excitation (i.e. steps at a regular frequency); this periodic force will cause the floor to vibrate, and if that vibration is too large it may cause occupant discomfort.

Floor vibration is of particular concern to mass timber floor systems. Mass timber's high strength-to-weight ratio makes it an appealing material for use in long-span floor system. However, the lower mass means it is more susceptible to footfall excitation and vibration will more often govern the size of the floor structural elements. As mass timber continues to grow as a building material, it is important to have both accurate structural models and a body of research and in-situ data on vibration performance.

Guidance for designing structure for vibration and completing vibration assessments is established in many well-known documents such as the CCIP design guide for footfall vibration, or the US Mass Timber Floor Vibration Design Guide [1-4]. However, an area of uncertainty that often arises is how non-structural elements in a building, such as the partitions or concrete topping, impact vibration. Some physical testing has noted that non-structural elements have a large impact on floor behaviour [5]. However, the impact of non-structural elements has not been quantified for many in-situ mass timber floors, and few studies have been completed on long-span mass timber floor systems.

As Fast+Epp recently constructed a new home office, an opportunity arose to study some of the uncertainties affecting vibration during the construction of that office. A research program was developed to conduct vibration tests on a full-scale mock-up, and the Fast+Epp home office itself. Assessment of the floor system in Fast+Epp's home office was completed in two phases. In phase one, a physical mock-up of the floor plate used in the office was built and assessed for vibration. A summary of the first phase is provided in Slotboom et al. [6].

The second phase of testing would occur on the home office, and is presented in the following study. Of particular interest for the second phase was how the vibration performance of the structure would change throughout construction. Testing was therefore set up to understand:

- The overall vibration performance of the long span floor system in the building.
- The impact of various structural and non-structural components on the floor's vibration performance.
- How predictions from finite element models compare to actual measurements of the floor system, and how modelling of future floors can be improved.

The results are used to recommend some practical guidelines for modelling practices.

2 EXPERIMENTAL PROGRAM

2.1 BUILDING OVERVIEW

The Fast+Epp home office, as shown in Figure 1, is a four-story structure with a 35m long by 12m wide floor plate and 3mx12m structural grid. The office features a long span mass timber floor system, consisting of 3ply CLT panels supported on glulam beams that span up to 11.5m

[1] Christian Slotboom, Graduate Engineer, Fast+Epp, cslotboom@fastepp.com
[2] Carla Dickof, Associate Principle, Fast+Epp, cdickof@fastepp.com
[3] Tobias Fast, Partner, Fast+Epp, tfast@fastepp.com

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between steel columns and a CLT wall. The floor panels are V2M1.1, 105mm SPF CLT, that span perpendicular to the beams and continuously across 2 or 3 bays. The beams used were made of grade 24f-E 608mm deep Douglas Fir Glulam, with width that varies between 265mm, 315mm, and 365mm depending on the floor and location. On the west side of the building, the glulam beams are connected to a series of steel HSS columns with a bolted knife plate connection. A similar detail is used to connect the beams to a CLT wall on the north side of the building. Figure 2 below summarizes floor grid building, while Figure 3 shows the knife plate connection used.

***Figure 1:** The Vancouver home office of Fast+Epp.*

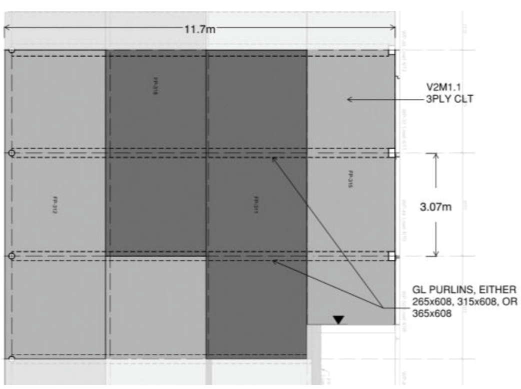

***Figure 2:**Summary of the floor grid*

The CLT panels were connected at panel joints plywood splines. These consisted of a 140 mm x 25 mm thick D.Fir strip of plywood along the CLT panels full length. Splines were connected to the panel with 4x60 ring shank nails spaced at 64mm on centre. CLT panels were connected to beams using 6Øx220 partially threaded screws with counter sunk heads, spaced at 400mm. The screw spacing was informed using studies performed on the mock-up [6], where it was found adding additional screws had diminishing returns on stiffness. After the panel was installed, a 50mm concrete topping was applied over an acoustic layer on top of the CLT panel. The concrete had a compressive strength of 32Mpa, and no supplementary connectors were used to join the concrete floor with the CLT panel.

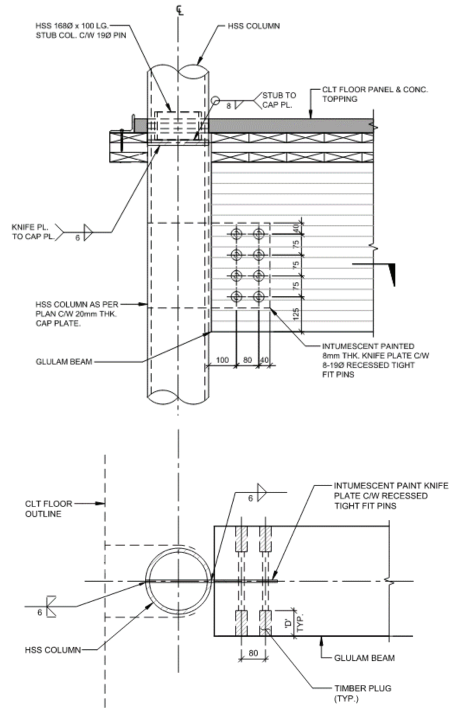

***Figure 3:** Colum to beam connection detail.*

2.2 EXPERIMENTAL PROGRAM

Vibration testing on the office was completed at several points during construction to both gauge the performance of the building, and understand the effect of non-structural components on it. A total of four building states were measured:

1. B1: bare superstructure,
2. B2: post concrete topping,
3. B3: post partition installation, and
4. B4: with full building furniture fit-out.

For each building condition, three types of vibration tests were run: ambient vibration, heel-drop, and walking vibration. Vibration was measured with a Crystal Instruments Spyder Analyzer vibration sensor and two accelerometer sensors used to measure vertical accelerations. Most of the testing used a recording increment of 0.002s. Tests were run in a suite, where a receiving location was chosen, and a series of walking trials were completed. Note that because testing occurred during construction, it was not always possible to test all floors of the building, or use the exact same test locations between each test, due to material on site. In particular, testing on L4 was not completed in B1, because the level had not been constructed on the test day.

Walking tests were completed by recording the acceleration response of the floor while a single tester walked along specific walking paths at specific frequencies. Figure 4 provides an overview of the recorder location and typical paths used for the walking tests on the structure. For each receiver location chosen, a

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total three walking paths were chosen, and four walking frequencies were tested for each path.

Table 1: *Walking test suite designations.*

Building State	Level	Receiver Locations
B1	L2	R1, R2, R5
	L2	R1, R2, R5
B2	L3	R1
	L4	R1, R2, R5
B3	L4	R1, R2, R5
B4	L3	R1
	L4	R1, R2, R3

Figure 4 also shows typical walking paths and heel drop locations taken for the structure. For each receiver location, several walking paths are walked. The walking paths were chosen and named as follows:

- W1: Walking transverse to the primary beam span, along the centre of the outermost panel.
- W2: Walking parallel to the primary beam span, at the mid-point of the panels.
- W3: Walking parallel to the primary beam span, directly over the beam.

For each walking path, walking tests were completed using four walking frequencies. These were named as follows:

- F1: 1.25 Hz (75 Steps per min).
- F2: Hz (95 Steps per min).
- F3: Hz (110 Steps per min).
- F4: Hz (125 Steps per min).

In heel-drop tests, one vibration sensor was placed at the location of the heel-drop, and another was placed far from them. A tester performed a heel-drop near the accelerometers and the floors acceleration response to that impact was recorded. The heel-drop tests were completed in a grid of points across the floor, where the main sensor location remained in the same place, and second sensor moved with the heel drop location. These tests were used to get a more accurate measurement on the frequencies floor modes were responding at, as walking vibration tests do not have a clear signal decay. Table 2 overviews the heel drop grids that were completed on the floor.

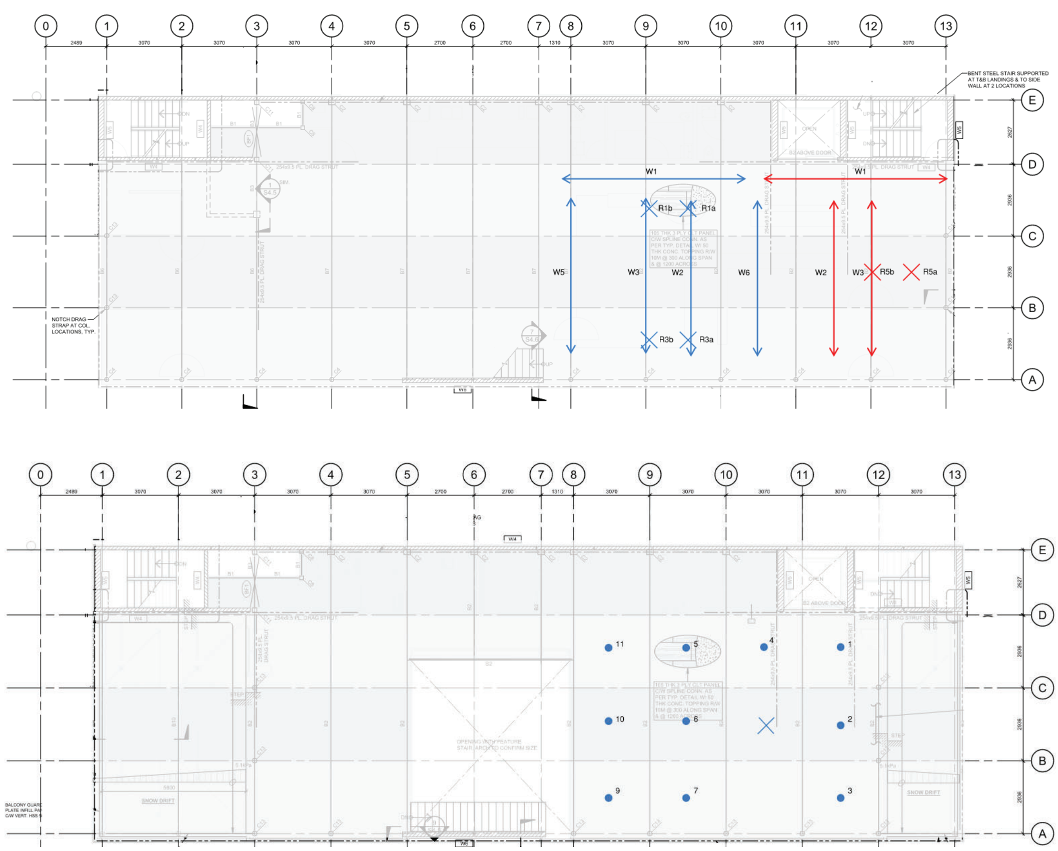

Figure 4: *Summary of walking paths (Wi) and receiving locations (Rj) for testing on 3rd floor, and the heel drop locations on the fourth floor.*

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Table 2: Heel Test Suite Designations.

Building State	Level	Receiver Location	Heel drop Locations
B2	L3	R6	H1-H9
B3	L4	R6	H1-H11
B4	L3	R6	H1-H11
	L4	R6	H1-H8

Combined, the above naming convention can be used to give each test a designation, based on a test's building state and location. Each walking suite is given a representation based on where it was completed, Ba-Lb-Rc-Wd-Fe, where:

- "a" represents the building state,
- "b" represents the level testing occurred at,
- "c" represents the receiver location chosen,
- "d" represents the walking path taken,
- "e" represents the walking speed used.

Similarly, heel drop experiments were given a designation Ba-Lb-Rc-Hd, where parameters "a", "b", "c" are as described above, and "d" is the number of the heel drop test used.

2.3 EVALUATION METHODOLOGY

There are two significant outputs from each vibration measurement: a signal of the raw acceleration over time, and a frequency spectrum of the Fourier transform that shows which frequencies are dominant in the vibration. The time-acceleration response is used to measure the magnitude of vibration that occurs in the floor, as well as qualitative gauge the response of the floor. Qualitative measurements were completed by visual inspection of vibration signals, to determine if the floor has resonate response to loads and can be characterized as a low frequency floor. The magnitude of vibration is measured by calculating the Root mean Square (RMS) of the vibration signal. Recommendations from CCIP-016 [2] were used to calculate the RMS, including using an averaging time of 1s, and applying a band pass filter to the raw data.

The results frequency domain data were largely used to predict the fundamental frequency of the floor, and estimate changes in system stiffness. The fundamental frequency of the floor was estimated using the frequency of the lowest local peak in the frequency response spectrum. Changes in frequency between building states estimated using first principles, as frequency (ω) is known to be proportional to system stiffness (k) and mass (m) per equation (1).

$$\omega \sim \sqrt{\frac{k}{m}} \tag{1}$$

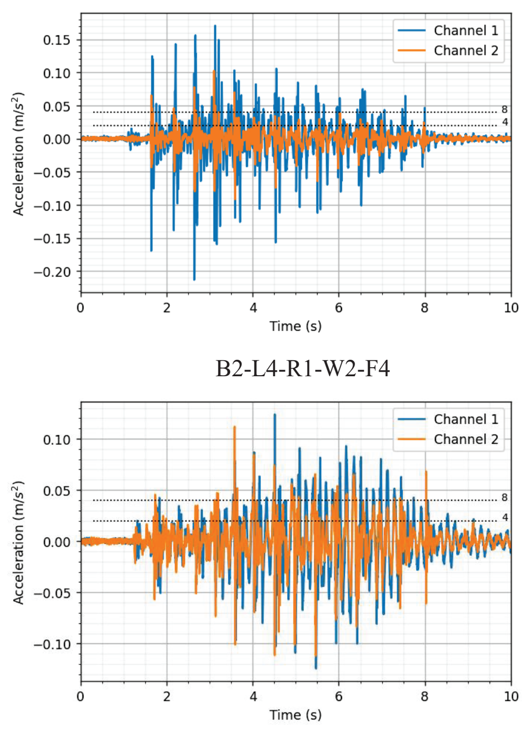
Damping measurements for the floor were also measured using the results of heel-drop tests. A logarithmic decrement method was used to measure damping, where peaks from the damped signal were extracted and then an exponential function was fit to those peaks. The resulting function was used to estimate damping in the system.

3 EXPERIMENTAL PROGRAM RESULTS

Testing was completed. Due to issues accessing the site during construction, different testing locations were used between the L3 test on B1, and L3 tests on B2/B4. The effect of this change in location is discussed in Section 4.3.

3.1 ACCELERATION RESULTS

Raw acceleration signals were examined qualitative assess if the floor behaved like a low frequency floor, where response is governed by a resonant build-up of vibration, or a high frequency floor, where response is governed by impulse. Based on the data collected, the floor had a mixed response but mostly behaved as a high frequency floor. However, for many tests it was also observed that the vibration did not fully decline between footfalls. This indicates that there was some resonant response, and the floor did not behave purely as a high frequency floor. Figure 5 shows summarize typical vibration signals seen at walking speed F4. Some resonant behaviour is observed in the second signal, while the first signal is governed by transient response.

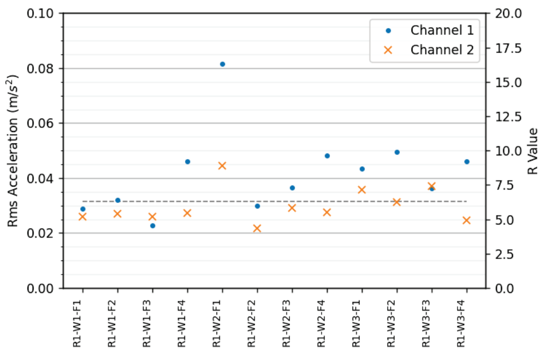


B2-L4-R1-W2-F4

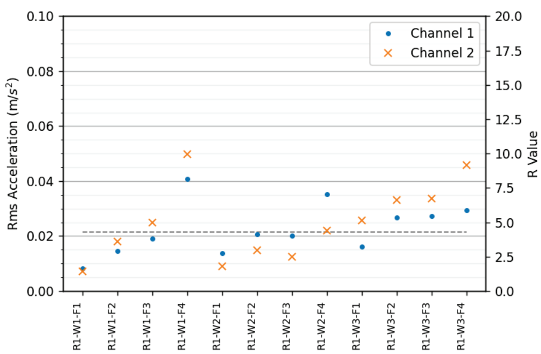
B2-L4-R2-W3-F4

Figure 5: Acceleration observed in for different walking paths in B2-L4.

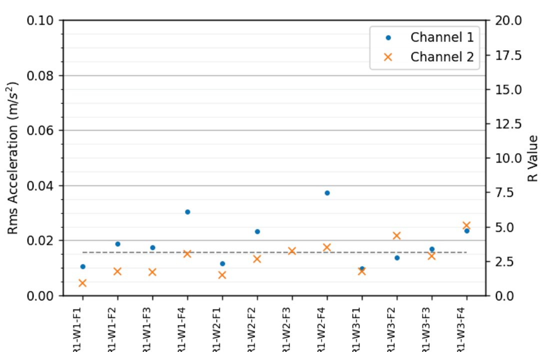
To gauge the comfort of the floor for vibration, the RMS for each acceleration signal was calculated, and the peak value was extracted from the RMS plot. Figure 6 overview the peak RMS and R value recorded by each channel for mock-up one and two, respectively. R values were determined by dividing the RMS acceleration by a baseline value of 0.005m/s^2. Also included in each figure is a line of the median data observed in the experiment. There is generally an upward trend in response factors as walking speeds increase, and the median RMS acceleration generally decreased as the building became more complete. The median R value in each suite of tests is recorded experiment in Table 3. The same trend from Figure 6 is observed, with decreasing RMS accelerations as the structure progresses through construction.



B2-L4-R1



B3-L4-R1



B4-L4-R1

Figure 6: *Peak RMS acceleration for the test series path L4-R1 for conditions B2-B4*

Table 3: *RMS Acceleration vibration results for L3 and L4*

Floor	State	Suite	Median R-Value
L3	B1	R1	16
		R2	8
		R5	16
	B2	R2	5
	B4	R1	6
L4	B2	R1	6
		R2	4
		R5	6
	B3	R1	4
		R2	5
		R3	6
		H	7
	B4	R1	3
		R2	3
		R3	3

For office occupancy, a response factor in the range of 4-8 is a typical acceptable limit. The observed response factors were generally within the range of 5-20, showing the floor was on the edge of what would be considered commercially acceptable.

3.2 FREQUENCY RESULTS

The frequency response of the structure was characterized by an initial peak in the range of 5-10Hz, followed by various local peaks between 15Hz and 30Hz. Based on experience from phase one of testing on the full-scale mock-up, it's expected that the lower peak corresponds to vibration modes from exciting the beams, and the higher frequency peaks come from modes where the panel between beams is excited. To allow for easy comparison, the frequency response plots have been normalized by dividing each graph by its peak response. Figure 7 show typical frequency responses observed for a walking test, and a heel drop test.

Like RMS acceleration, the frequency where the peak occurs is extracted in each test, with a sample plot shown in Figure 8. Table 4 summarize the predicted fundamental frequency of the floor observed in each test, based on figures similar to Figure 8. Here the fundamental frequency of the system is estimated as approximately the lowest frequency peak observed during testing for all channels.

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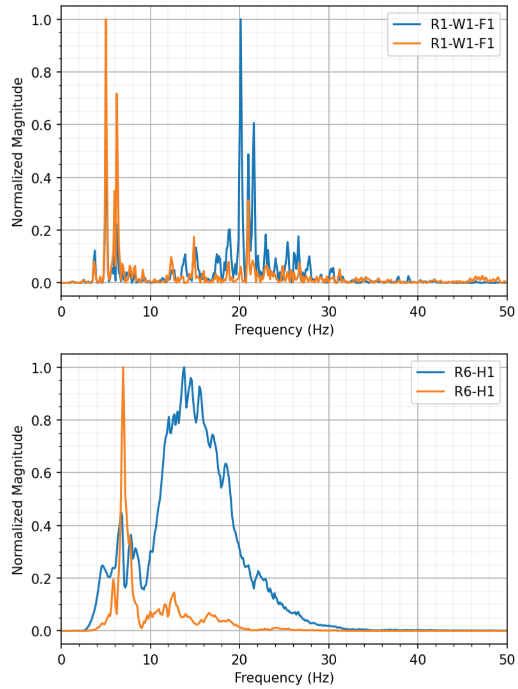

Figure 7: *Sample frequency response plots for Channel 1 (blue) and Channel 2 (orange) from Building condition 4 on the 4th floor. Channel 1 is centred on a beam on the, while Channel 2 is centred between beams in the middle of the CLT panel.*

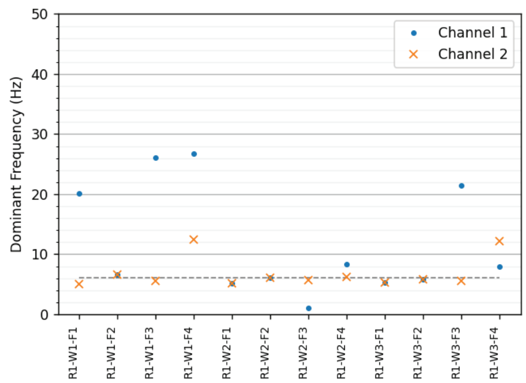

Figure 8: *Peak frequencies observed for walking experiments at R1 in B4-L4.*

Table 4: *Estimated fundamental frequency results for L3 and L4*

Floor	State	Suite	Fundamental Frequency
L3	B1	R1	8
		R2	9
		R5	11.5
	B2	R2	7.5
		H	7
	B4	R1	8
		H	8
L4	B2	R1	5.5
		R2	5.5
		R5	8
	B3	R1	6.5
		R2	16
		R3	7.5
		H	7
	B4	R1	6
		R2	6.5
		R3	7
		H	7

Comparing results between B1 and B2 for L3, a smaller decrease in frequency was noted than expected. When the topping was added to the building, the weight of the system increases from approximately 0.75kPa to 1.95kPa – more than doubling. Based on equation (1), the additional weight of concrete should have decreased the fundamental frequency approximately 40% however, a drop in frequency of about 15% was observed. This effect is discussed further in section 4.3.

3.3 EQUIVALENT VISCOUS DAMPING

The equivalent viscous damping of the floor was also estimated using the heel drop test data. It was generally found that there was a high variance in the predicted damping. This is likely due to vibration not decaying in a cleanly exponential decay, with instead a large drop occurring in the signal after the initial peak. While there is a large spread in the observed values, the damping generally increase in the building through construction.

Table 5: *Damping measurement in the floor system.*

Floor	Condition	Mean (%)	S.D. (%)
L3	B2	3.0	1.0
	B4	4.7	1.3
L4	B2	1.6	0.5
	B3	2.2	1.2
	B4	3.6	1.8

3.4 COMPARISON WITH MOCK-UP

Comparisons were also made between the frequency of the mock-up observed in phase one of testing [6] and the building in condition B1 on L3. Results were used for the mock-up model in condition M1c, which used 6Øx220 screws between the CLT panel and glulam column spaced at 300mm. Table 6 summarizes median frequency observed in both experiments. A higher fundamental frequency was observed in the building compared to the mock-up, indicating that the building was stiffer. The mock-up rested directly on supports in bearing, so an explanation for this increase in stiffness is that the knife plate connectors used have more stiffness than a simple bearing connector.

Table 6: *Observed frequency in Mock-up vs. in the building.*

Test Location	Test Designation	Median Frequency (Hz)
Mock-up	M1c	7.5
Building	B1-L3	8.5

4 NUMERICAL STUDIES

To understand how well Finite Element Model (FEM) predictions compared to observations in experimental data, a set of simplified numerical models were created of the floor in the software package RFEM [9]. The goal of this analysis was not to create highly detailed FEM models; it's known that a sufficiently detailed finite element model can closely match experiment. Instead, the numerical models were built to understand how typical models used by a designer might compare to experimental data. Therefore, only information that designers are likely to have during design was considered, for example, the stiffness from partitions was neglected as this is not typically known. First, baseline models are made and compared to experimental results, then parameters in those models are varied to assess the impact on the dynamic properties of the floor.

4.1 MODEL OVERVIEW

The models considered the whole floor plate for L3, and the right portion of the floor plate on L4. The structural elements considered were as described in section 2.1, and material properties of the elements were estimated from CSA 086.6 [8] for timber and CSA A23.3 [9] for concrete. Figure 9 provides an overview of the baseline models for the first and fourth floor respectively. The CLT using 2D shell elements and RFEM's RF-laminate module [10] while beams were simple beam elements.

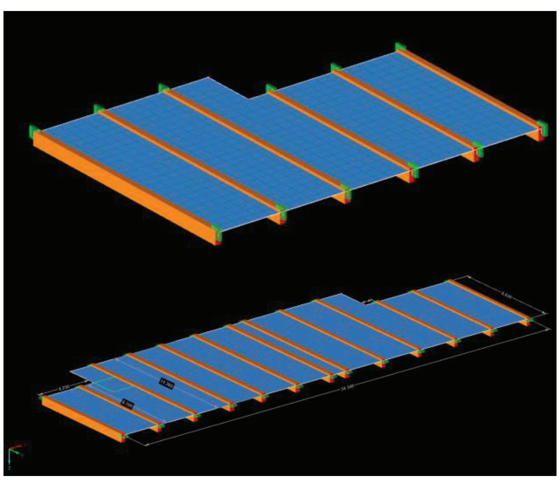

Figure 9: *The FEM model for L4 (top) an L3 (bottom).*

The mass in the model is estimated based on the site conditions observed for each test. Table 7 summarizes the total loads considered, broken down by load source for dead load (DL), superimposed dead load (SDL), and live load (LL). Loads are reported in kPa and converted by RFEM into an appropriate mass. Note that DL has been reported as an equivalent uniform load, however, in the model it assigned to elements as a self-weight. To account for moisture uptake of the members, a self-weight of 4.6kN/m3 is used for the CLT, while 5.3 kN/m3 is used for glulam. In B1, only the self-weight of the structure is considered, while in B2 a 50mm concrete topping is also included in the analysis. For B2, B3 and B4, the weight of the mechanical systems and furnishings are estimated based on the state of the building when tested. No change in mass was noted between B2 and B3 because a similar amount of construction material was on site for each study.

Table 7: *Vibration loads applied to model.*

Condition	DL (kPa)	SDL (kPa)	LL (kPa)
B1	0.7	-	-
B2	0.7	1.2	-
B3	0.7	1.2	-
B4	0.7	1.5	.2

How connections and boundary conditions are modelled will generally have a big impact on system stiffness and the fundamental frequency. For the Yukon office floor there are three main connection fixities to consider: fixity of the panel-to-panel spline connection, fixity of the beam-to-panel, and fixity of the beam to column/wall connection. In the base model a fixed connection was used between the beams, which is also the boundary condition for the model. Because the fixity of beam to column connection has a large impact on system stiffness, both a fixed and pinned connector were studied.

The beam to panel connection will affect composite action of the system. Findings from the mock-up were used to estimate the degree of composite action, with a predicted increase in system stiffness of 40%. Composite included in the model using an offset between the slab and the beam that would increase the system's EI by the appropriate amount. The effect of spline connections is not considered in the assessment, and it is assumed that the concrete toping transfers vibration between with an unobstructed load path.

4.2 COMPARISON ON OF MODELS

Once the base model was finished, several parametric studies were run on typical model assumptions. These studies are used to understand what parameters are most important to include in the model. The parameters considered include: beam stiffness, CLT stiffness, degree of composite action, stiffness from concrete topping, and boundary connection.

Variations of beam and CLT stiffness were completed by applying a stiffness modifier of 0.75 or 1.25 to the system. increasing and decreasing stiffness by 25%. These studies were run to understand how the uncertainty in timber

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material might change observed results, and only run on L3. Degree of composite action was measured by running models that had no offset between the CLT and glulam beam, and thus no composite action. The effect of boundary condition and composite action assumptions were also examined. In the pin-connected model, the moment restraint was released at each column to beam connection.

Stiffness from concrete topping was included by adding the concrete stiffness to the CLT stiffness. Including the stiffness this way assumes no composite action between the two layers. Table 8 summarizes the change in stiffness of the CLT in each direction, where EI_x and GA_x are in the strong axis, and EI_y and GA_y are the weak axis. It can be observed that the concrete topping significantly increases the weak axis bending and shear stiffness.

Table 8: *Stiffness per unit meter of bare CLT and topped CLT.*

Condition	EI_x	EI_y	GA_x	GA_y
	($Nmm^2/m \times 10^9$)		($N/m \times 10^6$)	
CLT	883	34	7.5	7.5
CLT & Topping	1230	386	682.5	682.5

Table 9 and Table 10 summarizes the changes made to the base model, as well as the resulting changes to fundamental frequency. These can be compared to the in-situ experimental data to understand how accurate the trends are. For each model and test, the mode shape is extracted from the FEM models at the input location of vibration testing. The effect of concrete topping was examined in the base model, the model using no composite action, and the model using a pin connection. It was found that including the concrete topping increased system stiffness in the range of 5-10%. For variations in stiffness, it was found that variations in CLT stiffness had a low impact on the total stiffness of the system, while variations in beam stiffness affected the frequency on the order of 10-15%. Changes to the composite action also had a moderate effect on the system, on the order of 5-10%. Finally, it was noted that boundary condition had a large influence on the systems stiffness, where the fundamental frequency decreased in the range of 30-40% for models with a pin connection.

Table 9: *Measurements of on L3.*

Model	Condition	Level 3 B1 (Hz)	Level 3 B2 (Hz)	Level 3 B4 (Hz)
In-situ Experimental Data	-	8.5	7	8
Base Model	No topping	14.4	10	8.7
	Topping	-	10.6	9.3
Beam Stiffness	0.75	12.9	8.9	7.8
	1.25	15.7	10.8	9.5
CLT Stiffness	0.75	14.1	9.8	8.4
	1.25	14.7	10.1	8.8
Non-composite	No topping	13.5	9.4	8.3
	Topping	-	10	8.9
Pin Connection	No topping	9.1	6.3	5.6
	Topping	-	6.5	5.8

Table 10: *Measurements of on L4.*

Model	Condition	Level 4 B1 (Hz)	Level 4 B3 (Hz)	Level 4 B4 (Hz)
In-situ Experimental Data	-	-	5.5	6.5
Base Model	No topping	14.2	8.6	7.6
	Topping	-	9.0	8.2
Non-composite	No topping	13.3	8.1	7.2
	Topping	14.1	8.7	7.7
Pin Connection	No topping	8.9	5.5	4.9
	Topping	9.2	5.7	5.1

4.3 IMPACT OF NON-STRUCTUAL ELEMENTS

Results from FEM data can be used to predict the effect of non-structural elements on the system. In all numerical models considered, a downward trend in fundamental frequency is observed. Figure 10 provides an overview of the FEM base and pinned model predictions, compared to the experimentally observed data in experiment B1 to B4 on the third floor. The experimental frequencies reported are approximately the lowest from all test groups in Table 4. FEM results are taken from Table 9, and the mode used corresponds to the lowest mode active at the testing location. Results for the experimental data were closer to the pinned model in the bare floor condition, and closer to the fixed model in the final building condition.

A different trend in the observed frequencies was also noted between the FEM data and experimental response. The FEM models had their fundamental frequency decrease by approximately 30% from B1 to B2, and B2 to B4. This contrasts with the experimental model that decreased by only 10% from B1 to B2, and increased marginally between B2 to B3.

Based on Table 9, it's expected that the FEM would predict frequencies approximately 5% higher if the concrete topping is included in the model in a non-composite way. This difference is not enough to explain the differences observed between FEM predictions and experimental data, indicating that some other factor increased the system stiffness between both building conditions. This suggests that the non-structural concrete topping and other elements had a larger impact on stiffness than expected.

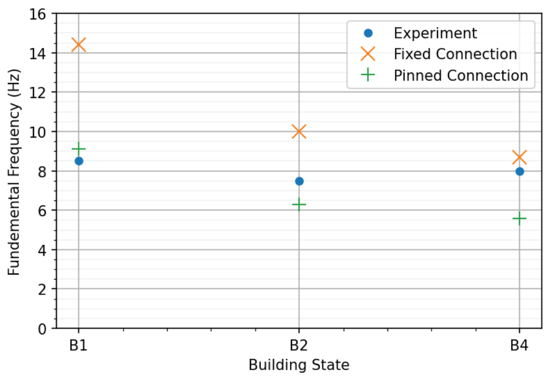

Figure 10: *Comparison of FEM and experimentally observed frequencies.*

The FEM data can also be used to assess the effect changing locations had on the experimental fundamental frequency. As noted prior, different testing locations were used between B1 and B2/B3. A prediction of the frequency at the original testing location can be made based on FEM data. In the models, it was noted that a different mode was active at L3 for the test location used in B2 and B4. This mode had a frequency approximately 20% higher than the base mode shape used in L1. The frequency can therefore be adjusted to approximately 6Hz at B3, and 6.5 Hz at B4.

The fundamental frequency decrease of 25% between B1 and B2 is more closely in line with the expected 40% reported in section 3.2, but still less than expected. This suggests that the concrete topping, or another factor had a larger than expected impact on the stiffness of the system. Some possible reasons for the increases in stiffness in B2 include: additional composite action occurring in the CLT to beam system due to clamping from the concrete weight; or contribution of the concrete slab to the floor stiffness, in either composite or non-composite action.

Similarly, the increase in frequency between B2 and B4 observed in experiment suggests that the non-structural elements such as partitions had a significant effect on the stiffness of the system for vibration.

5 CONCLUSIONS

A series of vibration tests were performed on Fast+Epp's new home office during its construction. In the final state of the building, it was found that the long-span floor system of Fast+Epp's home office was within vibration tolerances for office building. Most observed acceleration signals during walking tests had a response value less than 8. It was also found that non-structural elements had a large impact on the vibration performance of the floor. Based on experimental data normalized with FEM results, on the third floor, the fundamental frequency changed from to 8.5Hz to a 6Hz going from bare floors to floors with topping. This was less than the decrease of approximately 40% that would have been expected with no additional stiffness. The fundamental frequency increased as partitions and furnishings were added, to the building, indicating that these elements added enough stiffness to the floor to overcome the additional mass added.

Finally, during numerical modelling of the floor it was found that the parameter with the largest effect on the floor system was the boundary condition used. It was found that the floor in its final built condition was most accurately described using a fixed connection. It was also found that the trend of frequencies observed during construction in FEM modelling did not match the trend of data from experiments. The inclusion of non-composite stiffness from the concrete topping was not enough to account for the change in stiffness expected. Both trends suggest that non-structural components added more stiffness to the system than expected.

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