SERVICEABILITY OF FLOOR SYSTEMS WITH WOOD I-JOISTS AND CONCRETE TOPPING

By Y.H. Chui
and I. Smith

May 1997

CMHC Project Officer: Silvio Plescia

This project was carried out with the assistance of a grant from Canada Mortgage and Housing Corporation under the terms of the External Research Program (CMHC File 6585-C082). The views expressed are those of the author and do not represent the official views of the Corporation.

SERVICEABILITY OF FLOOR SYSTEMS WITH WOOD I-JOISTS AND CONCRETE TOPPING

CMHC External Research Program Project
Final Report

by

Y. H. Chui and I. Smith

Faculty of Forestry and Environmental Management
University of New Brunswick
P.O. Box 44555
Fredericton
New Brunswick E3B 6C2

This project was carried out with the assistance of a financial contribution from Canada Mortgage and Housing Corporation under the terms of the External Research Program. The views expressed are those of the authors and do not represent the official views of CMHC.

May 1997

CONTENT

EXECUTIVE SUMMARY

This project studies the serviceability behaviour of a form of floor construction comprising wood I-joists, wood-based sub-floor and a concrete topping, which has become increasingly popular in recent years in multi-family residential buildings. The prime purpose of the concrete topping is to provide enhanced fire safety and acoustic performance. Its presence however also has an impact on two of the most important serviceability performance criteria: static deflection and vibration of a floor system since it adds significant mass and stiffness to the system. Despite its increased use very little is known with regards to how the addition of concrete topping influences deflection and vibration performance of wood floor systems. This project is intended to provide some understanding in this area, and generate preliminary design and construction guidelines.

The project was conducted in three phases. In Phase 1 a series of composite beams comprising a single wood I-joist, a narrow strip of wood-based sheathing and a concrete topping were tested. These beams were tested before and after the addition of concrete topping. This phase studied primarily the influence of joist size and use of light shear connectors (double-headed nails) between concrete and sheathing material on beam deflection and vibration properties. The results showed that the addition of concrete topping increased the stiffness and lowered the first natural frequency of a beam. In addition the use of light shear connectors did not produce any noticeable increase in beam stiffness. These test data were supplemented by results from computer modelling of beam behaviour in Phase 2. The computer model was first verified by test data from Phase 1 to ensure the accuracy of its output. It was then used to study the influence of connection stiffness between concrete and sheathing, concrete width (i.e. joist spacing in a floor) and concrete thickness on beam behaviour. It was found that beam behaviour was not sensitive to connection stiffness for most practical situations. While any increase in concrete width and thickness within the practical range led to a moderate increase in beam stiffness, it could cause a substantial reduction in first natural frequency of the beam. This reduction in natural frequency may cause vibration problems in floor systems since humans are more sensitive to low frequency than high frequency vibrations. Phase 3 involved testing of a full-size floor system which was constructed using one of the joist sizes tested in Phase 1. The objective was to evaluate if some of the findings in Phases 1 and 2 were still applicable to a two-way floor system. The test data showed that a substantial increase in system stiffness was achieved by the addition of a 38mm thick concrete topping compared with the bare floor. However the beam analysis result with respect to the large reduction in natural frequency was evident in the floor test measurements. In addition, it was found that the shrinkage of concrete caused the topping to deform as a 'dish' which led to localised low frequency vibration especially near the edges of the floor.

Based on this study, it is recommended that:

1. Shear connectors be used to provide some form of connection between the sub-floor and concrete to minimize potential dishing of concrete during drying.
2. Natural frequencies of floor systems with an estimated service loading, be checked at the design stage to ensure that they are above the human sensitive range of 4 to 8 Hz.
3. The substantial contribution of concrete topping to the floor system stiffness be properly utilized at the design stage using an appropriate calculation procedure.

RÉSUMÉ

Cette étude porte sur la tenue en service d'un plancher réalisé avec des solives en I, un support de revêtement de sol en bois ainsi qu'une chape de béton, une forme de construction de plus en plus populaire depuis quelques années dans la réalisation de collectifs d'habitation. L'utilité première de la chape de béton est la sécurité incendie accrue et l'amélioration de l'isolement acoustique. Toutefois, sa présence a aussi une incidence sur deux des plus importants critères de performance en matière de tenue en service : le fléchissement statique et la vibration du plancher. En effet, il procure à l'ensemble une masse et une rigidité considérables. Malgré cette utilisation accrue, on en sait très peu sur les conséquences que peut avoir l'ajout d'une chape de béton sur le fléchissement et la vibration des planchers en bois. Cette étude vise à jeter un peu de lumière sur cette question dans le but de proposer une amorce de directives de conception et de construction.

L'étude a été menée en trois phases. À la phase 1, une série de poutres composites constituées d'une seule solive de bois en I, d'une mince bande de revêtement intermédiaire en bois ainsi que d'une chape de béton ont été mises à l'essai. Ces poutres ont fait l'objet de tests avant et après la mise en oeuvre de la chape de béton. Cette phase a surtout servi à étudier l'incidence de la taille des solives et de l'utilisation de petits connecteurs (clous à deux têtes) entre le béton et le revêtement intermédiaire sur le fléchissement de la poutre et sur ses caractéristiques de vibration. Les résultats montrent que l'ajout de la chape de béton augmente la rigidité et réduit la première fréquence propre de la poutre. De plus, l'emploi de petits connecteurs n'a pas permis d'obtenir une augmentation appréciable de la rigidité de la poutre. À ces données d'essai s'ajoutent les résultats d'une modélisation par ordinateur du comportement de la poutre dans le cadre de la phase 2. Le modèle informatique a d'abord été vérifié au moyen de données d'essai tirées de la phase 1 afin de déterminer la précision de ses résultats. On s'en est alors servi pour étudier l'incidence de la rigidité de l'assemblage béton-revêtement intermédiaire, de la largeur du béton (c.-à-d. l'espacement des solives constituant le plancher) et de l'épaisseur du béton sur le comportement de la poutre. On s'est aperçu que le comportement de la poutre n'était pas tributaire de la rigidité de l'assemblage pour la plupart des situations pratiques. Bien qu'une augmentation de la largeur et de l'épaisseur, dans des limites raisonnables, ait entraîné un accroissement modéré de la rigidité de la poutre, elle pourrait se traduire par une réduction substantielle de la première fréquence propre de la poutre. Cette réduction de la fréquence propre pourrait entraîner des problèmes de vibration dans les planchers puisque les humains sont plus sensibles aux vibrations de basse fréquence qu'aux vibrations de haute fréquence. La phase 3 consistait à mettre à l'essai un plancher en vraie grandeur réalisé au moyen de l'une des solives utilisées lors de la phase 1. Il s'agissait de déterminer si certains résultats des phases 1 et 2 étaient toujours valables dans le cas d'un plancher à double portée. Les résultats ont montré qu'il était possible d'obtenir une augmentation substantielle de la rigidité de l'ensemble lorsqu'on ajoutait une chape de béton de 38 mm d'épaisseur comparativement à un plancher nu. Cependant, le résultat de l'analyse de la poutre concernant une importante réduction de la fréquence propre s'est avéré évident lors des mesures d'essai effectuées sur le plancher. En outre, on s'est aperçu que le retrait du béton provoquait une déformation de la chape qui prenait ainsi la forme d'une «soucoupe» (concavité), produisant des vibrations locales de basse fréquence surtout près des extrémités du plancher.

Cette étude permet de formuler les recommandations suivantes :

1. Utiliser des connecteurs pour offrir une certaine forme de liaison entre le support de revêtement de sol et le béton afin de réduire au minimum la possibilité qu'il se forme une concavité dans le béton pendant le séchage.

2. Vérifier dès la conception les fréquences propres des planchers par rapport à une charge en service estimative afin de s'assurer qu'elles sont supérieures au registre de perception humain qui oscille entre 4 Hz et 8 Hz.

3. Utiliser de façon appropriée, et ce dès la conception, l'apport substantiel de la chape de béton à la rigidité du plancher au moyen d'une méthode de calcul appropriée.

CMHC SCHL

Helping to house Canadians

Question habitation, comptez sur nous

National Office

Bureau national

700 Montreal Road
Ottawa, Ontario
K1A 0P7

700 chemin de Montréal
Ottawa (Ontario)
K1A 0P7

Puisqu'on prévoit une demande restreinte pour ce document de recherche, seul le sommaire a été traduit.

La SCHL fera traduire le document si la demande le justifie.

Pour nous aider à déterminer si la demande justifie que ce rapport soit traduit en français, veuillez remplir la partie ci-dessous et la retourner à l'adresse suivante :

Le Centre canadien de documentation sur l'habitation
La Société canadienne d'hypothèques et de logement
700, chemin de Montréal, bureau C1-200
Ottawa (Ontario)
K1A 0P7

TITRE DU RAPPORT : ______________________________

____

Je préférerais que ce rapport soit disponible en français.

NOM ______________________________

ADRESSE ______________________________
rue app.

____
ville province code postal

No de télephone (   ) ____________

1.0 BACKGROUND AND SUMMARY OF PROJECT

An increasingly popular form of floor construction found in multi-family residential buildings consists of wood I-joists and a wood-based flooring, overlayed with a concrete topping. Very little is known at present with regards to the serviceability performance of such floor systems. This project evaluates floor stiffness and vibration characteristics before and after the addition of a concrete topping to floor systems constructed with wood I-joists and a wood-based flooring. The evaluation is achieved by a combination of full-sized composite beam and floor tests, and computer modelling. The objective of the study is to develop some preliminary design and construction guidelines for this type of floor construction.

This project is conducted in three phases: composite beam tests, computer modelling and full-sized floor test. In Phase 1 (composite beam tests), beams comprising a single wood I-joist, a narrow strip of wood-based flooring and a concrete topping were fabricated and tested. Tests were conducted before and after the addition of concrete topping. Tests conducted included static deflection and vibration. This would lead to a better understanding of the influence of the concrete topping on both static stiffness (deflection) and vibration characteristics of these composite beams, which form the basic component of the type of floor system studied here. In Phase 2 the influence of the concrete topping on composite beam performance was studied more extensively using a computer model. Test results from Phase 1 are used to guide the computer modelling work. Based on results from this phase, preliminary design and construction guidelines will be developed for the type of systems considered here. Phase 3 will involve testing of a full-size floor system to verify the guidelines.

2.0 PHASE 1 - TESTING OF COMPOSITE BEAMS WITH CONCRETE TOPPING

The objectives of this phase are to evaluate the effects of concrete topping experimentally and to provide test data to verify the computer model predictions in Phase 2. The joists used in this project were supplied by the Montreal plant of Jager Industries Inc. Two joist sizes were obtained: 9½" and 14" deep JSI-20 wood I-joists. Twelve composite beams were tested in total. Half of the beams were tested with no mechanical connection between flooring and concrete (i.e single-headed nails). The other half were tested with mechanical connection through the use of double-headed nails. The width of the concrete topping and floor sheathing was 150mm. The flooring was 18mm thick O2 grade Oriented Strand Board (OSB). The concrete was ready-mixed concrete supplied by a local concrete company, with a specification of 20 MPa compressive strength at 28 days. The spans were based on I-joist manufacturer's recommendations. Typical details of a test beam is illustrated in Figure 1, and details for the twelve beams are given in Table 1.

Table 1 - Details of test beams in Phase 1.

Joist	Span (mm)	Shear connection*	Replicate no.
JSI-20 14"	5600 (18'-1")	2½" Single-headed nails	1, 2, 3
JSI-20 14"	5600 (18'-1")	2¼" Double-headed nails	4, 5, 6
JSI-20 9½"	4178 (13'-5")	2½" Single-headed nails	7, 8, 9
JSI-20 9½"	4178 (13'-5")	2¼" Double-headed nails	10, 11, 12

* Single-headed nail OSB and I-joist flange. Double-headed nail connects OSB, I-joist flange and concrete. Both are common smooth shank nails.

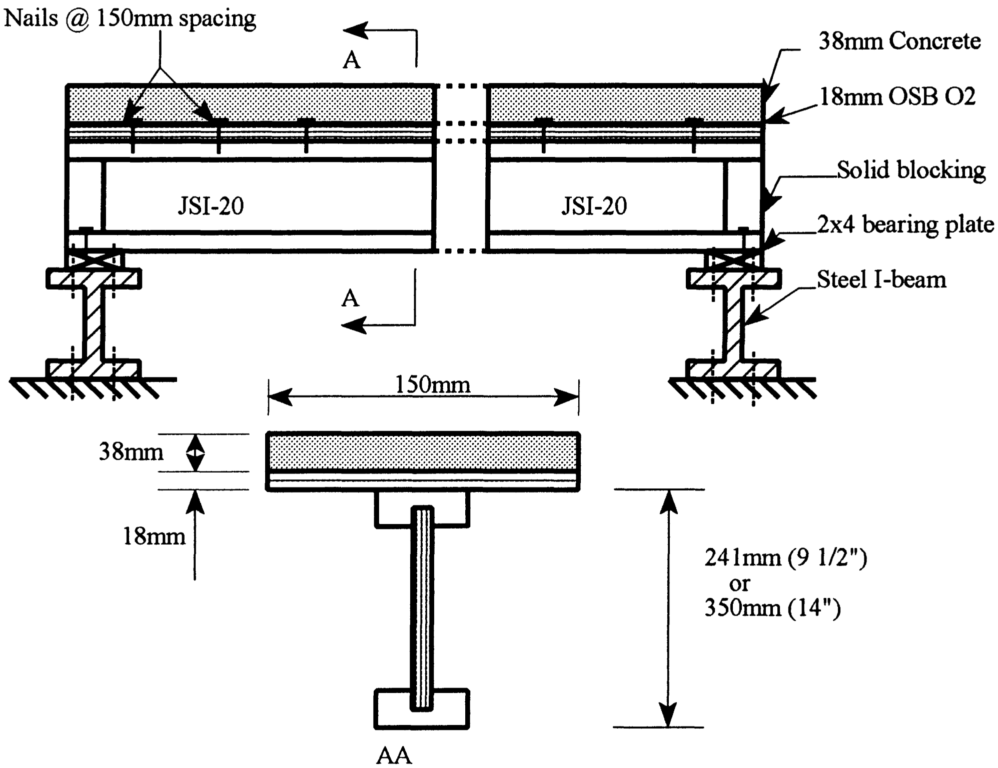

Figure 1- Details of a test beam (schematic).

The test procedure is as follows:

1. <u>Measurement of mass per unit length and mechanical properties of joists</u>

- The mass was measured by direct weighing, and the flexural rigidity (EI) and shear rigidity (G'A) of each joist were determined by a free-free vibration test method proposed by Chui (1991). These properties are required for verification of the computer model in Phase 2. Test results are summarised in Table 2.

Table 2 - Properties of I-joists used in composite beam tests.

Beam no.	Joist	Mass (kg/m)	EI (x 10^{12} Nmm^2)	G'A (x 10^6 N)
1	14" JSI-20	4.47	1.74	5.91
2	14" JSI-20	4.34	1.26	8.62
3	14" JSI-20	4.23	1.46	7.44
4	14" JSI-20	4.62	1.78	8.06
5	14" JSI-20	4.26	1.53	7.25
6	14" JSI-20	4.27	1.32	9.51
7	9½" JSI-20	3.69	0.93	0.58
8	9½" JSI-20	3.40	0.81	0.52
9	9½" JSI-20	3.59	0.48	0.42
10	9½" JSI-20	3.47	0.81	0.54
11	9½" JSI-20	3.60	0.87	0.59
12	9½" JSI-20	3.71	0.52	6.00

2. <u>Determination of natural frequencies and static deflection characteristics of composite beams</u>

- As illustrated in Figure 1, the ends of each composite beam were fastened to steel I-beams. Vibration and static deflection tests were conducted at the following stages of construction :

  i) bare I-joist

  ii) I-joist + 18mm OSB floor sheathing

  iii) I-joist + 18mm OSB floor sheathing + concrete (1 month)

  iv) I-joist + 18mm OSB floor sheathing + concrete (4 months)

The first natural frequency (f_1) of each beam was determined using the modal testing technique (Ewins, 1986). The static deflection characteristic was determined by applying a point load at mid-span using a pneumatic actuator and measuring the deflection under the load. To avoid over-stressing, they were loaded gradually up to maximum loads of 5 kN and 3 kN for the beams made with 14" and 9½" deep joists respectively. The load was measured by a force transducer and the deflection monitored by a displacement transducer. A continuous record of load versus deflection was then obtained. It was noted that the load-deflection response was clearly linear. The stiffness characteristic discussed here is defined as the load per unit deflection (K) and is represented by the slope of the load versus deflection line. Test results at each stage of testing are presented in Table 3.

Table 3 - Summary of test results on composite beams.

No.	Joist	Nail[1]	Joist		Joist+OSB		Joist+OSB+Conc (1 month)		Joist+OSB+Conc (4 months)	
			f_1 (Hz)	K (N/mm)	f_1 (Hz)	K (N/mm)	f_1 (Hz)	K (N/mm)	f_1 (Hz)	K (N/mm)
1	14"	DH	31.12	473	28.13	444	17.87	455	15.37	508
2	14"	DH	30.87	303	26.13	375	16.12	356	15.62	455
3	14"	DH	32.50	366	29.50	465	16.87	443	15.62	518
4	14"	SH	33.12	451	29.50	472	16.75	491	17.12	436
5	14"	SH	32.62	420	31.12	470	16.75	449	15.75	470
6	14"	SH	30.87	369	29.25	424	17.00	381	15.75	469
7	9½"	DH	37.00	345	31.15	375	20.75	369	18.37	396
8	9½"	DH	36.00	286	29.75	319	19.25	335	17.62	343
9	9½"	DH	35.50	279	31.75	294	19.75	344	17.62	407
10	9½"	SH	35.25	273	33.00	305	17.62	330	17.27	371
11	9½"	SH	37.25	301	29.75	317	17.62	296	17.62	394
12	9½"	SH	34.75	300	32.50	355	20.25	338	17.62	346

[1] SH- Single-headed nail, DH - Double-headed nail

Two important observations can be made from the results presented in Table 3. The first is that there is no evidence to support the proposition that the use of double-headed nails improves the stiffnesses of the test beams. Laboratory experience revealed that there was no bonding between concrete and OSB sheathing. The fact that the two types of nails produced similar beam stiffnesses, as evidenced by the closeness of K and f_1 values for the two groups for each joist size, indicates that some form of shear transfer occurred between concrete and OSB, presumably through friction. The joint stiffness produced by friction is probably of similar magnitude to the stiffness of the joint containing double-headed nails, on a per unit length basis. The second observation is the increase in beam bending stiffness with time, as evidenced by the increase in K values. The exact reason for this is unknown, but it is postulated that it may be caused by the shrinkage of OSB which closed any drying cracks in the concrete, which helped to produce a stiffer beam system. Sagging of the beams was also observed after 4 months. This may be caused by a combination of the shrinkage of concrete and creep under the sustained dead load. The sagging of beams is believed to explain the lowering of the natural frequency, despite the increase in beam static stiffness. However further studies are required in order to fully explain this behaviour.

3. Evaluation of concrete-to-OSB connection stiffness and modulus of elasticity of concrete

\- Tests were conducted to determine the stiffness of the concrete-to-OSB connection and the modulus of elasticity (MOE) of concrete. Small joint specimens and standard concrete cylinders (150mm diameter x 300mm height) were prepared when concrete was placed onto the composite beams. In the case of the joint tests, twelve specimens were made for each nail type (single- and double-headed). Details of the joint test specimens are given in Figure 2. It turned out that in the joints with single-headed nails, there was no bonding between concrete and OSB. Thus only the joints with double-headed nails were tested.

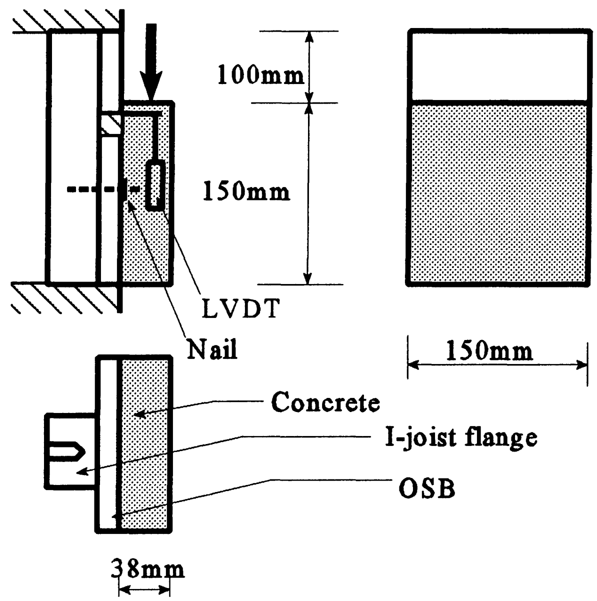

Figure 2 - Details of nailed joint specimen with a double-headed nail.

In the case of concrete cylinders, six specimens were made. Half of the number of specimens in each group (either joint or concrete) were tested after 1 month and the other half after 4 months. In the joint tests, the OSB and I-joist flange were clamped in place and a load applied to the concrete in a shearing action. The slip between concrete and OSB was measured by a displacement transducer (LVDT) and the applied load measured by a force transducer. A typical load-slip response curve is shown in Figure 3. The stiffness value is assumed to be the secant stiffness at one-third of the ultimate load of each joint. The ultimate load and stiffness of each joint are given in Table 4. The concrete test results are given in Table 5.

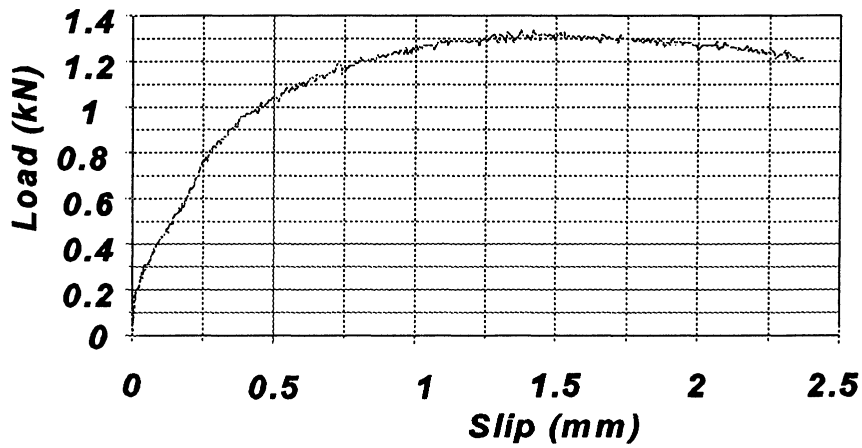

Figure 3 - Typical load-slip response curve for concrete-to-OSB joint.

Table 4 - Results of concrete-to-OSB joint tests.

Joint no.	Tested after 1 month		Tested after 4 months	
	Ultimate load (N)	Stiffness (N/mm)	Ultimate load (N)	Stiffness (N/mm)
1	1254	2888	1164	4200
2	1321	1467	1300	4183
3	1095	5612	1418	1257
4	1507	1900	1400	4281
5	1217	1637	1339	2837
6	1339	2503	1357	1187
Mean	1289	2667	1330	2990
St. Dev	126	1404	83	1343

Table 5 - Concrete test results.

Specimen no.	Tested after 1 month		Tested after 4 months	
	Density (kg/m^3)	MOE (MPa)	Density (kg/m^3)	MOE (MPa)
1	2182	13480	2170	15326
2	2222	18893	2207	18897
3	2188	18594	2220	19131
Mean	2197	17000	2200	17784

The results in Table 4 indicate a slight increase in strength and stiffness of the joint with time. For concrete MOE, the limited results also indicate a slight increase in MOE with time.

3.0 PHASE 2 - MODELLING OF COMPOSITE BEAM BEHAVIOUR

In this phase the composite beam (concrete, OSB and I-joist) is modelled as a three-layer beam system with adjacent layers connected by linear spring elements using the ANSYS finite element computer program (ANSYS, 1994). The objective is to use the computer model to study the effects of some construction parameters such as concrete thickness and layer joint stiffness on beam stiffness and first natural frequency. Before the model is used for that purpose, it is necessary to ensure that the model predictions are reasonably accurate. This is done by comparing the model predictions with test measurements.

3.1 Verification of computer model

For the purposes of modelling the composite beam behaviour under static and vibrational loading, each layer is treated as a member with a rectangular cross section. For the concrete and OSB layers the actual dimensions of these layers were used. For the I-joist, an equivalent rectangular cross section was used which gave the same mean flexural rigidity (EI), shear rigidity (G'A) and mass per unit length (m) of the actual I-joists used in the tests. Initial computer analyses indicated that the computer model under-estimated significantly the first natural frequency (f_1) and beam static stiffness (K) for the 9½" deep joist. Upon further evaluation, it was found that the flexural and shear rigidities of those joists measured from the component tests (Table 2) were significantly different from the published values by the supplier of those joists. This was believed to be caused by the use of springs with stiffness which is too high in comparison with the stiffness of the joist during the beam vibration test (Chui, 1991). The method developed by Chui (1991) requires that the beam be suspended with two light springs which simulates a free-free support condition. The use of stiff springs could violate this free-free support assumption and contaminated the test measurements. Another group of 9½" joists, selected for the floor test in Phase 3 and from the same source, were then tested. The test results are shown in Table 6. These results are much closer to the supplier's published values and are therefore used in this phase for modelling purposes. Using the new values, the predicted f_1 and K values are in a much better agreement with test values.

Table 6 - Properties of I-joists used in floor test in Phase 3.

Joist no.	Joist	Mass (kg/m)	EI (x 10^{12} Nmm^2)	G'A (x 10^6 N)
1	9½" JSI-20	3.70	0.72	3.94
2	9½" JSI-20	3.64	0.54	4.94
3	9½" JSI-20	3.62	0.71	3.99
4	9½" JSI-20	3.45	0.52	3.52
5	9½" JSI-20	3.50	0.59	3.72
6	9½" JSI-20	3.39	0.44	6.96
7	9½" JSI-20	3.37	0.49	4.32

Apart from the properties of the three layers, input is also required for stiffness properties of the connections between concrete and OSB and between OSB and I-joist. The connection stiffness (K_1) between I-joist and OSB is assumed to be 900 N/mm per nail for both single- and double-headed nails. This value is close to the value used for deriving the span tables in the 1995 edition of the National Building Code (NRC, 1995) for solid sawn lumber floors. With a nail spacing of 150mm, this gives a connection stiffness of 6 N/mm/mm on a per unit length basis. For the connection stiffness between OSB and concrete (K_2), as the test results indicate, there appears to be a significant composite action between OSB and concrete caused by friction, even without the use of double-headed nails. Therefore it does not seem reasonable to use joint stiffness value from the joint tests. This property is the most suspect of all the input properties into the model. It was decided to estimate this connection stiffness, by a comparison of the model predictions with test results.

To demonstrate the influence of K_2 of concrete-to-OSB connection and shear properties of I-joist on composite beam performance, graphs of K versus G' and f_1 versus G' are given for the composite beams made with the two joist sizes and spans tested in Phase 2, where G' is the shear modulus of the equivalent section representing the I-joist layer in the composite beam. Figures 4 and 5 show the results for the 14" JSI joist while Figures 6 and 7 present results for the 9½" JSI joist. Properties of the components are given in Table 7.

Table 7 - Input properties into computer model.

Case	A (14" JSI-20)			B (9½" JSI-20)		
	Concrete	OSB	"I-joist"	Concrete	OSB	"I-joist"
E (MPa)	18000	6000	10000	18000	6000	10000
G' (MPa)	2000	2000	560[1]	2000	2000	440[1]
Density (kg/m^3)	2200	650	310	2200	650	340
Cross sectional area (mm x mm)	150 x 38	150 x 18	40 x 356	150 x 38	150 x 18	43 x 241
K_1 (N/mm)	900 per 150mm spacing			900 per 150mm spacing		
K_2 (N/mm)	Varied			Varied		
Span (mm)	5600			4178		

[1] The shear modulus is varied in Figures 4 to 7.

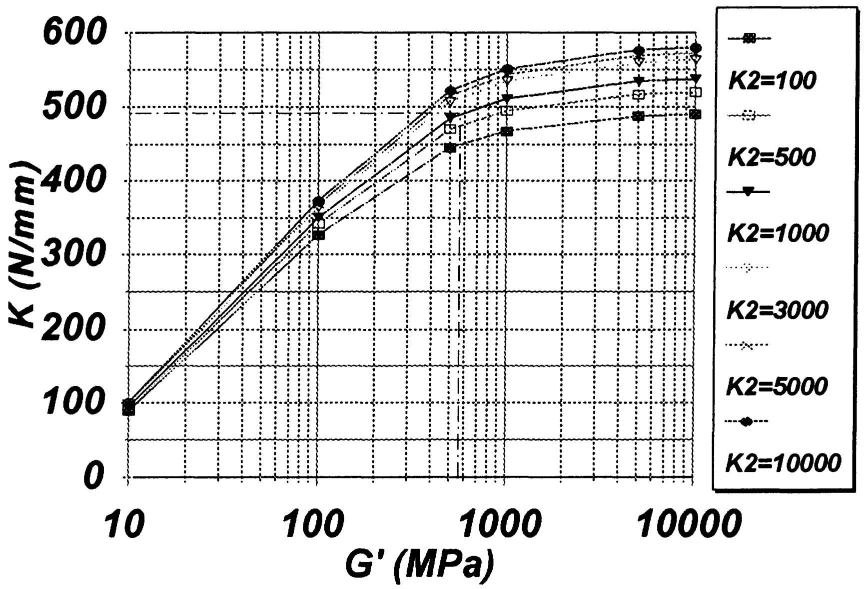

Figure 4 - Variation of K with G' and K_2 (N/mm per 150mm spacing) - Case A (14" JSI20).

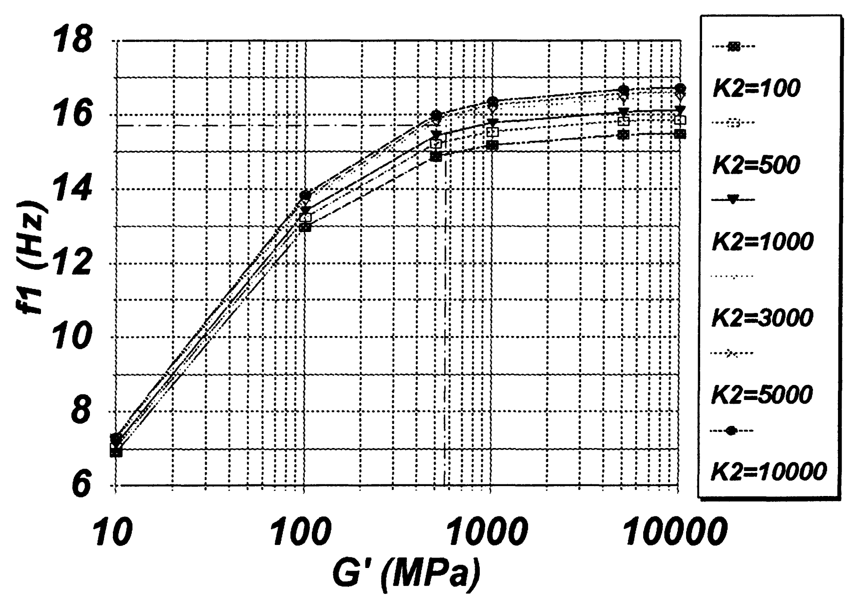

Figure 5 - Variation of f_1 with G' and K_2 (N/mm per 150mm spacing) - Case A (14" JSI20).

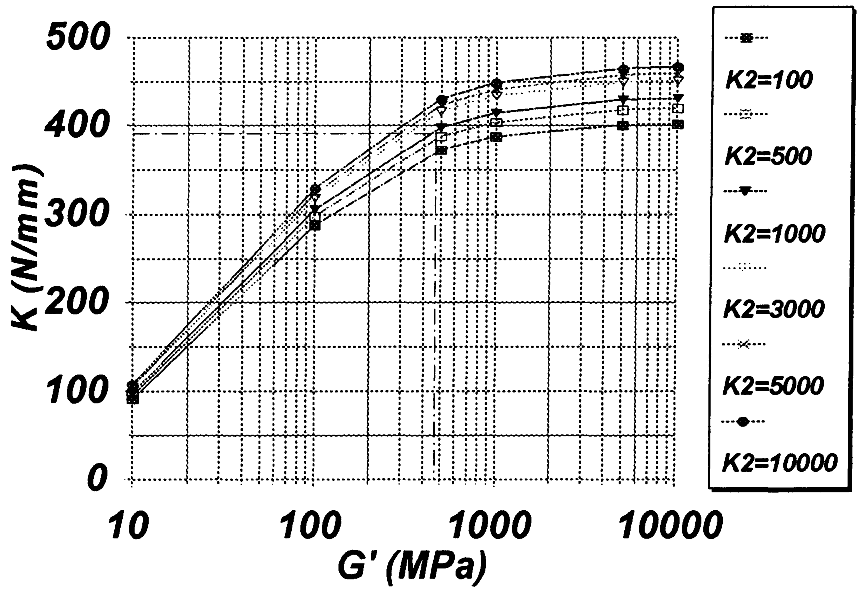

Figure 6 - Variation of K with G' and K_2 (N/mm per 150mm spacing) - Case B (9½" JSI20).

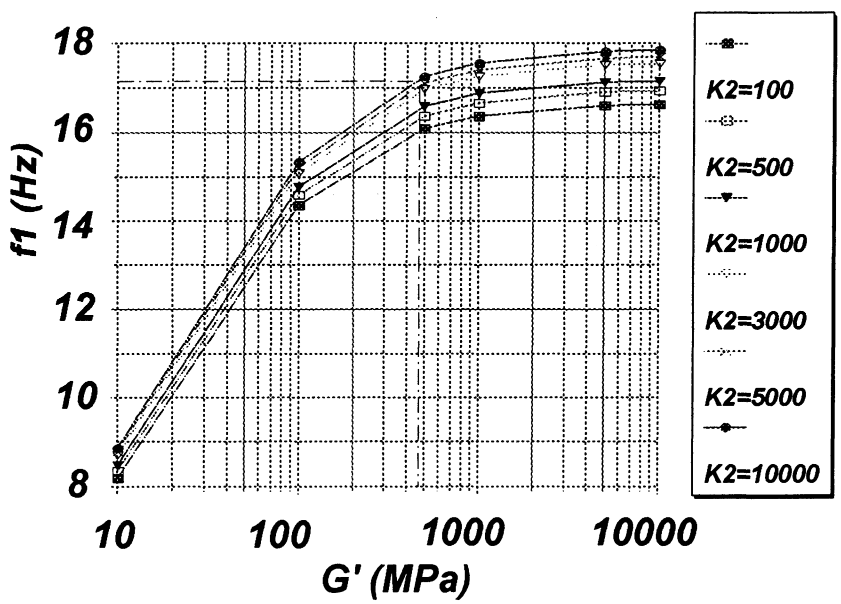

Figure 7 - Variation of f_1 with G' and K_2 (N/mm per 150mm spacing) - Case B (9½" JSI20).

To estimate the connection stiffness between concrete and OSB (K_2), the measured test value is indicated on each graph in Figures 4 to 7 by the intersection of the two dotted lines. It can be seen that all measured K and f_1 are generally within the region in which they are not very sensitive to K_2. Based on these comparisons, it appears that K_2 lies between 1000 and 7000 N/mm per 150mm spacing. This seems to be a large range but the beam behaviour, as mentioned above, is not sensitive to changes in K_2 within this range. For the purposes of modelling the behaviour of composite beams, a value of 3000 N/mm per 150mm spacing is adopted in Phase 2. This value is close to the stiffness of connection made with double-headed nails, Table 4.

It is also noted from Figures 4 to 7 that the stiffness of the beam as predicted from vibration tests (f_1) is higher than that predicted by static deflection tests (K). This difference could well be explained by the presence of residual stresses in the system caused by the shrinkage of various beam components (concrete and OSB). These residual stresses are thought to affect static and vibrational behaviours differently. This topic however requires further study.

3.2 Effects of construction parameters on composite beam behaviour

The effects of connection stiffness on beam stiffness and natural frequency have been demonstrated in Figures 4 to 7. It has been mentioned above that the range of connection stiffnesses encountered in practice, even with the use mechanical shear connectors such as double-headed nails, lie within the range in which beam response is not sensitive to changes in connection stiffness. Thus it seems that the use of shear connectors would not produce any noticeable improvements in the stiffness of the composite beam. Also the figures indicate that the shear deformations of I-joists need to be accounted for when calculating static deflection and vibrational characteristics of floor systems comprising wood I-joists. This is because shear stiffnesses of wood I-joists currently available in the market place appear to lie in the sensitive regions for both K and f_1. This finding is similar to that observed by Hu (1992).

The computer model was further utilized to study the effects of concrete width and thickness on composite beam stiffness and natural frequency. Since the maximum joist spacing of wood floor systems is 600mm (24"), the width of concrete and OSB in the composite beam is varied between 100mm and 600mm. The thickness of concrete is varied from 25mm (1") to 75mm (3"). Other properties are as given in Table 7. As suggested above, the connection stiffnesses are 900 N/mm and 3000 N/mm per 150mm spacing respectively for K_1 and K_2.

The effects of concrete width on K and f_1 are illustrated in Figures 8 and 9 respectively. Results for both Cases A and B are presented. As expected, static stiffness increases with any increase in concrete width. The degree of sensitivity of static stiffness to changes in concrete width appears to be similar for both cases. In both cases the increase in beam stiffness is in the order of 10% when concrete width is increased from 100mm to 600mm with the concrete thickness kept at 38mm (1½"). For the first natural frequency, f_1, the effects of increasing concrete width is to reduce f_1. Figure 9 indicates that f_1 for 600mm width is about half the value for 100mm width. At 600mm width, the frequency is close to the human sensitive frequency range of 4 - 8 Hz (ISO, 1989). Since it has been shown (Hu, 1992) that the first natural frequency of a wood floor system is essentially that of the joist, floor systems comprising I-joists and a concrete topping with wide joist spacing can potentially have natural frequencies close to that human sensitive region.

The corresponding effects of concrete thickness are shown in Figures 10 and 11. It is noted in these figures that the trends are similar to those observed for concrete width. i.e. increasing static beam stiffness and decreasing natural frequency with any increase in concrete thickness.

It can be concluded from the above results that while static stiffness of a floor system can be increased by increasing the quantity of concrete topping, an excessive amount of concrete can cause potential problems in vibrational performance by significantly lowering its natural frequencies. Such a problem cannot be detected at the design stage based on a static deflection limitation alone.

It must be mentioned that all the above results apply to single beam systems. It is recognised that the interest of this study is to generate information for better construction and design of wood floor systems with a concrete topping. The results obtained so far however may not be readily extrapolated to floor systems, since the effects of concrete topping on floor stiffness in the across-joists direction cannot be evaluated in a beam analysis. Also the spans used in the analysis were fixed irrespective of concrete thickness or width. In practice the span would be changed with any increase in concrete quantity to reflect the increase in dead load, and possibly the increase in stiffness. The floor to be tested in Phase 3 is intended to provide some indications as to whether the results obtained from single beam test and analysis can be equally applied to two-way floor systems.

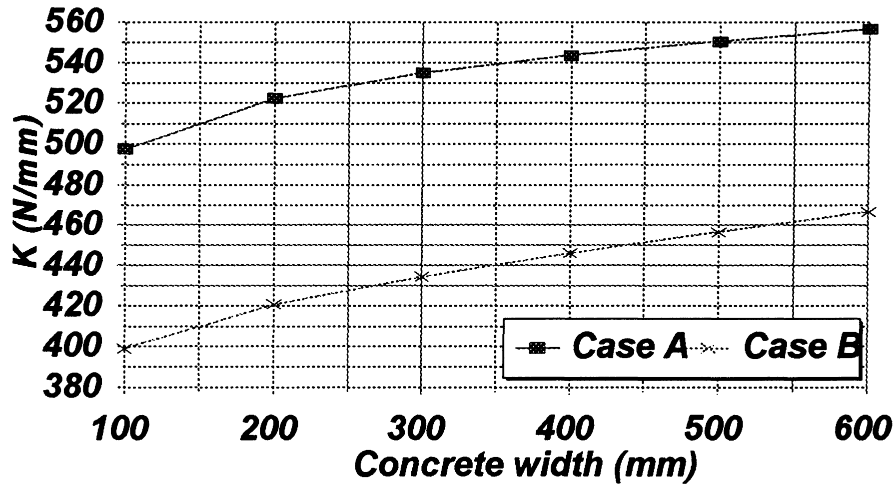

Figure 8 - Variation of K with concrete width (concrete thickness = 38mm).

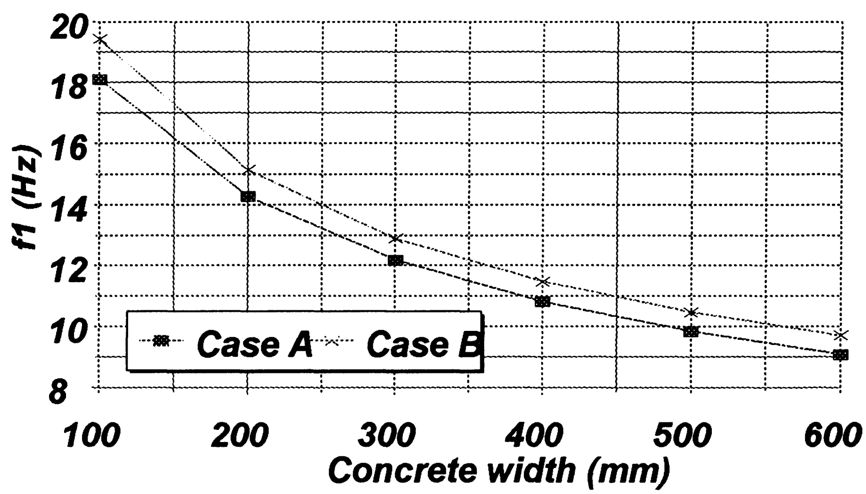

Figure 9 - Variation of f_1 with concrete width (concrete thickness = 38mm).

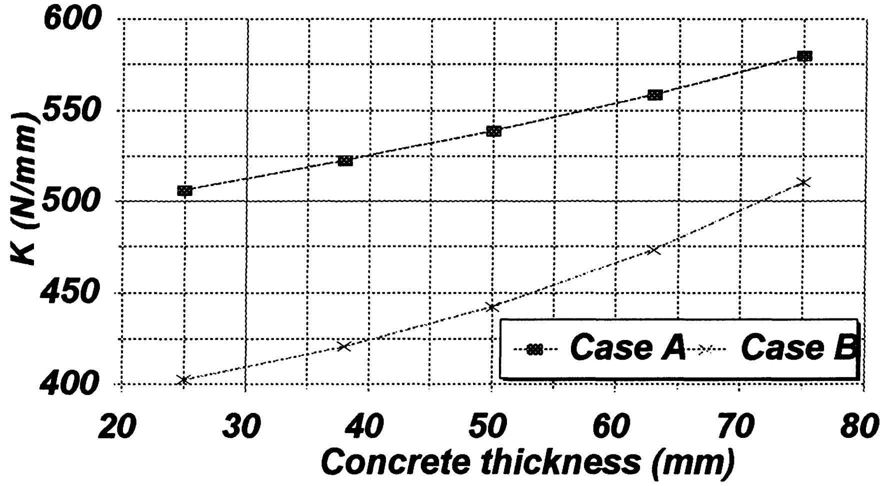

Figure 10 - Variation of K with concrete thickness (concrete width = 300mm).

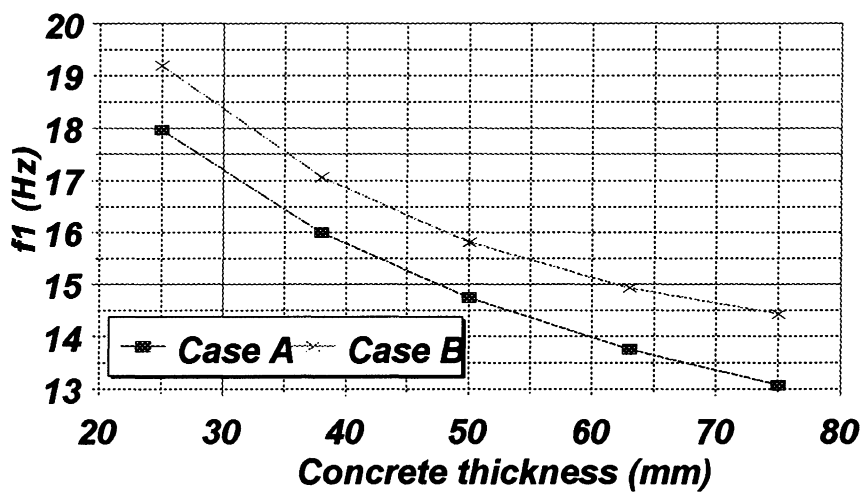

Figure 11 - Variation of f_1 with concrete thickness (concrete width = 300mm).

4.0 PHASE 3 - TESTING OF A FLOOR SYSTEM

In Phases 1 and 2 the behaviour of single composite beams containing a concrete topping, a wood-based sheathing and a wood I-joist was studied. It was found that static stiffness of a beam was less sensitive to changes in concrete topping dimensions (width or thickness) than the first natural frequency was. The reduction in natural frequencies is a potential concern as it was found that for wide joist spacing, the first natural frequencies for the two joist types investigated can approach the human sensitive range of 4 -8 Hz (ISO, 1989). In addition, both static beam stiffness and first natural frequency did not appear to be sensitive to the stiffness of the connections (K_1 and K_2) between the three beam components within the practical range. The objective of this phase is to ascertain if these findings can be extrapolated to two-way floor systems. The other objective is to assess if the lowering of natural frequencies caused by the addition of a concrete topping leads to vibration problems.

4.1 Details of test floor and test procedure

To meet these objectives a floor with a wide joist spacing was selected for investigation in this phase. The chosen floor system meets the manufacturer's design specifications for a floor subjected to a live load of 1.9 kN/m^2 (40 psf) and a dead load of 0.75 kN/m^2 (15 psf), and for a live load deflection limit of span/360. The use of the higher than a normal dead load (10 psf) for residential floor construction was to reflect the presence of concrete topping. Construction details of the floor were: floor span = 4470mm, floor width = 3600mm, joist type = 9½" JSI-20, joist spacing = 600mm, sub-floor = 18.5mm OSB, concrete thickness = 38mm, fastening = 63mm single headed nails at 150mm c/c at panel edges and 300mm c/c at internal supports. Properties of the seven joists used in constructing the test floor are presented in Table 6. Details of the end support condition are simialr to those for the composite beam tests and are illustrated schematically in Figure 1. Each end of a joist was nailed to a 2x4 bearing plate using two 3" screws on either side of the web. The bearing plate was bolted to a steel beam which was in turn bolted to the ground.

The floor was tested prior to, one month after and three months after the placing of concrete. For the purpose of reporting, these three systems are referred to as BARE, CON1 and CON3 respectively. At each stage, three tests were conducted:

1. Static deflection test - A concentrated load was applied at the floor centre using a pneumatic actuator. The applied load was measured using a force transducer. A linear variable displacement transformer (LVDT) was placed at mid-span of each joist to monitor the deflection continuously until a 7 kN load was applied. Deflections were measured relative to the ground.
2. Dynamic test - This test was conducted using an instrumented hammer or shaker to excite the floor into vibration and the response measured using an accelerometer. The signals were recorded and analysed using a spectrum analyser to determine the natural frequencies and damping of the floor system.
3. Heel-drop test - The heel-drop test has been used over the years by a number of researchers including the authors to evaluate response characteristics which have been shown to correlate with human reaction to vibration. In this study the heel-drop was applied at the floor centre, and the response of the floor at various locations was measured using an accelerometer. A heel impact was applied by a person dropping his/her heels through a distance of about 50mm while standing on his/her toes. To ensure consistency, all heel-drop impact tests were performed by the same person who weighs approximately 60kg. The acceleration signals were then analysed using computer software developed by the authors to determine the frequency-weighted root-mean-square (rms) acceleration, a response characteristic proposed by ISO for the purpose of evaluating human response to building vibration (ISO, 1989). Details on the calculation of this parameter have been discussed by Chui (1987).

4.2 Results and discussion

Static test

The mid-span deflection of each joist under a concentrated load at the floor centre at the three stages of construction are presented in Figures 12 to 14 respectively. It can be noted that the addition of a concrete topping reduced the maximum floor deflection from 11mm to 3.25mm after one month of curing of concrete. The deflection characteristics were similar between floors CON1 and CON3. The bare floor results show that only the loaded joist and the two adjacent joists on either side were effective in resisting the applied load. Small upward deflections were detected at the edge joists. This is an indication of the relatively low stiffness of the floor in the direction perpendicular to the span. The presence of a concrete topping caused the load to be shared by a greater number of joists, Figures 13 and 14.

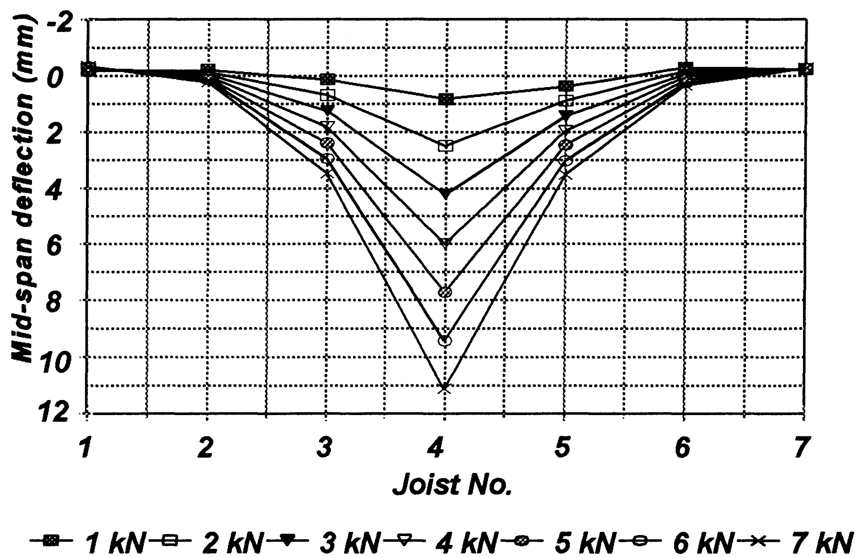

Figure 12 - Deflection characteristics of floor BARE.

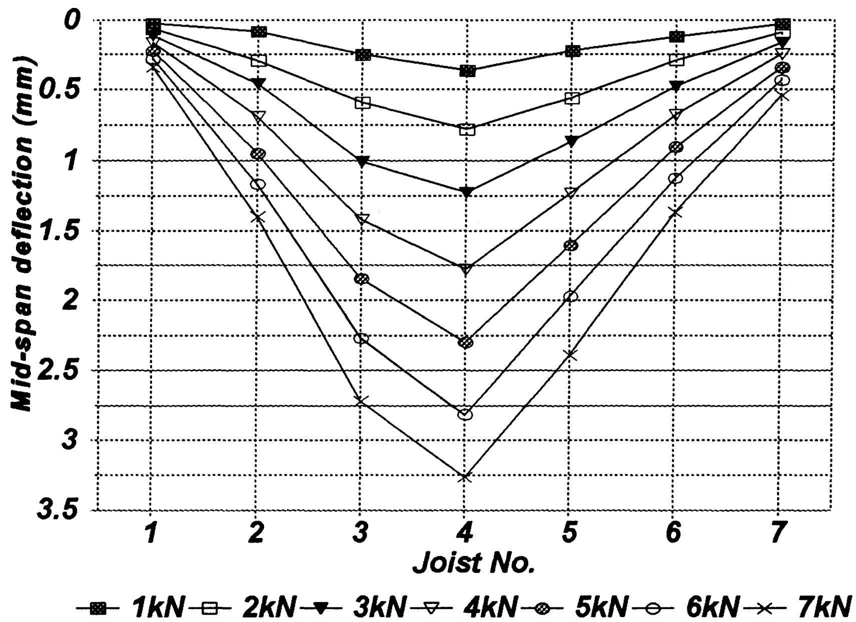

Figure 13 - Deflection characteristics of floor CON1.

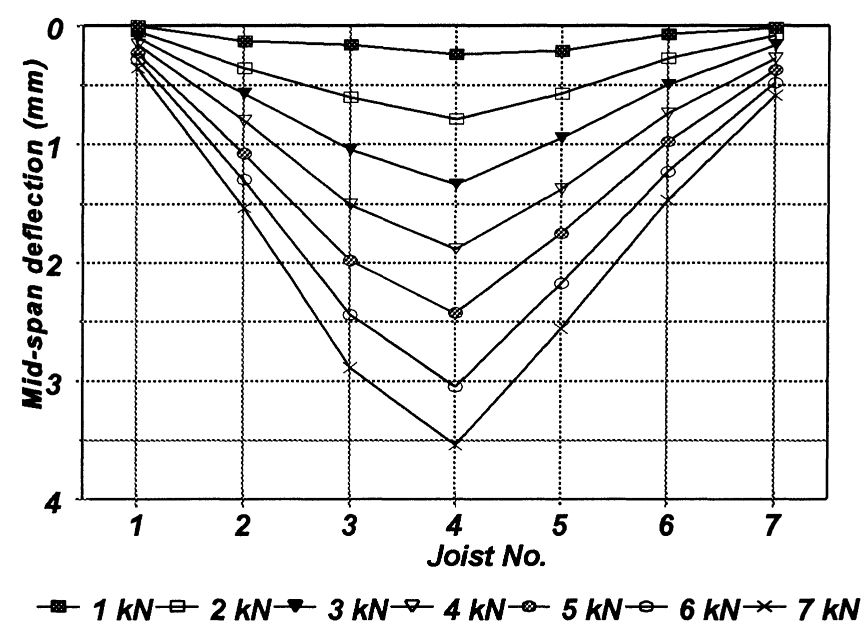

Figure 14 - Deflection characteristics of floor CON3.

A bare wood floor is a so-called highly orthotropic two-way system i.e. its stiffness in the parallel-to-joist direction is considerably higher than that in the perpendicular-to-joist direction. The deflection characteristics are controlled by both floor stiffnesses. In Phase 2 we observed from the numerical study that the stiffness of the composite beam, which controls the floor stiffness parallel to the span, increased by about 20% when a concrete topping of 600mm width was added. However the observed reduction in floor deflection in this phase was about 3½ fold. This indicates that the addition of a concrete topping had a larger influence on floor stiffness in the perpendicular than parallel to the joist direction.

Another change in static behaviour brought about by the addition of a concrete topping was the non-linear load-deflection behaviour, Figure 15. This compares with the clearly linear response of the bare floor, Figure 16.

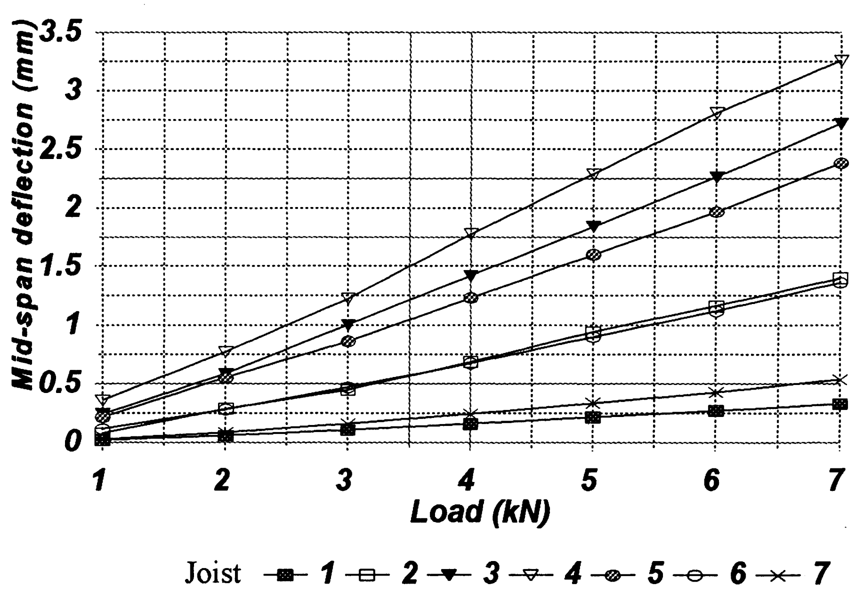

Figure 15 - Load-deflection response of floor CON1.

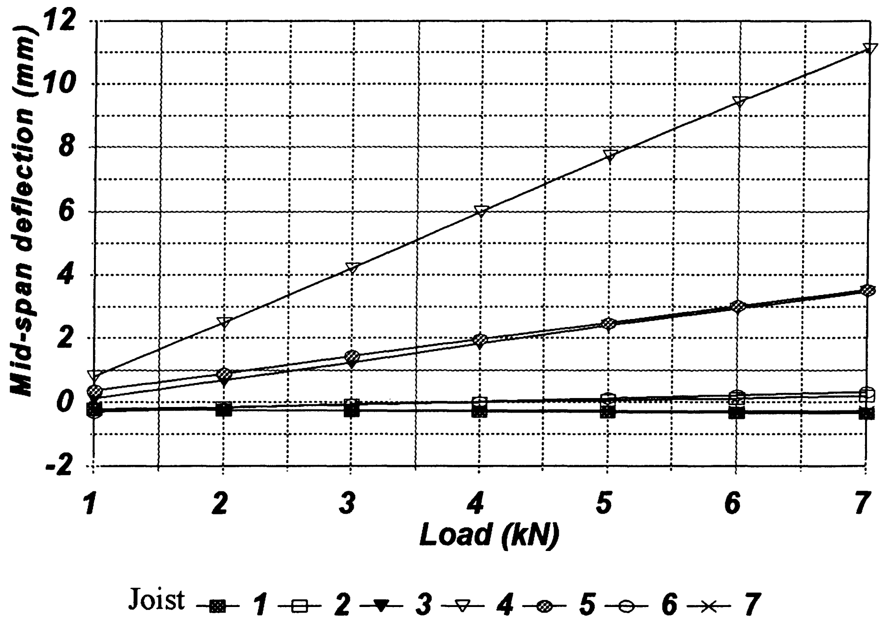

Figure 16 - Load-deflection response of floor BARE.

Dynamic test

The dynamic tests provided information on the natural frequencies and damping of the floor at different stage of construction. Table 8 presents the test results.

Table 8 - Natural frequencies (f) and viscous damping ratios (δ) of test systems.

Floor	Concrete mode		Floor mode 1		Floor mode 2		Floor mode 3		Floor mode 4	
	f (Hz)	δ (%)	f (Hz)	δ (%)	f (Hz)	δ (%)	f (Hz)	δ (%)	f (Hz)	δ (%)
BARE	--	--	18.3	2.4	21.6	2.1	25.7	2.1	29.8	2.4
CON1	8.9	1.86	10.1	3.3	13.6	1.5	19.7	3.9	29.6	1.8
CON3	--	--	11.8	2.6	13.8	1.7	19.7	1.9	31.5	3.5

The column labelled concrete mode in Table 8 relates to the localised vibration mode of the concrete topping itself. This mode was only noticeable in floor CON1. Visual examination of the test floor revealed that shrinkage of concrete caused the topping to deform as a 'dish'. This was evident by the gaps developed at the four corners of the floor between the sub-floor and the concrete topping. These gaps measured approximately 5mm. This dishing effect was not as noticeable after about 3 months i.e. the gaps became smaller. Exact reasons for this reduced dishing effect are not known, but could well be caused by the development of cracks in the concrete with time. One major impact of adding a concrete topping was to lower substantially the first natural frequency. The first natural frequency of a wood floor system is positively related to stiffness, primarily in the span direction, and inversely related to its mass. The addition of a concrete topping therefore has two counteracting effects on the natural frequency because it increases both stiffness and mass. The significant reduction in natural frequency suggests that the mass effect is more dominant.

Concrete can be considered to be an isotropic material which does not display directional variations in properties. When a concrete topping is added to a conventional wood floor system, which is usually highly orthotropic, i.e. the stiffness in the span direction is considerably greater than that in the across joist direction, it leads to a greater percentage increase in the across joist stiffness than in the along joist stiffness. This causes a larger separation between adjacent vibration modes as indicated by the natural frequency values in Table 8. Comparing the results from CON1 and CON3 shows that there appears to be slight increases in natural frequencies with time. This can be caused by the combined effect of the drying out of concrete and the increase in mechanical properties of concrete with time, albeit small after the first month.

Studies on human response to building vibration revealed that humans are most sensitive to vibration having frequency content within the range of 4 to 8 Hz (ISO, 1989). It is therefore desirable to construct building systems which have natural frequencies well above this range. For conventional wood floor systems, this is usually not of concern because of their light weight nature. However, it is shown in this phase that the natural frequencies of wood floor systems with a concrete topping can be close to this human sensitive range. This problem may be aggravated by the dishing of the concrete plate which causes localised sub-structure vibration mode with natural frequencies even lower than those of the floor system itself.

Heel-drop impact test

The heel-drop impact tests have been used by the authors for comparing performance of wood floor systems. This test has certain limitations such as the low degree of repeatability in impact characteristics, even if applied by the same person. However, experience of the authors shows that while the impact characteristics vary, the variations in rms acceleration are generally small if the impacts are applied by the same person. This variation decreases as the distance between response measurement and impact locations increases. The popularity of this test stems from the fact that the characteristics of the impact reflect those found under normal service conditions, which are difficult to be reproduced using mechanical devices. Therefore despite its limitations, it remains one of the favourite tests used by engineers in assessing vibration performance of floors.

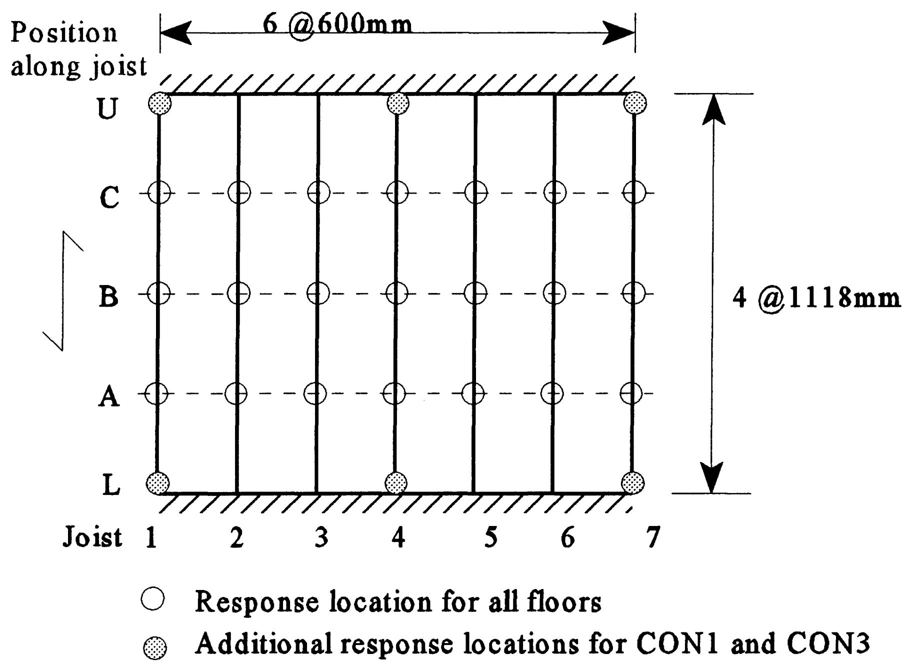

Figure 17 - Response measurement locations in heel-drop impact test.

In this phase the heel-drop impacts were always applied at the floor centre. Responses of the floor at various locations were recorded. Normally, the responses near the supported floor edges and joist ends are not measured because they are usually negligible. In this case, however, because of the 'dishing' of the concrete measurements were also taken at the four corners of the CON1 and CON3 systems and at selected locations near the support of the centre joist. The measurement locations are as denoted in Figure 17. At each location three response signals were recorded. These were caused by three separate heel impacts applied at the floor centre. The mean frequency-weighted rms acceleration at each location is presented in Tables 9, 10 and 11 for the three systems respectively. It can be noted that in general the rms acceleration recorded at the interior joists were much lower in systems CON1 and CON3 compared with system BARE. The vibration levels recorded at the edge joists were higher due to the dishing of the concrete topping. Relatively large vibration magnitudes were recorded close to the impact location (B4) in the bare floor. The response magnitude decreased gradually moving away from the impact location. The addition of the concrete topping significantly reduced the rms acceleration at the floor centre (Table 9 vs Table 10). It was also noted that the rms accelerations measured on the loaded joist were similar to those of the adjacent joists, indicating that the impact force was shared more evenly between the joists. The response magnitudes were much more uniform in the floor with concrete topping compared with the bare floor. This is supported by the static test results.

Another major difference between the bare floor and the floor with a concrete topping is the opposite trends observed in rms acceleration moving from the floor centre to the floor edge. For the bare floor rms acceleration always decreased from floor centre towards the edge. Because of the dishing effect in the concrete, rms acceleration values of the floor with concrete topping show an increasing trend or a slight decrease and then an increasing trend when the same locations are compared from floor centre towards the edge. In fact, as evident from Tables 10 and 11, the response magnitudes measured at the edge joists of the floor with concrete topping are higher than the corresponding values at the centre joist. For conventional floor systems, vibration levels near the joist support are usually low. In the floor tested here however, because of the dishing of the concrete towards the floor centre, the measured rms acceleration values near the joist supports were quite high. For the floor tested after 1 month, the rms acceleration values at the floor corners were even higher than those measured at the impact location. This difference was not noted after the concrete has been dried for three months. At the 3-month tests, while the magnitudes were still relatively high, the corner measurements were lower than the centre measurement. This can be attributed to the lower degree of dishing observed after three month.

Comparing results in Tables 10 and 11 it can be noted that on average the CON3 rms acceleration is higher than the CON1 results. This may be caused by the reduction in the mass of the system, as indicated by the increase in the first natural frequency, due to drying out of concrete.

Table 9 - Frequency-weighted rms acceleration (m/s^2) for floor BARE.

Position	Joist						
	1	2	3	4	5	6	7
U							
C	0.344	0.372	0.442	0.742	0.470	0.407	0.385
B	0.419	0.492	0.630	1.080	0.687	0.579	0.507
A	0.421	0.351	0.461	0.767	0.472	0.409	0.451
L							

Table 10 - Frequency-weighted rms acceleration (m/s^2) for floor CON1.

Position	Joist						
	1	2	3	4	5	6	7
U	0.594			0.244			0.397
C	0.575	0.329	0.285	0.289	0.258	0.234	0.362
B	0.575	0.413	0.388	0.333	0.302	0.263	0.371
A	0.541	0.342	0.356	0.302	0.258	0.236	0.377
L	0.621			0.282			0.436

Table 11 - Frequency-weighted rms acceleration (m/s^2) for floor CON3.

Position	Joist						
	1	2	3	4	5	6	7
U	0.257			0.216			0.225
C	0.416	0.366	0.386	0.404	0.433	0.399	0.465
B	0.562	0.395	0.505	0.500	0.492	0.495	0.547
A	0.429	0.366	0.409	0.370	0.385	0.522	0.524
L	0.145			0.224			0.144

In addition to the quantitative information recorded from the floor test i.e. frequency-weighted rms acceleration, the three systems were also qualitatively assessed by laboratory technicians and the authors. It was felt that the vibration was not particularly perceptible in the BARE floor. However discernable movement was noted in systems CON1 and CON3 especially near the floor edge. This can be explained in part by the low frequency of vibration in CON1 and CON3. Figure 18 presents a comparison between responses measured at location B1 in system BARE and CON1. A corresponding comparison for location B4 is presented in Figure 19.

Figure 18 shows that except with the initial peaks the general magnitudes of vibration for the two system at the edge are similar. However the frequency of vibration is much lower for CON1. This caused the frequency-weighted rms acceleration to be higher since the higher weighting is towards the low frequency end. At the floor centre B4 near the impact location (Figure 19), the magnitude of the vibration is significantly lower in CON1 because of the higher load sharing provided by the concrete topping.

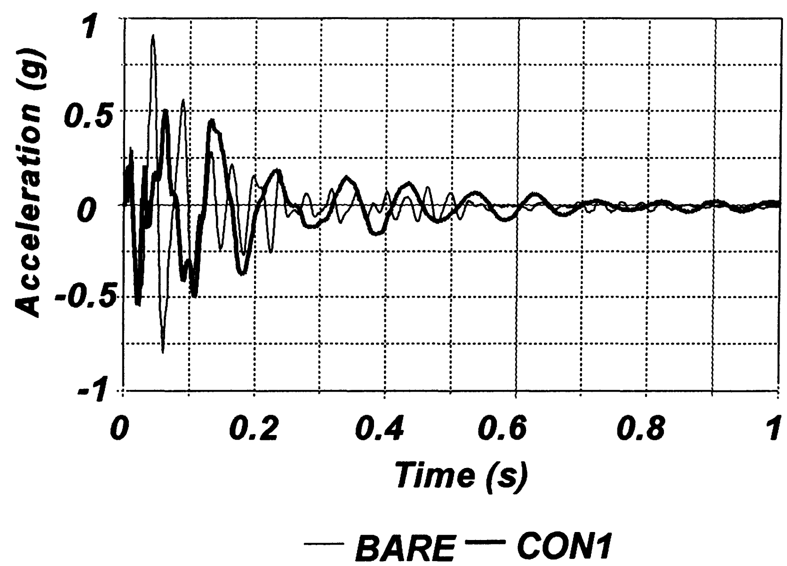

Figure 18 - Acceleration signals measured at location B1.

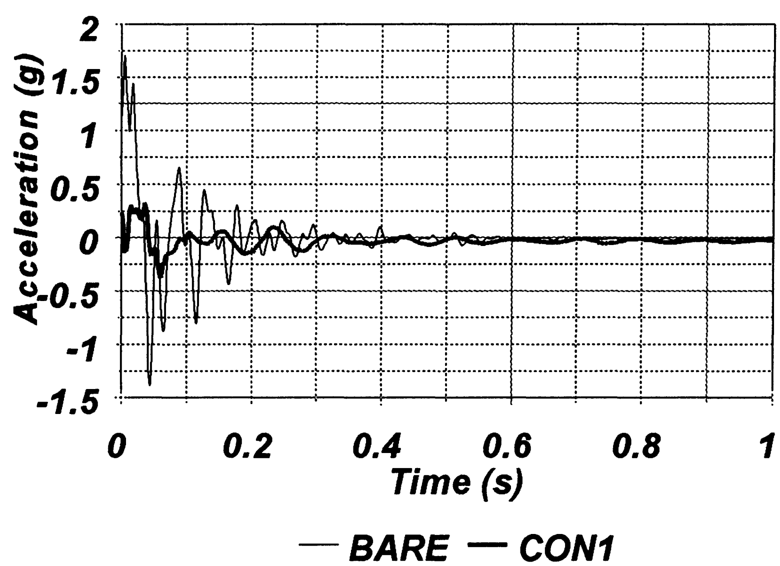

Figure 19 - Acceleration signals measured at location B4.

For controlling vibration in floors built with solid lumber joists, Part 9 of the National Building Code (NRC, 1995) provides a static deflection check under a 1 kN load applied the floor centre.

The deflection limit (Δ) in millimetre under a 1 kN load is related to the floor span (L) in metre by the equation

$$\Delta = \frac{8.0}{L^{1.3}} \qquad \textbf{(1)}$$

For the floor span used here (L=4.47m), Δ=1.14mm. The measured static deflections at 1 kN load for BARE and CON3 floors were 0.83mm and 0.36mm respectively. Thus under the National Building Code Part 9 criterion, both systems would be classified as having acceptable vibrational performance. However based on the assessment by the authors and the laboratory technicians, only the BARE floor performance agrees with the Part 9 classification. This raises the question as to the applicability of using the Part 9 criterion to assess vibrational performance of floors with a concrete topping.

4.3 Bonding between concrete and sub-floor material

The beam tests in Phase 2 revealed that there was no bonding between the OSB sheathing and concrete. Since plywood is also commonly used in floor construction, it is of interest to find out if the same result applies also to plywood. In addition it has been suggested that pre-wetting of the sheathing surface may have produced a different result. In this phase four small sheathing/ concrete specimens were prepared: dry plywood/concrete, pre-wet plywood/concrete, dry OSB/concrete and pre-wet OSB/concrete. OSB sheathing was the same product used in beam and floor tests reported in this report. Plywood was Canadian Softwood Plywood (CSP). Each specimen was first prepared by cutting a 600mm x 600mm piece of sheathing from a full-size panel. The piece was then lined with lumber at the edges which allowed a 38mm thick concrete topping to be poured and cured. Small specimens were used here as the intention was to provide an indication of the presence of bonding between sub-floor and concrete. A much larger test area was needed if the interest was to study the influence of shrinkage characteristics of concrete on the interaction between sub-floor and concrete.

The specimens were then left in the laboratory for three months before the concrete and the sheathing were separated. It was observed that no bonding occurred at all for both OSB specimens. Limited bonding, in small patches, was observed between plywood and concrete for both plywood specimens. These observations suggest that the degree of composite action between OSB sub-floor and concrete is lower than that between plywood sub-floor and concrete.

4.4 Comparison of beam model prediction with actual system behaviour

Most design calculations for wood floor systems are performed based on the behaviour of single beam behaviour. It is known that this approach errs on the conservative side when calculating deflection since it ignores the two-way action of a floor. With the test data obtained in this study, it is possible to compare the results of the single beam approach to the actual measured response. The calculations were performed for a single composite beam using the same ANSYS program (ANSYS, 1994) in Phase 2. Properties of the composite beam are the same as for Case B in Table 7, except that the width of the concrete and OSB layers, the span and K_2 were changed to 600mm, 4770mm and 3000 N/mm per 150mm fastener spacing respectively. ANSYS was used to calculate the first natural frequency and the deflection of the beam under a 1 kN point load at mid-span for the beam with and without concrete topping.

A comparison of the calculated beam response and the measured floor response is presented in Table 12. As can be seen in Table 12, the single beam approach overestimates the deflection by 650% and 285% compared with the actual system response for the cases with and without concrete topping respectively. In contrast the single beam approach provides a fairly accurate estimate of the first natural frequency in each case. This can be explained by the fact that while the floor deflection is dependent on both stiffnesses in the along-joist and across-joist directions, the first natural frequency is a function primarily of the along-joist (i.e. beam) stiffness.

Table 12 - A comparison of single beam prediction with floor system response.

Property	Without concrete		With concrete topping	
	Beam prediction	Measured floor response	Beam prediction	Measured floor response
f_1 (Hz)	18.50	18.30	8.30	10.10
Deflection (mm)	3.20	0.83	2.70	0.36

5.0 CONCLUSIONS

1. Within the practical range of connection stiffness, the behaviour of floor systems with concrete topping is not sensitive to changes in connection stiffness. Despite the lack of bonding between OSB sub-floor and concrete, the friction between the two components appears able to produce composite action similar to that achievable by the use of light gauge connectors such as double-headed nails.
2. When a concrete topping is added to a floor system, it leads to an increase in system mass and stiffness. These increases have counteracting effects on the first natural frequency. The results obtained here show that the mass effect is more dominant, and in most cases the result will be a reduction in the first natural frequency. The reduced frequency may fall close to the human sensitive range of 4 - 8 Hz, especially when the joist spacing is wide.
3. The addition of a concrete topping causes a significantly greater degree of load sharing to occur among the joists of a floor system, and hence a considerable reduction in maximum deflection under the same loading condition.
4. 'Dishing' of concrete topping may occur after drying out which leads to a relatively high level of localised vibration near the floor edge when excited dynamically.
5. There is no bonding between OSB sub-floor and concrete. However limited bonding occurs between plywood sub-floor and concrete.
6. Composite beam calculation models can predict the first natural frequency well, but not deflection characteristics of floor systems with concrete topping, based on a single beam

analogue.

6.0 RECOMMENDATIONS FOR DESIGNING AND CONSTRUCTING WOOD FLOOR SYSTEMS WITH CONCRETE TOPPING

This project has been conducted to provide some initial indications of the serviceability behaviour of wood floor system with a concrete topping. The work was limited in nature as only one full-size floor system was tested, although additional understanding was achieved by beam tests and numerical modelling of the beam behaviour. While further work seems necessary to address certain issues such as influence of concrete characteristics on floor behaviour, development of an appropriate design approach to properly account for the contribution of concrete topping to floor system stiffness and long term creep behaviour, the following construction and design recommendations are given based on the results from this project:

1. Although the use of light mechanical fasteners such as double-headed nails in connecting sub-floor to concrete does not lead to any appreciable increase in floor stiffness, its use may help reduce the amount of dishing in concrete after drying out.
2. Floors with concrete topping can have first natural frequencies close to the human sensitive range of 4 - 8 Hz. This is especially true for systems with wide joist spacing or thick concrete topping. Design calculation should be performed to check and ensure that this does not happen. To achieve this, the floor can be treated as a series of composite beams and simple dynamic beam formulas from standard text can be used to calculate the first natural frequency. The use of these formulas requires the mass and stiffness properties of the composite beam as input. Calculation of stiffness property of a composite beam is usually not straight forward, but the simple layer beam model proposed by McCutheon (1986) can be used to estimate the effective beam stiffness of a three layer beam.
3. The addition of a 38mm thick normal concrete topping adds about 0.7 - 0.8 kN/m^2 to the dead load on a floor. In cases where the total load deflection is not checked this additional dead load may not cause any reduction in the allowable span if the design is governed by live load deflection. When concrete topping is present, it is recommended that the total

load deflection be checked and that a proper two-way calculation model be used, in order to take advantage of the substantial increase in floor stiffness in the across joist direction.

7.0 ACKNOWLEDGEMENTS

This project is funded with a grant from the Canada Mortgage and Housing Corporation under the External Research Program. The wood I-joists tested in this study were provided by Jager Industries Inc.

8.0 REFERENCES

ANSYS. 1994. Swanson Analysis Systems, Inc., Houston, Pennsylvania.

Chui, Y. H. 1987. Vibrational performance of wooden floors in domestic dwellings. PhD Thesis, Brighton Polytechnic, U.K.

Chui, Y.H. 1991. Simultaneous evaluation of bending and shear moduli of wood and the influence of knots on these parameters. Wood Science and Technology, 25:125-134

Ewins, D.J. 1986. Modal testing : Theory and practice. Research Studies Press, Letchworth, UK.

Hu, L.J. 1992. Prediction of vibration responses ribbed-plates by modal synthesis. PhD thesis, University of New Brunswick, Fredericton, N.B.

ISO. 1989. Evaluation of human exposure to whole-body vibration - Part 2: Continuous and shock-induced vibration in buildings (1 to 80 Hz). ISO 2631-2, International Organization for Standardization, Geneva, Switzerland.

McCutheon, W.J. 1986. Stiffness of framing members with partial composite action. J. of Structural Eng., 112(7):1623-1637

NRC. 1995. National Building Code. National Research Council Canada, Ottawa, ON.