Revised

FINAL REPORT

USE OF CONVENTIONAL CROSS BRIDGING IN WOOD I-JOIST FLOORS

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EXECUTIVE SUMMARY

The use of cross bridging in improving performance and load-carrying capacities of conventional sawn lumber joist floors is well known. With the rapid substitution of solid sawn joists by prefabricated wood I-joists and the tendency of builders to adopt conventional construction techniques for this relatively new product, there is need to evaluate if the same type of bridging can be used equally effectively for floors built with wood I-joists. This study was conducted to answer this question.

A total of eight full-size floor systems were tested in this study for their deflection and vibrational characteristics. In addition, four narrow floor specimens were tested to evaluate the impact of omitted bridging members on failure load and mode of floors. These test systems were built with 302mm (11⅞") deep wood I-joists and an oriented strandboard floor sheathing. Construction details followed largely National Building Code requirements. In addition to the major objective of finding out if cross bridging can be similarly adopted for floors with wood I-joists, the experimental program was designed to evaluate the influence of parameters on the effectiveness of cross bridging. These parameters included joist spacing, number of rows of bridging and lumber strapping.

It was found that deflection and vibration levels decreased with the addition of bridging. Bridging was more effective in floors with close joist spacing than wide joist spacing. Increasing the number of rows of bridging from one to two (equally spaced) led to a large reduction in floor deflection but only a relatively small reduction in vibration levels. A large synergistic influence was achieved when a bottom lumber strapping was used in conjunction with cross bridging. This effect was particularly profound for systems with wide joist spacing. The current requirement of using two 57mm (2¼") nails at each end of a cross bridging member was found to be adequate if properly installed. Any weak fastening or omitted bridging member may lead to a large reduction in load-carrying capacities of floors, compared with properly fastened systems. The use of bottom strapping in conjunction with cross bridging, can however minimize this reduction.

RÉSUMÉ

L'utilisation des croix de Saint-André pour améliorer la performance et le pouvoir porteur des solives de plancher traditionnelles en bois scié est bien connue. Mais l'adoption rapide des poutrelles en I préfabriquées en bois, en remplacement des solives en bois scié massif, et la tendance des constructeurs à recourir à des techniques classiques pour ce produit relativement nouveau incitent à évaluer si le même type d'entretoise peut être utilisé tout aussi efficacement pour les planchers réalisés avec des poutrelles en I. Cette étude avait pour objectif de répondre à cette question.

En tout, huit planchers en vraie grandeur ont été mis à l'essai afin de déterminer leurs caractéristiques au chapitre du fléchissement et des vibrations. De plus, quatre spécimens de planchers étroits ont été étudiés afin de déterminer quelle incidence pouvait avoir l'omission des croix de Saint-André sur la charge extrême et le mode de défaillance des planchers. Ces assemblages destinés aux essais ont été construits avec des poutrelles en I en bois de 302 mm (11 7/8 po) de hauteur et un support de revêtement de sol en panneaux de copeaux orientés. Les détails de construction étaient, dans une large mesure, conformes aux exigences du Code national du bâtiment. En plus de l'objectif principal consistant à déterminer si les croix de Saint-André pouvaient tout aussi bien être adoptées pour des planchers réalisés avec des poutrelles en I en bois, le programme expérimental a été conçu pour évaluer l'influence de certains paramètres sur l'efficacité des croix de Saint-André, c'est-à-dire l'espacement des poutrelles de même que le nombre de rangées de croix de Saint-André et de lattes de bois continues.

On a découvert que les niveaux de fléchissement et de vibration diminuaient avec l'ajout de croix de Saint-André. Ces entretoises étaient plus efficaces pour les planchers dont les poutrelles étaient rapprochées que pour celles qui étaient plus espacées. Lorsqu'on a fait passer le nombre de rangées de croix de Saint-André de une à deux (également espacées), on a obtenu une importante réduction du fléchissement du plancher, mais une réduction relativement faible des vibrations. Un effet synergique important a été rendu possible par l'ajout de lattes continues à la sous-face des poutrelles de concert avec les croix de Saint-André. Cet effet s'est avéré particulièrement évident pour les poutrelles plus espacées. L'exigence actuelle qui consiste à utiliser deux clous de 57 mm (2¼ po) à chaque extrémité des pièces constituant les croix de Saint-André s'est révélée appropriée si la pose était correctement effectuée. Tout défaut de fixation ou toute pièce d'entretoise omise peut entraîner une importante réduction du pouvoir porteur d'un plancher, comparativement aux planchers convenablement assemblés. L'utilisation de lattes continues en sous-face des poutrelles en plus des croix de Saint-André peut toutefois limiter cette baisse de capacité.

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CONTENT

1.0 INTRODUCTION

To increase the stiffness and strength of a floor system a commonly used method is between-joist cross bridging. This method is effective because it distributes the load over a larger number of joists in the floor system. This distribution or load-sharing increases the stiffness, load-carrying capacity, and also reduces vibration levels under footfall impacts. The National Building Code of Canada (NRC, 1995) allows for greater spans of solid sawn lumber joists because of the reduction in vibration with the bridging in place.

With the increasing demand on the wood supply, the scarcity of large round wood has created the need for an alternative wood product for use in floor systems. Engineered wood products, such as prefabricated wood I-joists, have been designed to become the alternative. It has been estimated that by the year 2000 nearly half of the new floor systems in North America will be built with wood I-joists. Some manufacturers of I-joists recommend that cross bridging be used to enhance the floor system performance. Presently, it is unclear if the same bridging details for solid lumber joists can be used for prefabricated wood I-joists.

Even though cross bridging is commonly used in the majority of conventional floor systems, it has not been the topic of many research projects. One of the earliest studies in Canada was conducted by Onysko and Jessome (1973) in which they studied the effectiveness of the various bridging types of solid sawn lumber in floors under static loading conditions. They concluded from their investigation that the effectiveness of cross bridging was directly dependant on the initial joist stiffness and the method of fastening the bridging to the joists. Any increase in joist stiffness decreases the effectiveness of the bridging. In a study on floor systems built with sawn lumber joists, Chui and Smith (1991) evaluated the influence of bridging between joists on vibration, they found a reduction in vibration levels with a center row of bridging. Recent work by Hu (1998) has led to the speculation that vibration in floor systems may not always be reduced with a centre-line bridging, but may be more dependant on the bridging location with respect to the impact load.

The research reported here was designed to provide information on whether cross bridging in wood I-joist floor systems is equally effective compared with solid sawn lumber floors. It is hoped the information obtained through testing of the various floor and bridging configurations will provide a better understanding of how the bridging and wood I-joists react under loading. From the current research literature it is apparent that the effectiveness of any bridging in floor systems is dependant on the method of fastening bridging to the floor joists. The method of fastening cross bridging to solid lumber joists may not be easily adopted for wood I-joist floor systems. Therefore, one aspect of this research project is to address the fastening details of bridging for wood I-joist floor systems.

Other objectives of the study are: 1. To evaluate the influence of number of rows of bridging, presence of bottom strapping and the joist spacing on the static deflection, vibration performance and failure behaviour of floor systems; 2. To evaluate the influence of missing bridging members on failure behaviour of floors.

2.0 TEST FLOORS

This project was conducted in two phases. The first phase involved testing of three floor systems. Three base floors were constructed and tested. Each base floor was modified at least once and retested to investigate the effects of bridging details on both static and vibration behaviour. All floors had identical floor span, number of joists and floor components. The joists used were 300mm (11 ⅞") deep NJH12 wood I-joists manufactured by Nascor Inc. All joists were individually tested to determine the mechanical properties (i.e. stiffness). Selection of joists for use in constructing the three base floors was based on these stiffness properties and was done to ensure that the joists used for the three systems had similar properties. The NJH 12 series joists consist of 38 x 64mm (2 x 3") lumber flanges with a 9.5mm (⅜") thick oriented strandboard (OSB) web. The floors had a span of 4800mm (15' - 9") and contained seven joists at 610mm (24") centres for the 3660mm (12') wide floor or 406mm (16") centres for the 2440mm (8') wide floor. The joists were attached to a 38 x 89mm (2 x 4") bearing plate with two76mm (3") screws on each end of the joist. The bearing plate was fastened to a 900mm (35½") high knee wall with screws. The knee walls were built with 38 x 140mm (2 x 6") lumber spaced at 406mm (16") and was braced to a metal frame. Figure 1 details the construction of the floor attachment to the knee wall.

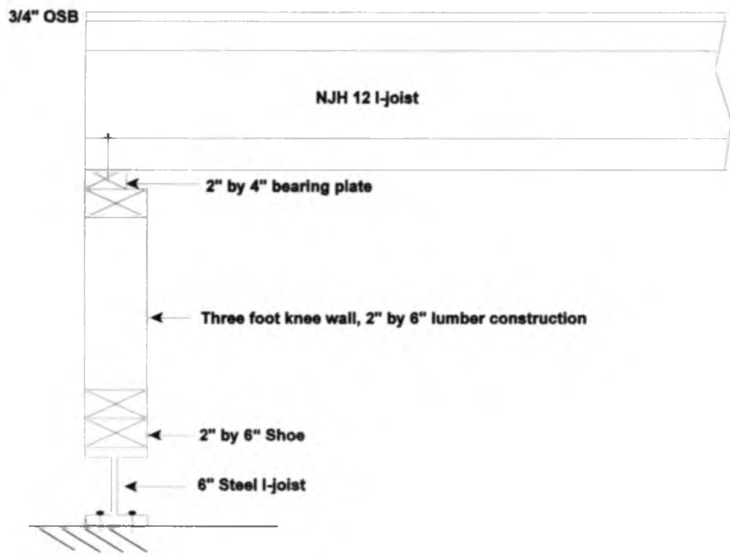

Figure 1 - I-joist support details.

The flooring material used was sheathing grade 18.5mm (¾") thick OSB panels. The OSB was attached to the I-joists using 51mm (2") thread fast screws. The spacing of the screws on the outside perimeter and the inside seams were 152mm (6"), with the rest having a 305mm (12") spacing.

For each of the three base floor systems (floors 1, 2 and 4), the base floor itself was first tested. Various bridging configurations were then introduced and the floor retested. In base floors 1 and 4 two modified systems were tested whereas in base floor 2 one was tested. Bridging details of each floor system tested are provided below:

Floor 1A - 610mm (24") joist spacing, base floor, no cross bridging
Floor 1B - 610mm (24") joist spacing, two rows of cross bridging, each placed at the third point of the span
Floor 1C - 610mm (24") joist spacing, one row of cross bridging at mid-span
Floor 2A - 610mm (24") joist spacing, base floor, no cross bridging / strapping
Floor 2B - 610mm (24") joist spacing, one row of cross bridging / strapping at mid-span
Floor 4A - 406mm (16") joist spacing, base floor, no bridging
Floor 4B - 406mm (16") joist spacing, one row of cross bridging at mid-span
Floor 4C - 406mm (16") joist spacing, one row of cross bridging / strapping at mid-span

In phase 2, four floor systems with a span of 4800mm (15' 9") consisting of three joists spaced at 610mm (24") were constructed and tested. These were tested to evaluate the influence of missing cross bridging members (i.e. poor workmanship) on the failure behaviour of floor systems. These floors were numbered 3A1, 3A2, 3B1 and 3B2 respectively. Figure 2 illustrates the attachment of the bridging and strapping for each system.

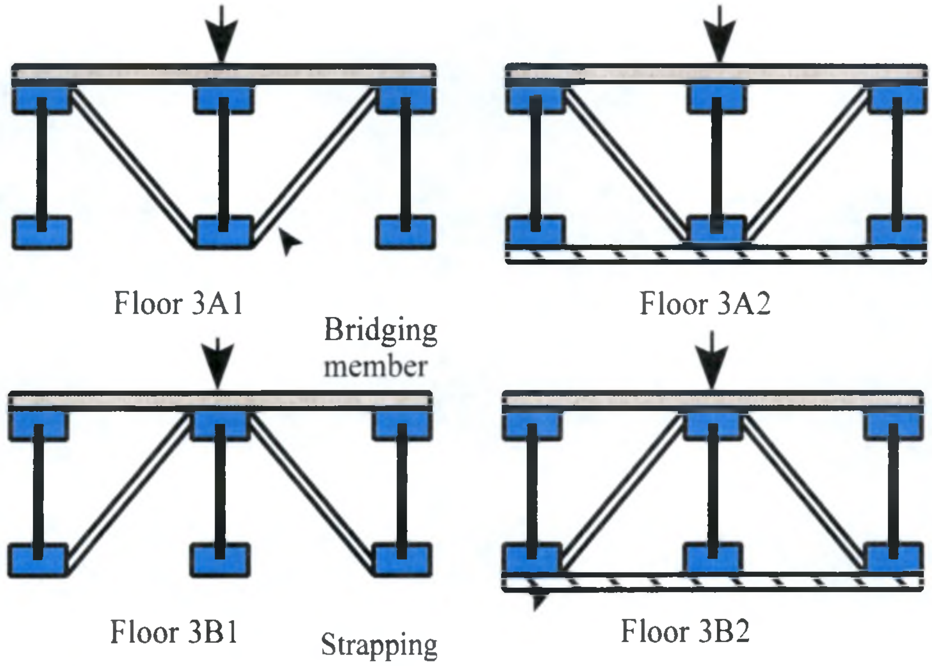

Figure 2 - Bridging and strapping details of floors in phase 2.

It should be mentioned that systems 4A, 4B and 4C were part of a separate study funded by Forintek Canada Corp. but are included in this report for the purpose of discussing the effects of joist spacing on cross bridging effectiveness.

The cross bridging used in the test floor systems was made up of green 38 x 38 mm (2 x 2") dressed lumber to simulate in-field situations. Testing was typically completed within one week of fastening bridging to a floor. The bridging was attached to the I-joists using two 57mm (2¼") spiral nails on each end. Nails were driven by a hammer horizontally into the I-joist flanges. The strapping used in Floor 2B and 4C was 25 x 102mm (1 x 4") green lumber. The strapping was attached to the underside of each joist using two 57mm (2¼") spiral nails.

All I-joists were tested using the vibration technique developed by Chui (1991) to determine their flexural and shear stiffness. As indicated above, selection of joists was to ensure that the mean flexural stiffness and mass per unit length were similar for all systems tested. Joist properties for each system are presented in Appendix A (Tables A1 to A7). Five OSB panels were tested using the vibration technique by Sobue and Katoh (1990) to determine their bending modulus and shear modulus. Results are shown in Table A8. Two 25 x 102mm (1 x 4") strapping members were tested in static bending to determine their moduli of elasticity. These are given Table A9.

For ease of presenting and discussing the test results, a coordinate system illustrated in Figure 3 is adopted here to signify the loading and response locations.

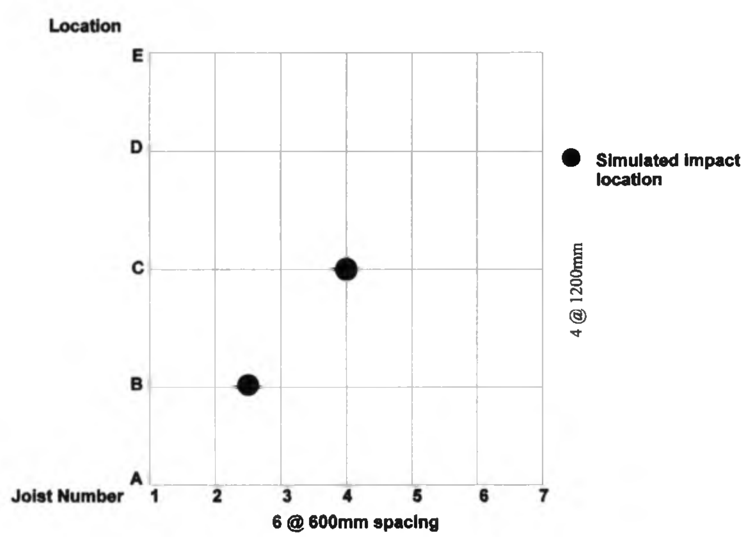

Figure 3 - Coordinate system.

3.0 TEST METHODS

For floors in phase 1, three types of test were conducted on each system : static deflection test, hammer impact test and simulated footfall impact test. In addition, floor systems 1C and 2B were tested to failure using the concentrated load test method. Details of each type of test are described below. Immediately after the construction of each floor system, it was loaded with a 0 - 5 kN (0-

1124 lbf) cyclic load 100 times. This was intended to loosen the connections and ensure the test response was representative of floor performance in service. In phase 2, floor systems 3A1 to 3B2 were tested using the hammer impact test and the concentrated load test to failure. The concentrated load was applied at floor centre and the deflection of each joist at mid-span recorded prior to failure.

3.1 <u>Static deflection test</u>

This test was applied to floor systems in phase 1 only. The test procedure basically followed Method A in "CCMC Technical Guide for Bridging for floors" (1994) for evaluating stiffness of bridging material. Under the test procedure, a load of 1kN (225 lbf) was placed on each joist at mid-span, and the resulting deflections at mid-span of all joists were recorded. This weight was achieved by using a concrete block. The deflections at mid-span of all joists were measured simultaneously using linear variable displacement transducers (LVDT). This test procedure therefore resulted in seven deflection profiles.

3.2 <u>Hammer impact test</u>

This is a modal testing method intended to determine the natural frequencies of each floor system. The test equipment consisted of an instrumented hammer, an accelerometer and a spectrum analyser. The principles behind modal testing has been discussed by Ewins (1986). The test procedure was such that the floor was excited into motion by a hammer impact and the response measured by the accelerometer. The signals were then analysed by the spectrum analyser which output the natural frequencies. The impact was applied at a point between C6 and C7, and the acceleration at C1 to C7 was recorded sequentially.

3.3 <u>Simulated footfall impact test</u>

This test was applied to floors in phase 1 only. The test procedure of this test was similar to the hammer impact test. The difference being the impact device and the method of analysing the data. The objective of this test is to apply an impact which resembled that applied by a human footfall in terms of force and duration of impact. Previous tests by Chui (1987) and Hu (1998) showed that the impact force level and duration of footfall impact were approximately 500 N (112 lbf) and 70 ms respectively. In this study the impact was applied by dropping a medicine ball through a distance of about 150mm (6"). The drop distance and weight of medicine ball were chosen to provide the desirable force and duration. The impact force was measured by a force transducer. The response was measured by an accelerometer.

To determine the effects of impact locations, two locations were impacted : at floor centre (C4) and between locations B2 and B3 (as per Figure 3). The test procedure was such that an impact was applied at one of the two impact locations, the response of the floor to the impact at the same or another location was measured by an accelerometer. Three impacts were applied and the corresponding three pairs of force and acceleration signals were recorded for each response location. The results presented later for each response location is therefore based on the mean of the three response signals. For each impact location, this procedure was applied 21 times to measure data at all quarter points (B and D) and mid-span (C) of the joists (3 locations x 7 joists = 21). The response signals were then analysed using software developed at WSTC to calculate the peak acceleration and frequency-weighted root-mean-square (rms) acceleration, as described by Chui and Smith (1997). Both these parameters have been found to correlate with human response to building vibration.

3.4 Concentrated load test

Floor systems 1C, 2B, 3A1, 3A2, 3B1 and 3B2 were tested to destruction with a concentrated load applied at floor centre. The deflection during failure loading was recorded until it was felt that the LVDT's may be damaged with a sudden failure. With each floor the failure mode was recorded to identify the weakness in each system. As multiple failure modes may follow one another, video taping was used in assisting in identifying the first failure in a system.

4.0 RESULTS AND DISCUSSION

This study allowed the influence of a number of bridging and strapping details on floor behaviour to be evaluated. These details included:

- One row of bridging at mid-span and influence of joist spacing
- Two rows of bridging
- Combined bridging/strapping
- Fastening details
- Missing bridging members

The following describes the effects of each of the above separately. Test results are summarized in the appendices:
Appendix B - Natural frequencies from hammer impact test.
Appendix C - Static deflection profiles under a 1 kN (225 lbf) load at mid-span.
Appendix D - Frequency-weighted root-mean-square acceleration and peak acceleration of response signals caused by simulated footfall impacts.

4.1 Effects of one row of bridging at floor centre and the influence of joist spacing on its effectiveness

The effects of adding one row of bridging and the influence of joist spacing can be determined by comparing the results of floor systems 1A, 1C, 4A and 4B.

Results in Table B1 show that higher natural frequencies were raised by adding one row of bridging, although the first natural frequency (f_1) appeared virtually unaffected for both joist spacings. As has already been discussed by a number of researchers (Ohlsson, 1982, Chui 1987), this is a result of the increase in floor system stiffness in the across-joist direction. No appreciable increase in the first natural frequency was observed for both spacing indicating that the along-joist stiffness is not affected by adding bridging.

The static deflection profiles for these four floors are shown in Figures C1, C3, C6 and C7. The measured deflections at floor centre when the 1 kN (225 lbf) was applied there were respectively 1.22mm (0.048"), 1.04mm (0.041"), 0.83 (0.033") and 0.73mm (0.029"). The ratios of the bridged floor deflection to unbridged floor deflection for both joist spacing were similar : 1.04/1.22 = 0.85 for 610mm (24") joist spacing and 0.73/0.83 = 0.87 for 406mm (16") spacing. This may lead to the conclusion that effectiveness of bridging in reducing static deflection is unaffected by joist spacing. However examining the deflection profiles leads to a different conclusion. Studying the deflection profiles can provide information on the effectiveness of a bridging system in transferring any applied load to adjacent joists. Before the addition of bridging, base floor (Figures C1 and C6) deflection profiles show that in both cases five joists were involved in resisting the applied load. (The number of active joists is determined by noting the number of joists which have non-zero deflections when load is applied at mid-floor.) After the addition of the bridging (Figures C3 and C7), the number of active joists in floor system 1C remained as before. However, as is indicated by the fact that the edge joist deflections are not negligible, the number of active joists in system 4B had increased from five to at least seven. This observation suggests that for the floor with 406mm (16") joist spacing, the reduction in deflection may be larger had a larger number of floor joists been used in the test floors, and that the effectiveness of a bridging depends on the joist spacing.

The relative performance of floor systems under vibrational loading can be assessed by examining the frequency-weighted rms and peak acceleration of the responses caused by simulated footfall impacts. These results are presented in Tables D1, D2, D5 and D6. Tables D1 and D5 are for impacts applied between locations B2 and B3 whereas Tables D2 and D6 present results for impacts applied at floor centre (C4). An interesting point from the results is that when impacts were applied between B2 and B3, the highest rms and peak accelerations generally occurred at the floor edge closer to the impact location i.e. joist 1. The reason for this is unknown.

As expected, both peak and rms accelerations decreased with the addition of bridging. Reduction was highest at locations closest to the impact. Comparing the percentage reductions in acceleration responses between floors with the two spacings, it can be noted that the floor systems with a closer joist spacing had higher percentage reductions. For example, consider the rms accelerations at location C4 under impact at that location. The ratios of rms accelerations before and after addition of bridging for both joist spacings are : 1.283/1.421 = 0.90 for 610mm (24") spacing and 0.527/1.032 = 0.51 for 406mm (16") spacing. This result suggests that the addition of bridging is more effective in reducing floor vibration levels for floors with close joist spacing compared with floors with wide joist spacing.

One of the objectives of measuring floor component properties such as modulus and density was to compare measured responses with those predicted by various predictive techniques. One of the predictive techniques which is of interest here is the method of calculating floor deflection under a point load applied at floor centre proposed in a CCMC report (CCMC, 1997). Based on the CCMC method, the predicted deflections for the four systems (1A, 1C, 4A and 4B) under 1 kN (225 lbf) are 0.95mm (0.037"), 0.85mm (0.033"), 0.70mm (0.028") and 0.64mm (0.025") respectively. These compare with the measured deflections of 1.22mm (0.048"), 1.04mm (0.041"), 0.83mm (0.033") and 0.73mm (0.029"). The ratios of predicted to measured deflection are 0.78, 0.82, 0.85 and 0.88. It seems that the accuracy of the CCMC predictive method is dependent on the joist spacing. Better agreement with measured deflection is achieved for the closer joist spacing. It should be pointed out that joist spacing is not an input parameter in the current CCMC method.

4.2 Effects of two rows of bridging compared with one row

Base floor 1 (1A) was retested with 1 row (1C) and 2 rows (1B) of bridging to study the benefits of having more than one row of bridging. In system 1C the bridging was attached at mid-span whereas in system 1B the two rows were installed at third points of the span. Natural frequency results are shown in Table B1. It can be seen that the separation between adjacent natural frequencies increases as the number of rows of bridging increases. This again is due to the increase in across-joist floor system stiffness. Static deflection profiles are shown in Figures C1, C3 and C2 for floor systems 1A, 1C and 1B respectively. The floor centre deflections under 1 kN load (225 lbf) are 1.22mm (0.048"), 1.04mm (0.041") and 0.79mm (0.031") respectively. Therefore, substantial reduction in deflection under a point load can be achieved by adding more rows of bridging. Significant uplift deflections were noted at the edge joists for system 1B. This behaviour was different from that observed in an earlier CMHC ERP project on floors with concrete topping (Chui and Smith, 1997) where no uplift deflections were observed after the addition of concrete topping, although in both cases the across-joist floor system stiffness increased. This difference in behaviour can be explained by the fact that the presence of a 'rigid' plate suppresses the uplift movement at the edges.

Although the reduction in static deflection was substantial with an increase in number of rows of bridging, the reduction in acceleration level was not, as is indicated by the results shown in Tables D1 and D2. As an example, the rms accelerations at location C4 in Table D2 are 0.877 m/s^2 (2.87ft/s^2), 0.792 m/s^2 (260ft/s^2) and 0.772 m/s^2 (2.53ft/s^2) respectively for systems 1A, 1C and 1B. Thus the two bridged floor rms acceleration at this location was 0.9 m/s^2 (2.95ft/s^2) (1 row) and 0.88 m/s^2 (2.89ft/s^2) (2 rows) of the unbridged floor rms acceleration.

The readers may recall that two simulated impact locations were used in this study. The purpose of doing this is to determine if the location of impact relative to the bridging location affects our conclusions on the effectiveness of cross bridging. For this reason an impact location between B2 and B3 was chosen since it was close to a row of bridging in floor system 1B. Comparing the results in Tables D1 and D2 reveals no indication that the effectiveness of the bridging is dependent on the location of impact relative to the bridging location. A supporting evidence of this is that in Table D2 the C4 rms and peak accelerations for floor 1B are lower than the corresponding values for floor 1C despite the fact that in the case of floor 1C the impact was applied directly over the single row of bridging.

4.3 Effect of combined bridging/strapping

The use of a bottom strapping was reported to have a positive synergistic influence on the effectiveness of a bridging in sawn lumber floors (Onysko and Jessome 1973, Chui and Smith, 1991). Floor systems 2B and 4C had a 25 x 102mm (1 x 4") strapping attached to the underside of joists and were tested to determine the influence of strapping on both static and vibrational behaviours of wood I-joist floor systems. Comparing the natural frequencies for systems 2A, 2B, 4B and 4C reveals that, as expected, natural frequencies of higher vibration modes increased substantially (Table B1) with the addition of strapping for both joist spacing. Deflection profiles for the four systems are shown in Figures C4, C5, C7 and C8 respectively. The floor centre deflections are 1.16mm (0.046"), 0.81mm (0.032"), 0.73mm (0.029") and 0.64mm (0.025") respectively. These deflection values suggest that floors with wider joist spacing benefit more from the attachment of a bottom strapping. This conclusion is supported also by studying the deflection profiles. It can be observed that the deflection profiles for systems 4B and 4C (Figures C7 and C8) are similar indicating minor changes in static floor characteristics. In contrast the differences between Figure

C4 and C5 are much more substantial.

Floor acceleration results are shown in Tables D3, D4, D5 and D6. These tables show that significant reductions in acceleration levels were obtained with the addition of strapping. As in the case for static deflection, the floor with the wider joist spacing (systems 2A and 2B) benefitted more from the attachment of strapping.

The above mentioned CCMC method of calculating static deflection recognizes the synergistic effects of having both cross bridging and bottom strapping. Using the CCMC method, the calculated deflections for the systems 2A, 2B, 4B and 4C are respectively 0.95mm (0.037"), 0.77mm (0.030"), 0.64mm (0.025") and 0.58mm (0.023"). Compared with test responses, these represent respectively 0.81, 0.95, 0.88 and 0.92 of measured deflections. It has been discussed above that the CCMC method appears to be more accurate for floors with bridging than without bridging. The results here demonstrate that best accuracy is achieved when predicting deflection of floors with both bridging and strapping.

4.4 <u>Fastening details of bridging and failure mode</u>

The above discussion focussed on the serviceability aspects of floor behaviour. As mentioned at the beginning, one of the objectives of this project is to study the failure behaviour in wood I-joist floors with bridging, and recommend improvement details if appropriate. To that end two floors in phase 1 were tested to failure : systems 1C (1 row of bridging) and 2B (1 row of bridging/strapping). These systems were tested to failure with a concentrated load applied at the floor centre (location 4C) using a hydraulic actuator. They failed at 25.6 kN (5755 lbf) and 26.5 kN (5958 lbf) respectively. Based on single joist response under a point load at mid-span, these values translate into bending moment capacities of 30.7 kNm (22644 lb·ft) and 32.04 kNm (23633 lb·ft)respectively. Comparing with the working stress design moment capacity of 4.62 kNm (3408 lb·ft)for the product indicates that these floors have bending moment capacity well above the design capacity under the common, conservative assumption that a point load is resisted by a single joist.

Because of the small number of replicates used here, it is impossible to conclude if the strapping led to the increase in strength. However the main objective of this test was to identify any weakness in the bridging system so that improvement can be suggested i.e. how a floor fails rather than the magnitude of the load.

Past experience indicated that if the bridging was to fail first, then it would occur at the connection to the joists. To that end, the movement at critical locations of bridging connection was video-taped to enable observation of failure mode. It was observed that first failure in both cases occurred at the finger joint closest to the loading point in the centre joist on the tension edge. There were signs of a slight pull-out of nails in the bridging connections near the loading point for floor system 1C. However, no complete pull-out was observed at failure. This indicated that if properly installed, the 2-nail bridging connection is adequate to provide effective transfer of the load.

Phase 2 of this project was originally reserved for studying improved fastening details. As tests in Phase 1 indicated that current fastening is adequate, the scope of Phase 2 was therefore modified to study the influence of inadequate bridging members which could arise as a result of omitting one of the cross members, excessive splitting of wood caused by fastening action or nail pull-out caused by shrinkage of wood. The results are discussed in the next section.

4.5 Influence of inadequate bridging members on failure mode

The difficulties in achieving good field connection for cross bridging are well known, and yet the effectiveness of cross bridging depends almost entirely on the quality of its connections to the joists. The problems are even more prominent for wood I-joists because of the limited nailing areas in the flanges. Improper nailing does not only lead to poor bridging effectiveness, but also splitting of wood material causing potential weakening of the joists themselves.

Because of its thin web there have been some concern regarding premature localised joist failure under the action of a heavy point load. Phase 2 of this project was intended to study the impact of missing cross bridging members on failure modes. Four narrow floor specimens were tested with details shown in Figure 2. Systems 3A1 and 3B1 were expected to lead to the two possible premature failure modes as the middle joist moves down : pull-out of nailed connections at the top flange and sideway deflection of bottom flange of the outside joists. Systems 3A2 and 3B2 had bottom strapping added to determine if these premature failure modes can be suppressed by the strapping.

These four systems were first tested by the hammer impact test to estimate their natural frequencies. Natural frequencies of the four systems are presented in Table B1. Higher natural frequencies were noted for the systems with strapping.

The failure load recorded in the fours systems are : 16.3 kN (3665 lbf), 27.9 kN (6272 lbf), 21.6 kN (4856 lbf) and 33.4 kN (7509 lbf) respectively. The corresponding failure moments are : 19.6 kNm (14457 lb·ft), 33.5 kNm (24709 lb·ft), 25.9 kNm (19194 lb·ft) and 40.1 kNm (29577 lb·ft). All of these are well above the design bending moment capacity of the joist (4.62 kNm or 3408 lb·ft).

Behaviour of each floor during loading to failure is described below:

System 3A1 (16.3 kN or 3665 lbf) - This floor had the lowest failure load. Significant nail pull-out at one top flange-to-bridging connection prior to failure, although no complete pull-out of nail was observed at failure. Failure occurred when the centre joist broke near a finger joint.

System 3A2 (27.9 kN or 6272 lbf) - Some degree of nail pull-out was observed at the top flange connection at failure which occurred near a finger joint in the middle joist. No over-stress was observed in the strapping to joist connections.

System 3B1 (21.6 kN or 4856 lbf) - This system achieved a higher capacity compared with 3A1, but lower than the other two systems. Expected side-way deflection of outside joists was observed. Side-way deflection of bottom flange at failure was about 10mm (0.394"). The magnitude of this horizontal deflection was similar to the vertical deflection of the same joists. Bridging connection performed adequately. Relatively low capacity was probably due to the side-way movement of outside joist bottom flange which means the outside joists did not contribute much in sharing the applied load.

System 3B2 (33.4 kN or 7509 lbf) - This system had the highest failure load. The system was able to undergo large vertical deflection before failure occurred at a finger joint in the middle joist. Side-way deflection of bottom flange was observed but magnitude was smaller than in 3B1 (about 6mm (0.236") at 21 kN (4721 lbf)). The large vertical deflection caused the strapping nails to come off the joist at one outside joist. However no signs of over stress at bridging to joist connections was observed. These observations suggest that the strapping played a much bigger role in resisting applied load than the bridging.

The above results show that a significant reduction in moment capacity can be encountered if bridging members are omitted or fastening is inadequate. However the failure moment capacity is still above the design moment capacity of a single joist. The negative impact caused by omitted bridging members can be mostly negated by the addition of a bottom lumber strapping.

5.0 CONCLUSIONS

1. The addition of cross bridging causes more joists to share any applied load, thereby lowering the response at the loaded joist. Both static deflection and vibration level decrease with the addition of bridging in floors built with wood I-joists. The effectiveness of a bridging depends on the joist spacing. All things being equal, floors with a closer joist spacing experience bigger reductions in deflection and vibration levels than floors with a wider joist spacing. For the test floors with 406mm (16") joist spacing, the reduction in deflection would have been larger had a larger number of floor joists been used in the test floors.

2. Both static deflection and vibration level decrease with increasing number of rows of bridging. However the sensitivity differs for static deflection and vibration. The reduction in static deflection is substantial with any increase in number of rows of bridging, but the reduction in acceleration level appears small.

3. There is no indication that the effectiveness of the bridging is dependent on the location of impact relative to the bridging location, for the size of floor studied here.

4. The addition of a bottom lumber strapping leads to a significant improvement in floor performance by lowering static deflection and acceleration levels, compared with the use of bridging alone. In contrast with the use of cross bridging alone, floors with wider joist spacing appear to benefit more from the attachment of a bottom strapping than floors with a closer joist spacing. The presence of a strapping also serves to negate some of the problems associated with improper installation of cross bridging.

5. If properly installed, the common practice of using two 57mm (2¼") spiral nails at each end of a bridging member for fastening to joists appears adequate to transfer an applied concentrated load until failure of a joist occurs.

6. The results obtained here suggest that a significant reduction in moment capacity can be encountered if bridging members are omitted or fastening is inadequate. However the failure moment capacity is still above the design moment capacity of a single joist. The negative impact caused by omitted bridging members can be mostly negated by the addition of a bottom lumber strapping.

6.0 ACKNOWLEDGMENTS

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APPENDIX A - Floor component properties.

Table A1 - Floor 1 I-joist properties.

Joist location and specimen #	Length (mm)	Depth (mm)	f_1 (Hz)	f_2 (Hz)	Mass/ length (kg/m)	Flexural stiffness ($x10^{12}$ Nmm^2)	Shear stiffness ($x10^{12}$ N)
1 = 29	5473	303	59	142.5	4.33	1.22	4.34
2 = 14	5473	300	59	146	4.21	1.16	5.47
3 = 15	5472	301	56.25	141.5	4.1	1	6.01
4 = 16	5473	302	57.5	144.3	4.19	1.08	6.18
5 = 18	5471	300	58.75	143.8	4.36	1.2	4.94
6 = 25	5473	302	58.75	143.8	4.34	1.2	4.92
7 = 26	5473	301	55.5	141.5	4.22	1	7.47

Table A2 - Floor 2 I-joist properties.

Joist location and specimen #	Length (mm)	Depth (mm)	f_1 (Hz)	f_2 (Hz)	Mass/ length (kg/m)	Flexural stiffness ($x10^{12}$ Nmm^2)	Shear stiffness ($x10^{12}$ N)
1 = 21	5473	303	59	144	4.36	1.22	4.85
2 = 2	5472	300	59.25	147.25	4.19	1.16	5.79
3 = 6	5473	300	54.25	136.75	4.25	0.97	5.98
4 = 27	5472	302	57.25	144.25	4.24	1.08	6.59
5 = 4	5473	300	58.5	143.75	4.33	1.18	5.1
6 = 13	5473	300	59.5	148.75	4.22	1.17	6.35
7 = 8	5473	302	56.5	145.5	4.24	1.03	9.44

Table A3 - Floor 4 I-joist properties.

Joist location and specimen #	Length (mm)	Depth (mm)	f_1 (Hz)	f_2 (Hz)	Mass/ length (kg/m)	Flexural stiffness ($x10^{12}$ Nmm^2)	Shear stiffness ($x10^{12}$ N)
1 = 1	5473	300	61	149.25	4.2	1.25	5.13
2 = 10	5473	302	58.5	142	4.22	1.16	4.36
3 = 7	5473	303	56.5	142.75	4.27	1.06	6.75
4 = 28	5473	300	57.25	144.25	4.2	1.07	6.53
5 = 23	5473	302	58.75	145.75	4.21	1.15	5.59
6 = 22	5473	300	58.75	142.75	4.22	1.17	4.45
7 = 3	5472	300	58.25	145	4.16	1.10	5.67

Table A4 - Floor 3A1 joist properties.

Joist location and specimen #	Length (mm)	Depth (mm)	f_1 (Hz)	f_2 (Hz)	Mass/ length (kg/m)	Flexural stiffness ($x10^{12}$ Nmm^2)	Shear stiffness ($x10^{12}$ N)
1 = 19	5473	303	57	143.25	4.41	1.11	6.55
2 = 17	5473	300	56.5	140	4.26	1.07	5.16
3 = 20	5473	300	54.75	139	4.32	1.00	6.92

Table A5 - Floor 3A2 joist properties.

Joist location and specimen #	Length (mm)	Depth (mm)	f_1 (Hz)	f_2 (Hz)	Mass/ length (kg/m)	Flexural stiffness ($x10^{12}$ Nmm^2)	Shear stiffness ($x10^{12}$ N)
1 = 19	5473	303	57	143.25	4.41	1.11	6.55
2 = 10	5473	302	58.5	142	4.22	1.16	4.36
3 = 20	5473	300	54.75	139	4.32	1.00	6.92

Table A6 - Floor 3B1 I-joist properties.

Joist location and specimen #	Length (mm)	Depth (mm)	f_1 (Hz)	f_2 (Hz)	Mass/ length (kg/m)	Flexural stiffness ($x10^{12}$ Nmm^2)	Shear stiffness ($x10^{12}$ N)
1 = 19	5473	303	57	143.25	4.41	1.11	6.55
2 = 9	5473	300	57	146.25	4.24	1.05	8.91
3 = 20	5473	300	54.75	139	4.32	1.00	6.92

Table A7 - Floor 3B2 I-joist properties.

Joist location and specimen #	Length (mm)	Depth (mm)	f_1 (Hz)	f_2 (Hz)	Mass/ length (kg/m)	Flexural stiffness ($x10^{12}$ Nmm^2)	Shear stiffness ($x10^{12}$ N)
1 = 19	5473	303	57	143.25	4.41	1.11	6.55
2 = 24	5473	300	57	138.75	4.28	1.12	4.32
3 = 20	5473	300	54.75	139	4.32	1.00	6.92

Table A8 - OSB panel properties.

Panel no.	Thickness (mm)	Density (kg/m^3)	Major bending modulus (MPa)	Minor bending modulus (MPa)	Shear modulus (MPa)
1	18.3	657	9963	5209	302
2	18.7	647	9975	4525	249
3	19.1	635	9690	4216	238
4	18.8	642	9964	4749	269
5	18.6	655	10100	4666	261

Table A9 - Lumber strapping properties

Strapping no.	Thickness (mm)	Width (mm)	MOE (MPa)
1	19.05	89	6905
2	19.05	89	7493

APPENDIX B - Natural frequencies of test floors.

Table B1 - Natural frequencies of test floors.

Floor system	Natural frequencies (Hz)				
	f_1	f_2	f_3	f_4	f_5
1A	19	21.125	25.25	27.25	----
1B	19.125	21.375	28.125	41.5	58.625
1C	18.75	21.375	----	31.125	43.00
2A	19.375	21.625	25.25	27.25	----
2B	19.75	22.25	32.625	43.625	51.125
3A1	21.375	26.625	51.875	----	----
3A2	21.625	27.125	69.250	----	----
3B1	21.625	24.125	53.875	----	----
3B2	21.625	25.250	60.000	----	----
4A	21.0	23.625	34.0	43.750	57.875
4B	21.375	24.000	33.75	59.875	----
4C	21.375	23.875	44.750	68.250	78.875

APPENDIX C - Deflection profiles under a 1 kN load.

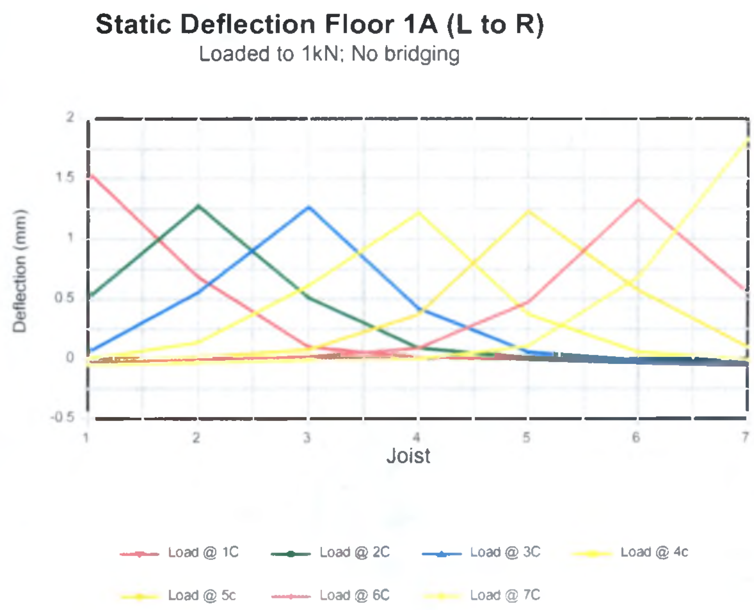

Figure C1 - Deflection profile of floor 1A.

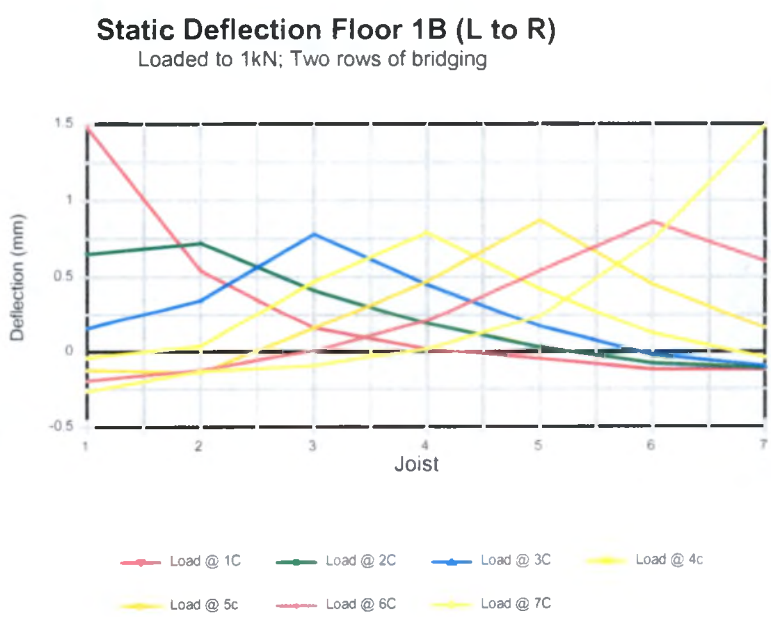

Figure C2 - Deflection profile of floor 1B.

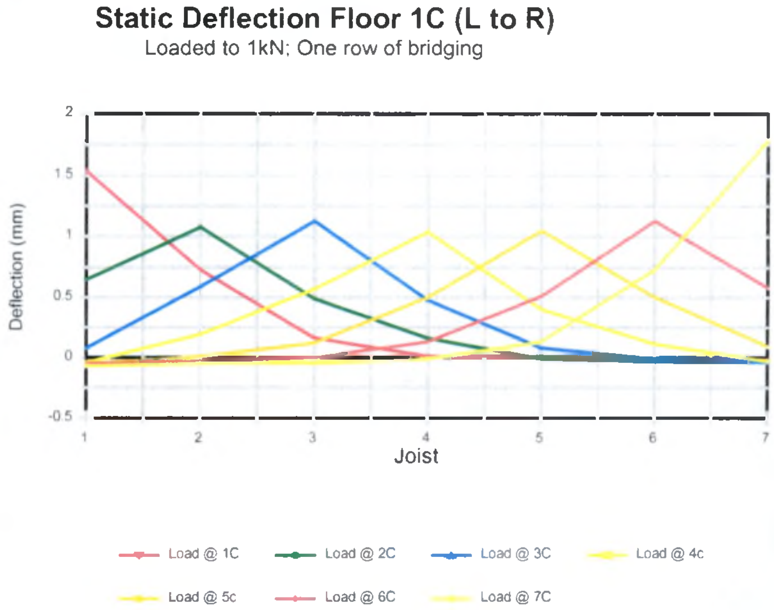

Figure C3 - Deflection profile of floor 1C.

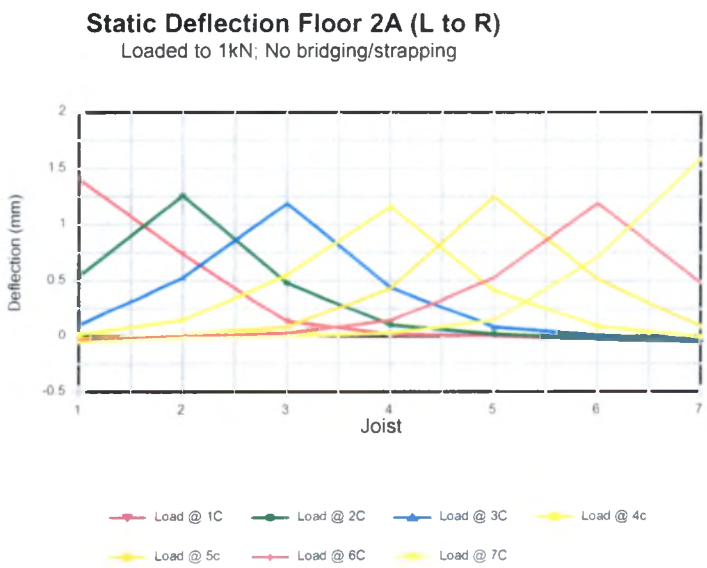

Figure C4 - Deflection profile of floor 2A.

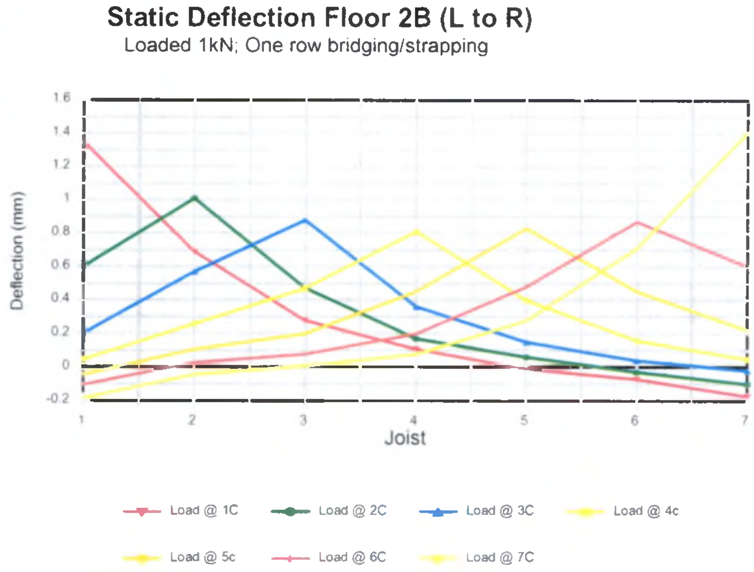

Figure C5 - Deflection profile of floor 2B.

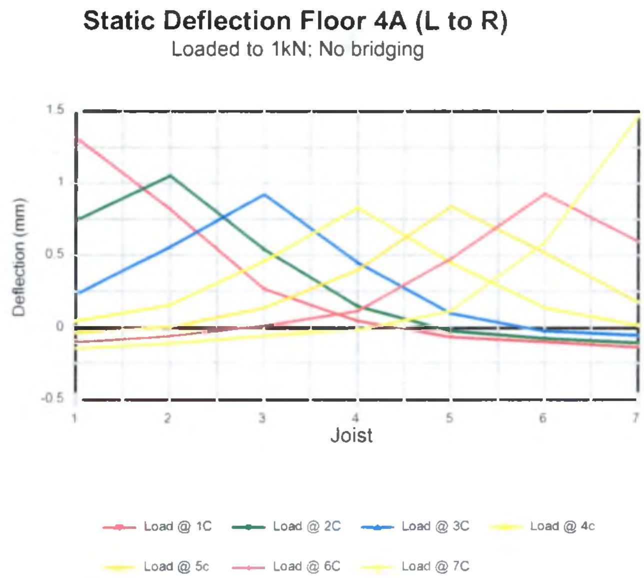

Figure C6 - Deflection profile of floor 4A.

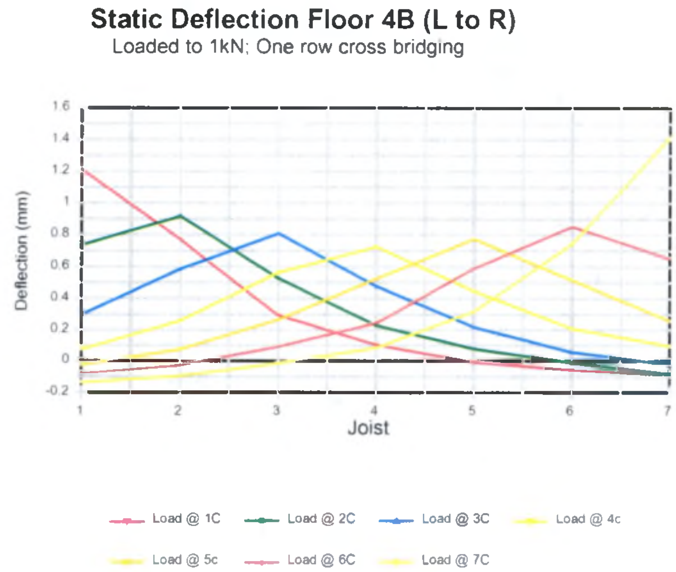

Figure C7 - Deflection profile of floor 4B

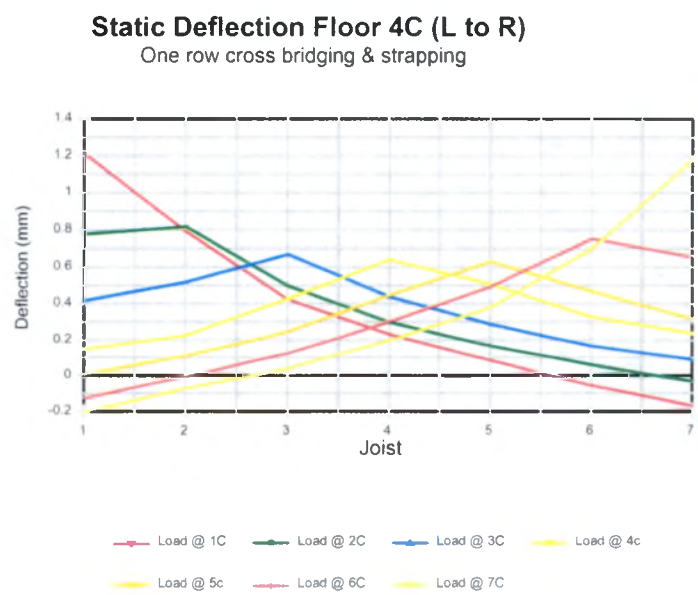

Figure C8 - Deflection profile of floor 4C.

APPENDIX D - Frequency-weighted root-mean-square acceleration and peak acceleration of floor response to simulated footfall impacts.

Table D1 - Frequency-weighted rms and peak accelerations for floor 1A, 1B and 1C with impact applied between location B2 and B3 (m/s^2).

Floor	Location and response	Joist no.						
		1	2	3	4	5	6	7
1A	D							
	Rms	.718	.765	.8115	.722	.7257	.693	.632
	Peak	18.80	15.565	16.252	13.047	11.97	9.91	14.49
	C							
	Rms	.934	1.013	1.011	.8772	.896	.941	.971
	Peak	18.835	15.01	17.397	10.625	9.679	9.241	12.786
	B							
	Rms	.696	.825	.822	.703	.685	.694	.758
	Peak	17.33	14.846	15.89	12.75	12.26	8.69	13.73
1B	D							
	Rms	.665	.602	.605	.588	.539	.579	.650
	Peak	14.682	8.773	11.87	10.163	8.747	6.02	9.748
	C							
	Rms	.919	.885	.879	.772	.730	.842	.893
	Peak	15.925	12.099	12.982	9.78	6.514	7.07	11.347
	B							
	Rms	.768	.697	.692	.608	.612	.675	.785
	Peak	15.271	11.131	13.146	9.166	7.950	6.389	11.216
1C	D							
	Rms	.728	.714	.750	.597	.579	.568	.598
	Peak	16.285	15.042	16.121	10.389	10.758	8.783	13.505
	C							
	Rms	.856	.839	.929	.792	.823	.878	1.024
	Peak	14.127	15.538	14.519	9.094	9.441	8.655	12.72
	B							
	Rms	.744	.753	.780	.566	.648	.649	.700
	Peak	16.187	14.061	15.467	9.55	12.263	8.365	12.001

Table D2 - Frequency-weighted rms and peak accelerations for floor 1A, 1B and 1C with impact applied at location C4 (m/s^2).

Floor	Location and response	Joist no.						
		1	2	3	4	5	6	7
1A	D							
	Rms	.792	.851	.925	1.149	.780	.620	.733
	Peak	12.786	10.791	11.445	18.405	10.932	8.00	12.23
	C							
	Rms	1.165	1.177	1.233	1.421	1.085	.970	1.016
	Peak	19.85	14.257	14.781	18.606	15.729	13.048	16.317
	B							
	Rms	.865	.86	.915	1.123	.777	.6929	.756
	Peak	13.80	9.414	10.595	18.017	11.085	8.58	13.571
1B	D							
	Rms	.738	.593	.697	.894	.681	.588	.738
	Peak	10.251	7.557	8.084	14.486	9.195	7.237	10.107
	C							
	Rms	.913	.86	.976	1.139	.962	.818	1.042
	Peak	12.328	11.085	11.249	15.598	10.219	8.937	13.211
	B							
	Rms	.821	.635	.746	.863	.759	.644	.84
	Peak	12.033	8.002	9.241	11.118	11.151	7.998	12.099
1C	D							
	Rms	.814	.681	.777	.956	.662	.552	.736
	Peak	11.543	9.126	8.956	14.748	8.835	6.763	11.838
	C							
	Rms	1.068	.900	1.1078	1.283	1.014	.805	.953
	Peak	13.472	11.347	12.132	17.233	13.538	10.673	11.968
	B							
	Rms	.879	.701	.793	.903	.713	.621	.791
	Peak	11.936	8.711	9.856	14.813	10.189	8.835	11.903

Table D3 - Frequency-weighted rms and peak accelerations for floor 2A and 2B with impact applied between B2 and B3 (m/s^2).

Floor	Location and response	Joist no.						
		1	2	3	4	5	6	7
2A	D							
	Rms	.767	.821	.723	.659	.726	.710	.676
	Peak	19.129	17.069	15.402	14.323	10.464	9.915	14.62
	C							
	Rms	1.106	1.005	.972	.871	.994	.710	.823
	Peak	22.17	16.121	12.671	11.772	10.392	9.915	13.145
	B							
	Rms	.779	.876	.733	.684	.714	.681	.638
	Peak	19.36	16.71	15.794	13.08	12.2	9.44	14.75
2B	D							
	Rms	.752	.663	.567	.493	.5442	.570	.646
	Peak	20.928	15.14	11.347	9.218	10.621	6.80	12.688
	C							
	Rms	.992	.967	.80	.697	.697	.783	.932
	Peak	19.62	15.281	14.454	10.045	7.103	6.903	15.892
	B							
	Rms	.734	.726	.586	.53	.574	.564	.678
	Peak	16.219	15.281	12.733	10.693	9.382	9.081	11.936

Table D4 - Frequency-weighted rms and peak accelerations for floor 2A and 2B with impact applied at C4 (m/s^2).

Floor	Location and response	Joist no.						
		1	2	3	4	5	6	7
2A	D							
	Rms	.825	.788	.860	1.063	.784	.753	.922
	Peak	13.003	10.987	10.987	18.482	10.33	9.676	13.734
	C							
	Rms	1.07	.990	1.045	1.342	1.075	1.059	1.155
	Peak	18.018	11.511	14.028	18.607	14.069	13.472	16.056
	B							
	Rms	.865	.856	.837	1.025	.743	.752	.876
	Peak	14.715	9.902	11.249	15.958	10.301	9.349	12.524
2B	D							
	Rms	.615	.613	.623	.706	.641	.565	.6091
	Peak	12.720	9.009	10.137	13.243	8.528	7.56	10.467
	C							
	Rms	.802	.791	.820	.946	.889	.735	.897
	Peak	13.8	10.065	10.858	14.486	11.641	9.202	15.467
	B							
	Rms	.616	.646	.652	.729	.602	.575	.6091
	Peak	10.359	9.659	10.689	14.192	9.166	9.192	10.467

Table D5 - Frequency-weighted rms and peak accelerations for floor 4A, 4B and 4C with impact applied between B2 and B3 (m/s^2).

Floor	Location and response	Joist no.						
		1	2	3	4	5	6	7
4A	D							
	Rms	.876	.882	.758	.583	.504	.350	.404
	Peak	23.152	18.377	16.906	10.464	13.832	7.747	15.392
	C							
	Rms	1.210	1.172	1.0188	.7503	.587	.506	.5145
	Peak	20.568	21.909	15.697	8.744	8.89	8.365	11.42
	B							
	Rms	1.043	1.017	.927	.594	.448	.3485	.3832
	Peak	15.565	16.219	17.069	8.685	8.43	7.413	11.785
4B	D							
	Rms	.3568	.3232	.2808	.2329	.1828	.2050	.1467
	Peak	8.786	5.968	6.586	5.317	5.033	3.773	4.500
	C							
	Rms	.4988	.4912	.4235	.3228	.2451	.179	.1683
	Peak	10.353	7.253	7.639	4.849	3.44	2.394	3.904
	B							
	Rms	.4729	.3987	.4027	.2674	.1876	.1569	.1497
	Peak	7.018	6.121	10.15	5.003	2.943	2.786	5.173
4C	D							
	Rms	.3117	.2719	.259	.2025	.1634	.1403	.1602
	Peak	8.214	5.479	5.255	4.042	4.431	3.146	5.134
	C							
	Rms	.4605	.3784	.3183	.2849	.213	.1610	.192
	Peak	9.82	5.576	5.291	4.98	3.126	2.174	4.434
	B							
	Rms	.3809	.3899	.5441	.2382	.1793	.1431	.463
	Peak	7.838	9.038	15.866	5.133	3.11	3.113	3.135

Table D6 - Frequency-weighted rms and peak accelerations for floor 4A, 4B and 4C with impact applied at C4 (m/s^2).

Floor	Location and response	Joist no.						
		1	2	3	4	5	6	7
4A	D							
	Rms	.8112	.777	.7358	.7658	.6103	.6142	.7187
	Peak	16.284	14.290	11.053	16.416	10.245	9.803	12.426
	C							
	Rms	1.162	1.036	.9408	1.0325	.8899	.9239	1.013
	Peak	19.228	13.047	13.701	17.985	15.546	16.808	18.803
	B							
	Rms	1.112	.8825	.7439	.7391	.6538	.6565	.7656
	Peak	20.67	10.726	11.658	15.860	13.734	11.521	13.587
4B	D							
	Rms	.3721	.2872	.2731	.2889	.2293	.2212	.309
	Peak	8.289	5.513	4.234	6.206	4.336	3.934	5.029
	C							
	Rms	.4735	.3834	.3971	.5269	.3353	.2837	.3758
	Peak	7.648	4.212	5.471	11.376	4.787	4.745	6.539
	B							
	Rms	.4602	.3756	.3427	.2844	.2255	.2337	.2947
	Peak	8.613	4.765	5.434	5.880	5.009	3.855	4.747
4C	D							
	Rms	.3093	.2798	.2544	.2245	.1958	.2089	.2406
	Peak	5.667	4.637	3.878	4.467	4.084	3.993	4.977
	C							
	Rms	.4576	.3867	.3331	.3867	1.9437	.2789	.3136
	Peak	7.274	4.806	4.362	7.678	7.475	5.339	6.347
	B							
	Rms	.3664	.3316	.3015	.2467	.2269	.2001	.2394
	Peak	5.896	4.313	4.127	5.206	4.685	3.678	4.388