Experimental investigation of system effects in stressed-skin elements

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Design of timber structures is, in general, based on single-element analyses considering the same material behaviour for every element (e.g. characteristic strength and stiffness properties). Wood is a material of relatively large variation in strength and stiffness properties. The variation in strength and stiffness can be found between different timber beams as well as within a single beam (Isaksson, 1999).

In general, system effects originate from two different mechanisms. The principle of load-sharing effect between beams result in one mechanism, i.e. a weak beam placed beside stronger beams will carry a smaller portion of load due to the high degree of correlation between stiffness and strength of timber. The type of loading and the distance between the beams influence the extent of load-sharing effect. Another mechanism leading to system effects is the probability of placing the weakest section of a beam in the most stressed position. (Hansson & Isaksson, 2001) carried out a study on the system effect in sheathed parallel-timber-beam structures. Systems are generated using Monte Carlo simulations and the system effects are evaluated by direct comparisons of strengths of single elements and by reliability methods. In this study, system failure was defined as failure of two adjacent beams or failure of any three individual beams in the system. This definition was established in an experimental investigation in Lund (Håkansson and Mauritz, 1999). The purpose of this project is to experimentally investigate the system effects of prefabricated roof units.

Test methods

In principle, evaluation of the results concerning system effects requires information of the strength of each beam placed in the system. Therefore, the strength is determined using non-destructive methods by measuring the bending stiffness of every beam used in the roof unit. A representative selection of 15 beams is tested for bending strength. Assuming a linear relation between bending stiffness and bending strength provides an estimate of the bending strength of the timber material used in the units.

Beams

The tests included 60 pieces of sawn timber (spruce) machine-graded for strength class K18 (characteristic bending strength of 18 MPa). The beams were sawn at Tvärskogs Träindustri in southern Sweden and are representative of ordinary deliveries for industrial fabrication of roof units. Dimensions of the timber beams are given in mm: depth/width/length: 195/35/4790. The sheathing consisted of plywood (12 mm) on the compression side of the roof units and a wood wool lightweight building board on the tension side.

Bending stiffness of the individual 60 timber beams was determined in 4 point bending tests according to EN408. Figure 1 shows the cumulative distribution function of measured beam stiffness. The bending direction was similar to the direction for bending considered in the testing of the roof units.
To predict the strength of the beams a selection was made of 15 beams representing the distribution of the total population of 60 beams. In Figure 1, the 15 selected beams are indicated by a circle around the corresponding stiffness measurement. Strength tests were conducted in bending according to EN408 with a few changes. Results from the tests are shown in Figure 2 as strength against beam stiffness. The Figure also shows a linear regression representing the relation between strength and stiffness of the beams used in this study.

A relation between strength and stiffness measurements was obtained by linear regression on the results shown in Figure 2 is presented in Equation 1.

$$f_{b,i} = 2.0 \frac{\text{MPa}}{\text{GPa}} \cdot E_i + 11.0 \text{ MPa}$$

(1)
where $f_{b,i}$ is the bending strength and $E_i$ is the stiffness of beam number $i$. The dotted line in Figure 2 represents a relation found under similar conditions in a major experimental study (Isaksson, 1999). The linear regression of that study predicts a significantly different relation from the relation in Equation 1.

The distribution of strength is shown in Figure 3 together with a distribution function. The characteristic value taken as the 5 percentile of the fitted distribution is 25.5 MPa. In Figure 3, also two distributions determined from the relation in Equation 1 and the relation in Isaksson (1999) are shown. Both distributions are determined from the distribution of measured stiffness of 60 beams shown in Figure 1.

The distribution for strength determined from the relation between strength and stiffness given in Isaksson (1999) differs significantly from the relation in the present study. The results in Isaksson (1999) are based on a large number of tests whereas the present study contains only very few strength and stiffness tests. Nevertheless, due to difference in grading criteries for the materials used in the two investigations, the results obtained in the present study are used to predict the strength of the material used.

Roof units
The remaining part of the beams (a total of 45 beams) not used for strength tests was used for the manufacture of 9 prefabricated roof units. The units are made from 5 parallel beams, attached to a transverse head beam at the ends. A plywood sheathing is glued and nailed to the top side (compression side) and a wood-wool lightweight building board is nailed to the bottom side of the units.

To increase the basis of information in the tests, the combination of beams in the units was predefined. This was due to the fact that the number of units was too small for a statistical investigation of randomly combined units. The selection of beams was based on 3 groups defined by modulus of elasticity ($w$: $E < 12$ GPa, $m$: $12$ GPa $< E < 13.8$ GPa, and $s$: $E > 13.8$ GPa). The combination of beams in units 1-9 is given in Table 1.
Table 1. Combination of beams in the 9 roof units. The individual beams are presented by measured stiffness. Unit type is referring the combination of: weak \((w): E < 12 \text{ GPa}\), medium \((m): 12 \text{ GPa} < E < 13.8 \text{ GPa}\), and strong \((s): E > 13.8 \text{ GPa}\) beams.

<table>
<thead>
<tr>
<th>Unit no.</th>
<th>Beam 1 (GPa)</th>
<th>Beam 2 (GPa)</th>
<th>Beam 3 (GPa)</th>
<th>Beam 4 (GPa)</th>
<th>Beam 5 (GPa)</th>
<th>Unit type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit 1</td>
<td>18.8</td>
<td>18.4</td>
<td>18.2</td>
<td>21.0</td>
<td>17.4</td>
<td>s-s-s-s-s</td>
</tr>
<tr>
<td>Unit 2</td>
<td>17.0</td>
<td>18.6</td>
<td>17.2</td>
<td>16.6</td>
<td>17.4</td>
<td>s-s-s-s-s</td>
</tr>
<tr>
<td>Unit 3</td>
<td>13.2</td>
<td>12.8</td>
<td>13.8</td>
<td>12.8</td>
<td>12.2</td>
<td>m-m-m-m-m</td>
</tr>
<tr>
<td>Unit 4</td>
<td>9.4</td>
<td>10.6</td>
<td>9.4</td>
<td>10.4</td>
<td>10.4</td>
<td>w-w-w-w-w</td>
</tr>
<tr>
<td>Unit 5</td>
<td>11.2</td>
<td>9.6</td>
<td>10.0</td>
<td>10.2</td>
<td>11.8</td>
<td>w-w-w-w-w</td>
</tr>
<tr>
<td>Unit 6</td>
<td>15.0</td>
<td>11.2</td>
<td>17.2</td>
<td>10.4</td>
<td>15.4</td>
<td>s-w-s-w-s</td>
</tr>
<tr>
<td>Unit 7</td>
<td>8.4</td>
<td>15.8</td>
<td>9.8</td>
<td>17.0</td>
<td>9.6</td>
<td>w-s-w-s-w</td>
</tr>
<tr>
<td>Unit 8</td>
<td>14.8</td>
<td>10.0</td>
<td>14.4</td>
<td>8.4</td>
<td>15.8</td>
<td>s-w-s-w-s</td>
</tr>
<tr>
<td>Unit 9</td>
<td>8.6</td>
<td>15.0</td>
<td>7.8</td>
<td>13.8</td>
<td>8.1</td>
<td>w-s-w-s-w</td>
</tr>
</tbody>
</table>

Roof unit strength was determined by testing 3 units turned up-side-down, connected side-by-side, fixed by nails through the adjacent beams, and loaded with a uniform surface load by an air cushion placed between the floor and the units (this load type is similar to snow load). Prior to testing, the wood-wool lightweight building boards were removed from the units to facilitate deformation measurements. The strength of the lightweight building boards was considered to have insignificant influence on the overall strength of the roof units. Load cells placed between the timber beams and the supporting steel beams measure the load carried by each beam in the units (see Figure 4). Supports consist of 2 steel beams (height 300 mm) with a span of 8 m allowing 3 units to be tested simultaneously.

At the first test the supporting steel beam buckled in the compression zone as a result of too non-centric loading. In addition, the stiffness of the I-beams is too low resulting in considerable deformations at mid spans – especially for the supporting I-beam used for the load cells. Hence, after the first test, an additional support was made at mid-span of the I-beam used for load measurements. The additional support is seen in Figure 4.

Deformation measurements were made at the supports and centrally on each roof unit during the initial loading. The centrally placed gauges were removed before expected failure load was reached. The positioning of deformation gauges, load cells, and the air cushion is sketched in Figure 4.
Figure 5. As indicated on the sketch, there were two adjoining beams at the connection between two elements. The load on two such beams was measured by one load-cell.

Figure 5. Sketch illustrating the placement of the air cushion (indicated by “pressure”), supports, deformation gauges (dots), and load cells (arrows). The three roof units are designated A, B, and C.

Figure 6 shows a photograph illustrating the test set-up after failure. The air cushion is visible between the roof unit and the floor.

Figure 6. Photograph taken after failure of a roof unit. The air cushion is visible between the roof unit and the floor. On the right the load cells placed at every beam is seen.
The test method was improved in the later tests as a result of experience gained when testing the first units. Hence, not exactly the same method was used in every test. Table 2 gives the basic improvements and chances throughout the test programme.

### Table 2. Table presenting the main changes during the test programme.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Unit combination</th>
<th>Improvements/changes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2:5-1</td>
<td>– Wood-wool lightweight building board is removed in bands of app. 30 cm at supports and at mid-span for deflection measurements &lt;br&gt; – 3 deflection gauges placed on centre beam in each unit &lt;br&gt; – Supporting steel I-beam was unstable at peak load</td>
</tr>
<tr>
<td>2</td>
<td>1:9-8</td>
<td>– All wood-wool lightweight building boards are removed &lt;br&gt; – Additional support at mid-span of supporting steel I-beam &lt;br&gt; – Additional pressure measurement to check load measurements in the air cushion</td>
</tr>
<tr>
<td>3</td>
<td>1:9-7</td>
<td>– A saw cut is made through the transverse head beam at the supports between every longitudinal beam to prevent misleading interpretations of reaction load and the load carried by each beam &lt;br&gt; – Tests were continued until failure of 2 beams (at least)</td>
</tr>
<tr>
<td>4</td>
<td>3:9-6</td>
<td>– A puncture appeared in the air cushion, the perforation was repaired for the remaining tests</td>
</tr>
<tr>
<td>5</td>
<td>1:9-4</td>
<td>– Deflection gauges were increased from 3 to 9 (3 middle beams of each unit)</td>
</tr>
<tr>
<td>6</td>
<td>1:9-2</td>
<td>– Supporting steel I-beam became unstable at peak load</td>
</tr>
</tbody>
</table>

### Results

The bending stiffness of each roof unit was predicted using the bending stiffness of each single beam. The bending stiffness was determined from assumptions of:

- no connection between beams and sheathing  
- rigid connection between beams and sheathing.

These two cases represented the bounds for the strength and stiffness of the units. In the case of no connection between beams and sheathing, the bending stiffness was easily determined from Equation 2.

\[
EI_{\text{beams}} = \frac{1}{12} b_w h_w^3 \sum_{i=1}^{n} E_i
\]  

(2)

where \(b_w\) is the width of the beams, \(h_w\) is the depth of the beams, and \(E_i\) is the experimentally determined modulus of elasticity of beam \(i\) in the roof unit. The bending stiffness of the sheathing is negligible in this case.

Considering a fully rigid connection between beams and sheathing the bending stiffness was determined according to the method described in (Larsen, 1999):

\[
A_i = b_w h_w + t_c b_{\text{ef}} \frac{E_c}{E_i}
\]

\[
e_i = \frac{t_c b_{\text{ef}} (h_w + t_c) E_c}{2 A_i} \frac{E_i}{E_c}
\]

\[
EI_{\text{rigid}} = \sum_{i=1}^{n} \left( E_i \left( \frac{1}{12} b_w h_w^3 + \frac{E_c t_c b_{\text{ef}} (h_w + t_c)^2}{4 E_i} \frac{E_i}{E_c} \right)^2 - A_i e_i^2 \right)
\]  

(3)

where \(b_w\) is the width and \(h_w\) is the depth of the beams. The modulus of elasticity in bending is \(E_i\) (the \(i\) referring to the beam number). For the sheathing, \(t_c\) is the thickness, \(E_c\) is the axial stiffness (12 GPa is used), \(b_{\text{ef}}\) is effective width (depending of position in unit).

The bending stiffness of the roof units was determined experimentally. Results from predictions and tests are shown in Stang (2002).
The maximum bending moment, $M_u$, at mid-span of the units can be determined from:

$$M_u = \frac{1}{2} R_u \left( l - \frac{c}{2} \right)$$

(4)

where $R_u$ is the maximum reaction force (reaction capacity) measured at one support, $l$ is the span between supports, and $c$ is the width of the air cushion used for loading.

Assuming no connection between sheathing and beams (and excluding the contribution from the sheathing), the predicted reaction capacity, $R_{pred}$, for the single beams can be determined from Equation 5.

$$R_{beam\,pred} = \frac{2 f_{b,i} E I_{beam,i}}{E_i \left( l - \frac{c}{2} \right) \left( \frac{1}{2} h_w \right)}$$

(5)

where the strength, $f_{b,i}$, of beam number $i$ is predicted from the relation given in Equation 1.

For a rigid connection between beams and sheathing, the corresponding predicted reaction capacity can be predicted from Equation 6.

$$R_{rigid\,pred} = \frac{2 f_{b,i} E I_{rigid,i}}{E_i \left( l - \frac{c}{2} \right) \left( \frac{1}{2} h_w + e_i \right)}$$

(6)

The reaction capacities predicted in Equations 5 and 6 represent the upper and lower bounds for the single beams in a unit. The strength is less for a rigid connection between beam and sheathing when the beam is positioned on the side of a unit than when positioned in the centre. This is due to a smaller effective width of the sheathing. Table 3 gives the sum of reaction forces at failure (1$^{\text{st}}$ and 2$^{\text{nd}}$) in the tests.

<table>
<thead>
<tr>
<th>Unit combination</th>
<th>1$^{\text{st}}$ failure [kN]</th>
<th>2$^{\text{nd}}$ failure [kN]</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-5-1</td>
<td>62.4</td>
<td>-</td>
<td>Unreliable load measurements (see below).</td>
</tr>
<tr>
<td>1-9-8</td>
<td>61.8</td>
<td>-</td>
<td>Unreliable load measurements (see below). Failure of 2 beams in unit 8 (beams not adjacent)</td>
</tr>
<tr>
<td>1-9-7</td>
<td>52.0</td>
<td>57.6</td>
<td>Failure in unit 7, unit 9 should be weaker. First failure in weakest beam in unit 7. Second failure in adjacent beam. Total load-carrying capacity higher at second failure than at first failure</td>
</tr>
<tr>
<td>3-9-6</td>
<td>73.6</td>
<td>73.0</td>
<td>Failure in unit 6 and later unit 3 but unit 9 contain weakest beams. First and second failure occurs in different units. Total load-carrying capacity almost similar at second failure and at first failure</td>
</tr>
<tr>
<td>1-9-4</td>
<td>66.6</td>
<td>71.6</td>
<td>Failure in unit 4, unit 9 should be weaker. Almost the same strength of beams in unit 4. Second failure in adjacent beam. Total load-carrying capacity higher at second failure than at first failure</td>
</tr>
<tr>
<td>1-9-2</td>
<td>72.7</td>
<td>61.0</td>
<td>Failure in unit 9, the weakest unit in the set. First failure simultaneously in two adjacent beams. Second failure in adjacent beam (3$^{\text{rd}}$ centre beam). Total load-carrying capacity smaller at second failure than at first failure</td>
</tr>
</tbody>
</table>

The results of the tests are shown in Figure 7, where the measured reaction force in each beam is shown at 1$^{\text{st}}$ and 2$^{\text{nd}}$ failure. In the same Figure, the predicted reaction force capacity is shown for the beams, based on individual measurements of modulus of elasticity. In the tests, two adjacent beams along the edge of a unit shared the load cell (beams 5 and 9). The load on the two beams (indicated by $a$ and $h$) is assumed to be the same corresponding to half the measured load on each of the beams.
Figure 7. Measured reaction forces in the 6 tests shown together with predicted reaction force capacity. The slender, dark columns represent the load measured at failure, whereas the fat columns represent predictions (upper bound is for rigid connection between beam and sheathing, lower bound for beams alone).

**System effects**

In this investigation, a unit was a system of 5 beams, and a set was a system of 3 units (thus a system of 15 beams). Considering a system, the first failure was assumed to occur at the weakest beams, hence the strength of a unit or set was determined from the predicted reaction force capacity according to Equations 5 and 6 for the beam with the lowest stiffness.

In the analysis of units the load on the two edge beams ("1" and "5") was less than the load on the beams in the middle ("2", "3", and "4"). This was taken into account by assuming the load on these beams to equal half the load on the centre beams. The principle is shown in Equation 7.

\[
R_{\text{unit},i}^{\text{sys eff}} = \frac{1}{4} (R_{\text{max}})_{\text{min}} + 3 \cdot (R_{\text{max}})_{\text{min}} + \frac{1}{2} (R_{\text{max}})_{\text{min}} = 4 \cdot (R_{\text{max}})_{\text{min}}
\]

where \((R_{\text{max}})_{\text{min}}\) refers to the lowest value of the predicted maximum reaction forces for the 5 beams in unit number \(i\). Prediction of the strength of a set is determined according to:

\[
R_{\text{set},i}^{\text{sys eff}} = \frac{1}{3} (R_{\text{max}})_{\text{min}} + \frac{1}{3} (R_{\text{max}})_{\text{min}} + \frac{1}{3} (R_{\text{max}})_{\text{min}} = 1 (R_{\text{max}})_{\text{min}}
\]
\[ R_{\text{set},ijk}^{\text{syseff}} = 3 \cdot \min \left( R_{\text{unit},i}^{\text{syseff}}, R_{\text{unit},j}^{\text{syseff}}, R_{\text{unit},k}^{\text{syseff}} \right) \]  \hspace{1cm} (8)

where \( R_{\text{unit},i}^{\text{syseff}} \) is the predicted reaction force capacity of unit number \( i \) according to Equation 7.

An alternative analysis can be made using the 5-percentile value for the beam strength measurements (the distribution of strength is shown in Figure 3). For comparison, Figures 8 and 9 show the results of both minimum beam strength and the characteristic strength of the grading. The predicted results are shown together with the reaction force capacity determined in the tests. The predictions are presented as a band where the upper bound represents a rigid connection between beam and sheathing and the lower bound represents the case of no interaction between sheathing and beam.

![Figure 8](image-url)

Figure 8. The dark grey area represents predicted reaction force capacity of units assuming that the weakest beam in the unit defines the strength of the unit. The light grey area represents the reaction force capacity of the unit if the 5 percentile of the strength defines the strength of the unit. The large dots represent reaction force capacities as determined in tests.

![Figure 9](image-url)

Figure 9. The dark grey area represents predicted reaction force capacity of units assuming that the weakest beam in the unit defines the strength of the unit. The light grey area represents the reaction force capacity of the unit if the 5 percentile of the strength defines the strength of the unit. The large dots represent reaction force capacities as determined in tests.
Figure 9. The dark grey area represents predicted reaction force capacity of 3 units assuming that the weakest beam in the set defines the strength of the set. The light grey area represents the reaction force capacity of the set if the 5 percentile of the strength defines the strength of the unit. The large dots represent reaction force capacities as determined in tests.

**Discussion**

**Beam stiffness and strength**

Bending stiffness of the beams used was determined in a standard 4 point-bending test according to EN408 providing a mean value of the stiffness taken over a length of about 1 meter at midspan of the beams. Unfortunately, the test method does not provide information of the position of weak sections or the stiffness along the whole beam. The strength results were also determined according to EN408 - a 4-point bending test applying a constant moment along approximately 1 m at mid-span of the beam. At mid-span, where the bending moment is large, the moment distribution is close to the parabolic distribution in the tests where a uniform surface load is applied, see Figure 10. Hence, even though different types of loading are applied, the results from the unit tests should be comparable to the results from the strength tests of the single beams.

The relation between bending stiffness and bending strength is different from other investigations, see Figures 2 and 3. There are, however, no indications that the results in the present investigations were affected by systematic errors. Therefore, despite the very few results, the relation between bending stiffness and bending strength determined in this study was considered superior for estimating the bending strength of the beams in the units from their bending stiffness.

**Unit strength**

Combinations of beams in the units are predefined according to measured stiffness (estimated strength) of the beams. Therefore, the analyses are deterministic rather than statistical.

The experimental test procedure of unit strengths changed considerably during the first three tests. Therefore, comparison between the first results and the last results should be made with care. The measured strength of units and sets are presented chronologically to show this effect of changes in test set-up. The results show a clear improvement in (apparent) load-carrying capacity for the last tests.

The unit strength shown in Figure 8 is compared with predicted unit strength based on the two bounds for rigid connection and no connection between beams and sheathing. The bounds are based on the weakest beam according to predicted strength of the single beams in each unit. The test results are all within the predicted bounds. In general, prediction and tests show the same trends regarding unit strength. Another strength criteria is the characteristic strength according to the grading (K18) of the beams. The characteristic strength of the units is less than that of any of the test results.

**Strength of sets of 3 units**

The test results presented in Figure 9 as a set of 3 units were also within the predicted strengths. The model predictions according to the weakest beam in the set were controlled by one beam in element 9 in the entire test except the first test. The test results, however, showed that this unit was stronger than units 3, 4, 6, 7, and 8. Hence, the predicted strength of unit 9 is...
most likely too low. This is partly confirmed in Figure 8 where the ratio between measured strength and predicted strength was higher for unit 9 than for the other units. A result of the underestimated strength of unit 9, an underestimation of the predicted strength of sets containing unit 9 was found in all tests. The characteristic strength of the sets was clearly underestimating the test results.

**Failure modes and load sharing**

Evaluation of the test results shown in Appendix A the results show that, in some cases, the predicted strength does not represent the actual strength very well. Some beams failed earlier than predicted and others were apparently stronger than expected. Therefore, it is difficult to conclude whether weak beams were in fact able to transfer load to the stronger beams. Nevertheless, the results from tests 3 to 6 gave some indications that first beam failure did not lead to failure of all 3 units. In tests 3, 4, and 5 first failure corresponded to failure of one beam with residual strength. The total load on the 3 units was higher at second failure than at first failure. Second failure in tests 3 and 5 occurred in beams adjacent to first failed beam. In test 4, second failure was difficult to locate. Several beams in two different units failed at the same time. In test 6, two beams failed at the same time at first failure. This indicates a brittle failure mode, which is confirmed by the lack of residual strength in one of the beams. Some residual load-carrying capacity was left in the units after failure of two adjacent beams, but the load capacity was less than before. Contrary to the load applied in the tests, loading by snow or furniture would lead to progressive collapse because the load did not decrease with deformations.

Only 7 units were tested until failure, though 9 units were built. The remaining 2 units have not been tested, but since they are the strongest units both according to estimated strength and according to the tests, the system effects of these units will be little. Also, the probability of brittle failure of the remaining units is large because of the high strength of the beams (like failure of unit 9 in test 6).

**Conclusion**

A major test programme was carried out to investigate system effects in prefabricated stressed skin-elements. A total number of 60 beams were used in the investigation. Every beam was tested to obtain the edgewise-bending stiffness. The bending strength of 15 of the 60 beams was determined. A linear regression of the stiffness and strength results found a smaller ratio between bending strength and bending stiffness than the ratio found in other projects.

The remaining 45 beams were built in 9 units containing 5 beams each. The locations of the single beams were predefined. The bending stiffness of the 9 units were determined. Unfortunately, the rigidity of loading system was insufficient to provide sound results. Nevertheless, the measured stiffness was in agreement with estimations of upper and lower bounds for unit stiffness based on the measured beam stiffness.

The strength of units was determined in a set of three adjoining units. Load was applied by a large air cushion covering the three units. Load cells placed at the supports of the single beams measured the load applied to the single beams. Changes made to the test set-up during the first tests made it difficult to draw conclusions from the first two tests. The remaining tests include tests determining the strength of 5 units. Hence, the experimental results determined were based on very few tests and no strong conclusions should be drawn from this investigation. However, some important observations were made.

Test results show that failure of one beam occurred at load levels below maximum load carrying capacity. This means that first failure did not lead to failure of the whole system. The beam at failure generally carried a small load after failure. This means that the beams had a residual strength after first failure. The load level was increased in the adjacent beams after first failure.

Due to few tests on bending strength very little information is available on the relation between stiffness and strength. Since the bending stiffness is the only parameter used for predicting the bending strength also limited accuracy is to be expected. Bearing this in mind, it is difficult to state that weak beams carry less load than adjacent stronger beams. However, a faint tendency can be noted.
Acknowledgements

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The experiments on system effects were performed at Lund University. The tests were conducted as part of a student project carried out by Catharina Kockmann, Germany, during the fall of 2000. Catharina did a remarkable job running the large test programme. Her work is greatly appreciated.

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