5.5. Load Test Procedure

A uniform load configuration was used for the load tests. An air bag, placed on the top surface of the slab, was used for this purpose, and the load was applied by gradually increasing the pressure in the air bag. The air bag has a capacity of 20 psi in a fully constrained condition. The view of the test set-up is shown in Fig. 5-11. Each span was tested separately and in an attempt to prevent development of negative cracks into the adjacent span, crack inducers were placed along the interior supports of LSS1 and LSS2. The crack inducers were groves, approximated 0.5 in. deep, and made along the interior supports on the top surface of the concrete when it was still wet after the casting.

![Figure 5-11. Test set-up](image)

At the beginning of each load test, the tested span was preloaded with approximately 0.35 psi (50 psf) to settle the system and check the instrumentations. The slab was unloaded afterward and the loading was restarted and continued until a permanent set in the system was obtained. This permanent set can be observed from the presence of the nonlinear relation of the load versus mid-span displacement. Load increments of approximately 0.25 psi (36 psf) was applied with a pause, of approximately two minutes before any data recording, to allow the system to settle. When a permanent set had been noted, the system was once again unloaded completely. The loading was then restarted until failure or excessive deflection was obtained.

In the inelastic region where the stiffness of the slab had decreased considerably, displacement control loading was used with a displacement increment of approximately 0.5 in.
The test was terminated after 7 in. (LSS1) or 8 in. (LSS2) deflection was obtained.

5.6. Test vs. Analysis Results

Before the load tests, fine cracks through the depth of the concrete were observed on the sides of the slabs over the interior supports. During the load test, as the load was increased, flexural cracks developed within the tested span. In LSS2, because of the continuity of the steel deck over the interior support, cracks appeared in the adjacent span on the top surface of the slab. Maps of the cracks of LSS1 and LSS2 after the test are shown in Figs. 5-12 and 5-13. In Fig. 5-13, cracks indicated by x are cracks that were developed during the test of the adjacent span.

Figure 5-12. Map of cracks in LSS1

Figure 5-13. Map of cracks in LSS2
Flexural cracks in the positive moment regime appeared on the side of the slabs tend to turn horizontally approximately at the level of the top flange of the steel deck. This may indicate some separation of the slab portion (concrete cover) from the beam portion (concrete rib) of the concrete.

Load vs. mid-span deflection response from the tests and analyses of LSS1 and LSS2 are compared in Figs. 5-14 and 5-15. It can be observed from these figures, that the response of the second test of each LSS was relatively weaker and softer compared to the first. This may be caused by damage that occurred in the adjacent span (first test), so that less (horizontal) restraint was resulted. In LSS2, the occurrence of the negative cracks before the test on the second span may have increased this effect.

![Figure 5-14. Load vs. mid-span deflection of LSS1](image1)

(a) 1st test  
(b) 2nd test

![Figure 5-15. Load vs. mid-span deflection of LSS2](image2)

(a) 1st test  
(b) 2nd test
Predicted responses using the iterative method, as shown in Figs. 5-14 and 5-15, show reasonable agreement to those of the tests, particularly the first test of each slab. In terms of the slab strength, the direct method also shows relatively good agreement to the test results. The SDI-M method, however, predicted rather low strength (very conservative). This is due to the very low values of the reduction factor, R, based on the required anchorage forces. These were 0.545 and 0.447 for LSS1 and LSS2, respectively. Finally, a summary of the maximum test load capacity and permissible load based on the allowable deflection is given in Table 5-3.

Table 5-3. Summary of maximum test load and permissible load based on allowable deflection

<table>
<thead>
<tr>
<th>slab #</th>
<th>ultimate load capacity (psf)</th>
<th>load at allow. deflection *) (psf)</th>
<th>test load / 50 ultimate load capacity</th>
<th>load at allow. deflection</th>
<th>test load / 150 ultimate load capacity</th>
<th>load at allow. deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>LSS1a</td>
<td>621</td>
<td>245</td>
<td>12.42</td>
<td>4.90</td>
<td>4.14</td>
<td>1.63</td>
</tr>
<tr>
<td>LSS1b</td>
<td>559</td>
<td>210</td>
<td>11.18</td>
<td>4.20</td>
<td>3.73</td>
<td>1.40</td>
</tr>
<tr>
<td>LSS2a</td>
<td>498</td>
<td>163</td>
<td>9.96</td>
<td>3.26</td>
<td>3.32</td>
<td>1.09</td>
</tr>
<tr>
<td>LSS2b</td>
<td>455</td>
<td>121</td>
<td>9.10</td>
<td>2.43</td>
<td>3.03</td>
<td>0.81</td>
</tr>
</tbody>
</table>

*) based on L/360

From the above table, it can be noted that for LSS2, the permissible loads based on the allowable deflection are relatively low compared to those of typical span slabs and LSS1. Therefore, in the case of long span composite slab, it is important to check the deflection limit state.

5.7. Evaluation of the Floor Vibrations

Vibration tests on LSS1 and LSS2 were conducted prior to the load tests to determine the frequency of the fundamental mode of these slabs. For LSS1, the frequency of the fundamental mode was 10.63 Hz., and it was 8.13 Hz. for LSS2. Plots of the frequency spectra in terms of the normalized relative power vs. the frequency resulting from the tests are shown in Figs. 5-16 and 5-17.
Figure 5-16. Normalized relative power vs. frequency of LSS1

Figure 5-17. Normalized relative power vs. frequency of LSS2
Analytical calculations were made to determine if the frequencies satisfy the acceptance criteria for human comfort (Murray et al. 1997). Two criteria were considered in this case. LSS1 was classified as a footbridge with 6 ft effective width. The estimated peak acceleration was 3.13% g. This estimated peak acceleration is higher than the specified value of 1.5% g, and thus the slab can not be considered satisfactory. The effective width and the occupational load of the slab influence the vibration performance of the slab. For an effective width of 15 ft and for office and residential use of the same slab, the estimated peak acceleration becomes 0.31% g, which is lower than the maximum peak acceleration limit of 0.50% g. The slab stiffness requirement was also satisfactory (5.89 k/in, experimental, compared to the minimum requirement of 5.70 k/in). Therefore, in the later case, the slab can be considered satisfactory. These estimations, however, are rather approximate, and further investigation is necessary.

The vibration response of LSS2 was not as good as those of LSS1. The estimated peak acceleration for a footbridge condition is 10.2% g compared to the maximum peak acceleration limit of 1.5% g, and the experimental slab stiffness was 2.48 k/in which is below the minimum required stiffness of 5.7 k/in. For the condition with an effective width of 15 ft for office and residential purpose, the estimated peak acceleration is 0.95%, and again is greater than the specified value of 0.50% g. Further evaluations are necessary based on these preliminary evaluations of the composite slabs.

5.8. Proposed Detail Connection

The total depth of composite floor system using steel deck profiles as described in this study is relatively shallow. In comparison with the 3 in. trapezoidal deck profile using a same thickness of concrete cover, profiles 1 and 2 will result in 3 in. and 1.5 in., respectively, of additional slab depth. Therefore, typical beam to girder connection for composite slabs with regular span length can be used without adding any significant height to most structures. However, should this additional structure height be objectionable, it can be reduced or eliminated by using a beam to girder connection as shown in Fig. 5-18.
5.9. Concluding Remarks

A study on long span composite slab systems has been presented and two steel deck profiles have been investigated. The study, verified by experimental tests, shows very promising results on the use of relatively slim slabs (8.5 in. and 7 in. total slab depth), with almost the same concrete volume or weight as of the typical span slabs. With the proposed beam to girder connection, the slab-beam depth may be reduced to a total floor depth comparable to currently used floors. This feature of slab depth and weight promise potential advantages over the slimflor systems that are now used in European countries.

The design method for the development of the deck profile by generating charts of the steel deck weight vs. the span length, and the analytical methods for the prediction of the composite strength and stiffness of the slab were shown to be good tools. These methods of analyses are very promising for the development of new deck profiles before any experimental tests. They can also reduce the number of full-scale tests needed.

Permissible loads based on the deflection limit state of the service phase may become the governing limit state in the case of long span composite slabs. This limit state rarely governs the design in typical span slab systems. Therefore, in the case of long span slab systems, both the construction (non-composite) and service (composite) phases have to be evaluated carefully.

Results of the evaluation of floor vibrations suggest further study be required to improve the performance of the slabs with respect to the floor vibration criteria. A deeper slab thickness with a little sacrifice in span length could be considered to give higher slab stiffness, which may improve the vibration characteristics.