Developments in Long Span Composite Slabs

BUDI R. WIDJAJA and W. SAMUEL EASTERLING

INTRODUCTION

Composite slabs constructed using cold-formed steel deck are by far the most common forms of floor used in steel-framed buildings in the United States. Indeed it is largely due to the attributes that steel deck provides that make steel-framed buildings as economically competitive as they are.

The maximum unshored span, using composite deck profiles manufactured in the U.S., is approximately 15 ft. However, the use of 16 ga deck and lightweight concrete is required to achieve 15 ft unshored clear spans. This number depends on the deck cross section as well as the concrete thickness and unit weight. Typically, deck spans in the range of 8-10 ft are used. The choice of unshored construction is common because of the savings in construction cost and time.

If the typical span lengths can be increased by a factor of say 1.5 to 2, significant cost savings can be expected because of the reduction in the number of filler beams and their connections to the girders. The cost of the steel deck will increase, however the savings realized by using fewer filler beams will be greater than this increase. The advantage stems largely from cost savings in fabrication and erection of fewer members and connection of those members.

The potential advantages identified above have motivated the research reported in this paper. The primary purpose of this paper is to initiate a discussion and consideration of using composite deck profiles deeper than 3 in. in the U. S. market. The profiles identified in the research are used to illustrate general performance and cost considerations. A self-imposed constraint during the research was to attempt to identify new deck configurations that do not result in significant increases in the slab weight and depth as compared to those commonly used. Additionally, the profiles identified should permit current deck manufacturing and erection procedures.

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Background

Notable research in the area of long span slabs has been carried out by Ramsden (1987), Hillman and Murray (Hillman, 1990; Hillman and Murray, 1990 and 1994), and by personnel at the Steel Construction Institute in the United Kingdom (Mullett, 1992; Lawson and Mullett, 1993; Lawson, Mullett, and Rackham, 1997). The reader should note that numerous research teams have worked on various aspects of the *Slimfloor* system, which is the subject of work referenced to members of the Steel Construction Institute. A complete literature review is not included here; rather only a brief introduction to the research is included. Further, several deep deck profiles have been developed for use in slimfloor and non-slimfloor construction.

Ramsden (1987) conducted a study on two prototype deck profiles that can span up to 24 ft. The prototypes, illustrated in Figure 1, have holes in the web to ensure the composite action between the deck and the concrete. The second prototype is an improved version of the first one. Because of the shape of the profile, the concrete slab is virtually a solid slab with a thickness of 5 in. to 6 in., which is disadvantageous because of it selfweight. There is no mention in the paper whether shoring of the slab during construction was provided. However, based on the cross section and span it almost assuredly was.

An innovative lightweight floor system was developed and reported by Hillman and Murray (Hillman, 1990; Hillman and Murray, 1990 and 1994). The floor system developed was not only lightweight but also able to span up to 30 ft without

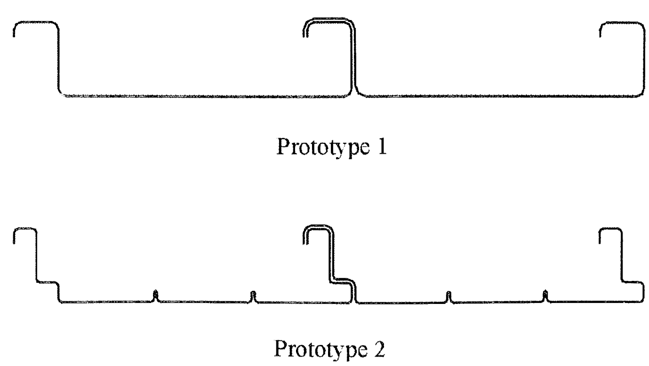


Fig. 1. Prototype 1 and Prototype 2 (Ramsden, 1987).

intermediate beams. The floor system is shown schematically in Figure 2.

The *slimfloor* system utilizes deep deck sections that are supported on unsymmetrical H-shapes that are generally fabricated by adding a cover plate to the bottom flange. Using lightweight concrete, the typical deck section can span up to 6 m (approximately 19.7 ft). A schematic view of the *slimfloor* system is shown in Figure 3.

DECK DESIGN

In the current study, two 16 ga, deep steel deck profiles were investigated. The first profile, referred to as Profile 1, has a 6 in. rib height. The profile is currently not available as a rolled product so it was designed and manufactured by a press-brake process for this project. Because of this, the length of the deck was limited to 25 ft. For long span slab specimens, the length is only enough for a single span configuration. The second profile, Profile 2, is a currently available roof deck section whose stiffness, as discussed later herein, satisfies the requirements for a long span slab in a double span configuration. This section was manufactured through a cold-rolling process. Cross sections of both profiles, along with a typical 3 in. composite deck profile, are shown in Figure 4. Note that neither Profile 1 or 2 incorporated embossments. This is because neither is currently available as *composite* deck profiles.

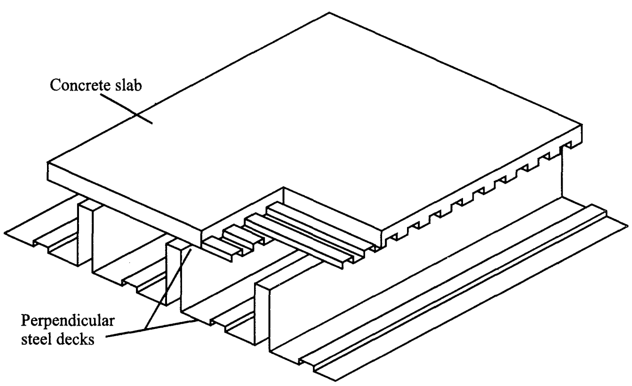


Fig. 2. Innovative light weight floor system.
(Hillman, 1990; Hillman and Murray, 1990 and 1994)

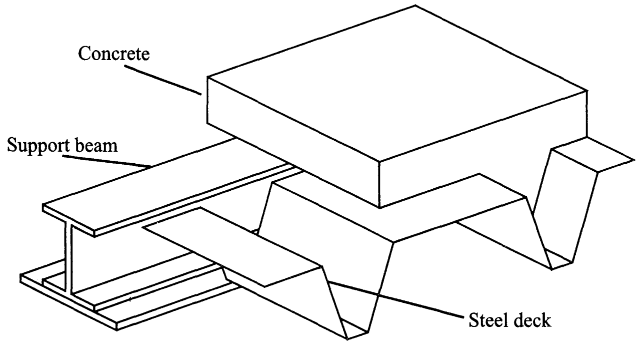


Fig. 3. Slimflor *system (Lawson et al., 1997).*

Two design phases have to be considered in the development of deck profiles, namely the *construction* (non-composite) phase and *service* (composite) phase. The construction phase considers the strength and stiffness of the steel deck as a working platform that is subject to concrete self-weight and construction loads. This phase is important in the determination of the required deck stiffness. It is shown later that when a long span system is involved, the deflection (stiffness) limit state generally controls the design.

In the service phase the composite steel deck-concrete slab is subject to occupancy loads. Studies on composite slabs with typical span lengths (Terry and Easterling, 1994; Widjaja and Easterling, 1995, 1996b, and 1997) revealed that actual slab strengths are high compared to the standard design live loads (50 to 150 psf). Results from 23 slab tests are summarized in Table 1. All the test specimens for which results are reported in Table 1 were constructed with spans less than 12 ft. For each test the maximum test load, in psf, and the load at a deflection corresponding to span/360 is given. Additionally, two sets of load ratios are shown. These are ratios of load to either 50 or 150 psf, which represent a range of design live loads. The ratios are shown for both the maximum test load and the load at a deflection corresponding to span/360. In all cases the load and deflection capacity of the slab exceeded the demand based on loading of 50 or 150 psf.

These ratios suggest that the service (composite) phase would rarely control the overall design. However, this is not always the case for *long span* composite slabs as shown later by the analysis and test results. For long span slabs, either the construction or service phase may govern the typical design.

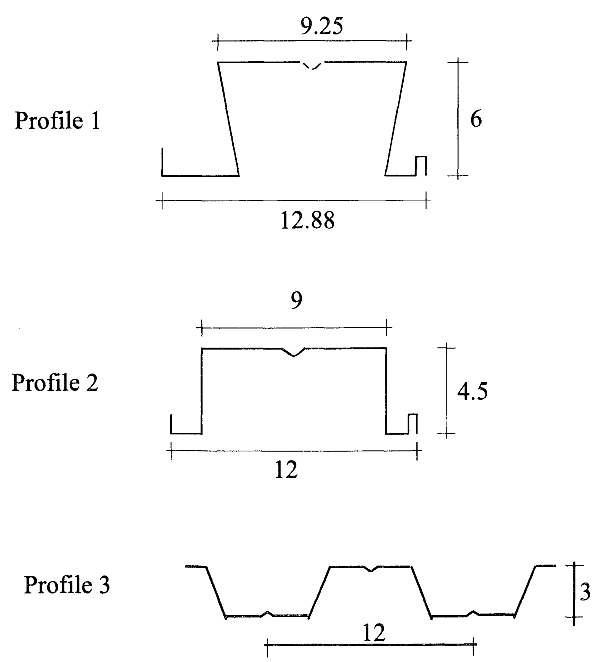


Fig. 4. Steel Deck Profiles.

Table 1. Ratios of Maximum Test Load and Load Based on Allowable Deflection for 50 and 150 psf Design Live Loads						
Slab #	Maximum Test Load (psf)	Load at Allow. Deflection* (psf)	Maximum Test Load/50 psf	Load at Allow. Deflection*/50 psf	Maximum Test Load/150 psf	Load at Allow. Deflection*/150 psf
1	730	345	14.60	6.91	4.87	2.30
2	700	326	14.00	6.52	4.67	2.17
3	600	238	12.00	4.76	4.00	1.59
4	600	223	12.00	4.47	4.00	1.49
5	490	310	9.80	6.20	3.27	2.07
6	590	316	11.80	6.32	3.93	2.11
7	375	301	7.50	6.01	2.50	2.00
8	490	320	9.80	6.41	3.27	2.14
9	900	374	18.00	7.49	6.00	2.50
10	900	388	18.00	7.76	6.00	2.59
11	750	352	15.00	7.04	5.00	2.35
12	870	418	17.40	8.36	5.80	2.79
13	480	399	9.60	7.98	3.20	2.66
14	500	389	10.00	7.78	3.33	2.59
15	1017	407	20.34	8.14	6.78	2.71
16	1185	466	23.70	9.32	7.90	3.11
17	565	301	11.30	6.02	3.77	2.01
18	368	303	7.36	6.07	2.45	2.02
19	523	396	10.46	7.93	3.49	2.64
20	523	445	10.46	8.91	3.49	2.97
21	467	229	9.34	4.57	3.11	1.52
22	494	206	9.88	4.11	3.29	1.37
23	507	246	10.14	4.92	3.38	1.64

* based on span/360

Construction Phase

As previously mentioned, the construction load phase considers the strength and stiffness of the steel deck profile subjected to the weight of fresh concrete and building code specified construction load. For typical span lengths of 8-10 ft, strength limit states will tend to govern the deck design. However, deflection limit states will tend to govern the design for span lengths above approximately 15 ft.

The deflection limit of span/180, as prescribed in the Steel Deck Institute (SDI) *Composite Deck Design Handbook* (Heagler, Luttrell, and Easterling, 1997) was used in the calculations to design the profiles used in this study. The SDI also recommends an absolute upper limit of 0.75 in. for deflection due to fresh concrete. This limit was not used however, because it was felt to be too restrictive for long spans.

As an aid in designing the profiles, plots of steel deck weight versus span length were generated for both single span and double span configurations, as shown in Figures 5 and 6. The plots were generated for the two shapes shown in Figure

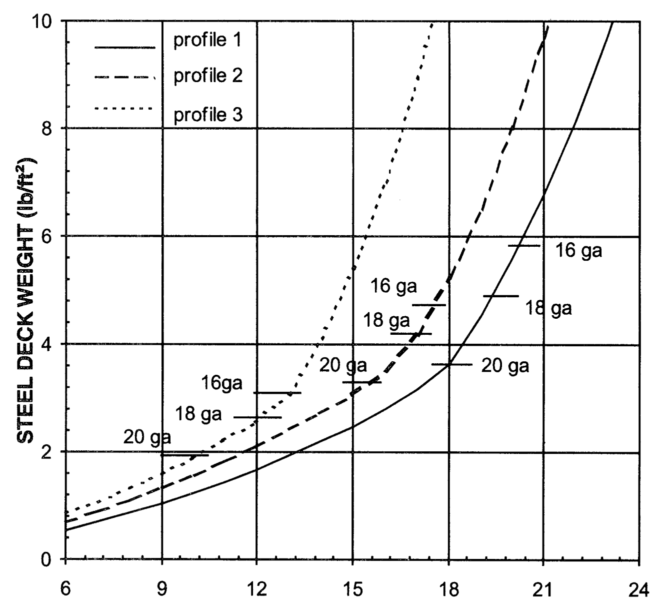


Fig. 5. Steel deck weight vs. span length for single span systems.

Table 2. Section Properties of Profiles 1, 2 and 3						
Profile #	Thickness (in)	Area (in^2/ft)	Inertia (in^4/ft)	y_p (in)	Deck Weight (psf)	Slab Weight (psf)
1	0.056	1.694	10.54	3.197	5.8	61.8
2	0.056	1.380	4.70	2.610	4.8	48.6
3	0.056	0.895	1.49	1.500	3.1	51.4

4, as well as a typical 3 in. deep composite floor deck (Profile 3), assuming a 2.5 in. normal weight concrete cover over the top of the deck. The steel deck weight was calculated from the deck thickness corresponding to the required moment of inertia, based on a given span.

Weight was selected as a key design parameter given its relationship to cost. Additionally, consideration of the weight of a single sheet of deck is important because of its influence on the manner used and effort required by deck erecting crews.

Comparing profile properties in Table 2, one observes that the steel deck moment of inertia of Profile 1 is approximately 7 times higher than that of Profile 3, which corresponds to an ability to span 1.6 ($=\sqrt[4]{7}$) times further. However, the steel deck self-weight for Profile 1 is almost double that of Profile 3. A 20 ft long piece of Profile 1 deck with a 12 in. cover width weighs 116 lb, which can be handled easily by two deck erectors.

For Profile 2, the increase of the moment of inertia is about 3 times that of profile 3, and it corresponds to an ability to span 1.3 times further. The total weight of the slab, for the same 2.5 in. concrete cover above the rib, is slightly lighter than the slab with Profile 3 steel deck.

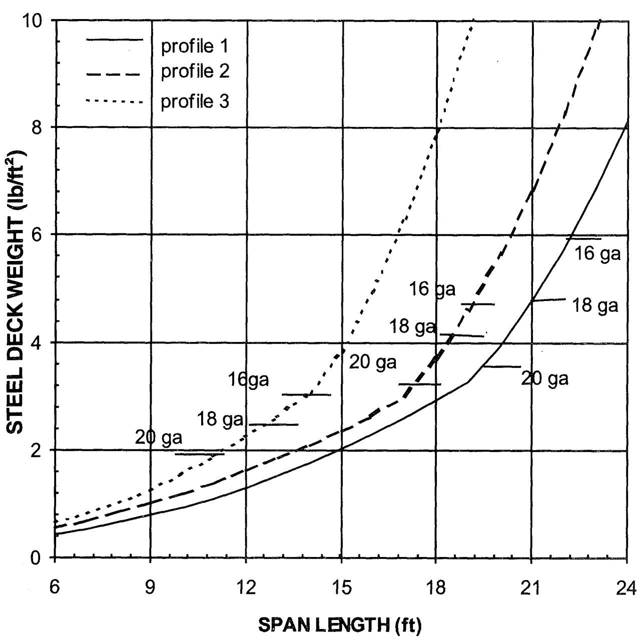


Fig. 6. Steel deck weight vs. span length for double span *systems.*

Service Phase

The load application in an experimental program is analogous to the *service phase* for a composite slab design. Predicted maximum test loads for composite slabs can be calculated using various techniques. Four techniques were used in this study, the iterative, direct, Steel Deck Institute-Modified (SDI-M), and finite element methods. Widjaja (1997) and Widjaja and Easterling (1996a) describe each of these in detail. Only the iterative and finite element methods provide incremental response histories of load vs. deflection for the slabs. In the SDI-M and direct methods, an idealized bi-linear response is generated using calculated load and an average composite moment of inertia to generate the elastic stiffness. Results from the iterative, direct and SDI-M methods are shown in figures that appear later in the paper.

The SDI-M method is a modified version of the SDI design procedure (Heagler et al., 1997). The modification pertinent to this study consisted of accounting for the affect of casting stresses in the determination of the first yield moment (Widjaja, 1997; Widjaja and Easterling, 1996a).

SPECIMEN DESCRIPTIONS AND TEST PROCEDURES

Long span slab 1 (LSS1) had a configuration of two single deck spans of 20 ft (center-to-center of supports) each and 1 ft cantilever as shown in Figure 7 (a). The total slab depth was 8.5 in. (2.5 in. concrete cover above the 6 in. deck rib). Six ¾ in. diameter, 8$\frac{3}{16}$ in. tall shear studs, spaced at 1 ft on center, were used at each end of the deck sheets. Therefore, there were 12 studs attached to the interior support beam and 6 each attached to the exterior support beams.

For long span slab 2 (LSS2), a two-span system was used with 20 ft span lengths (center-to-center of supports). The configuration is shown in Figure 7 (b). The total depth of the slab was 7 in. (2.5 in. concrete cover above the 4.5 in. deck rib). Six ¾ in. diameter, 6$\frac{3}{16}$ in. tall shear studs, spaced at 1 ft on center, were used along each of the three supporting beams.

No shoring was provided during construction of the slabs. In an attempt to prevent development of negative cracks into the adjacent span, control joints were placed along the interior

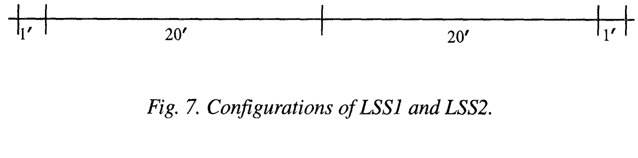


Fig. 7. Configurations of LSS1 and LSS2.

supports of LSS1 and LSS2. The control joints were grooves, approximately 0.5 in. deep, and made along the interior supports on the top surface of the concrete while it was still fresh.

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 Figure 8. Each span was tested separately. Note that the air bag has a nominal length of 10 ft. The bag was centered in the 20 ft span.

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 instrumentation. 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 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.

ANALYSIS AND TEST RESULTS

Deck deflection measurements resulting from concrete placement were made. The measured mid-span deflections of the steel deck were 0.695 in. and 0.685 in. for LSS1 and LSS2, respectively. These deflections were lower, in both cases, than the specified maximum deflection limit and lower than the SDI recommended value of 0.75 in. Recall that the limit of 0.75 in. was ignored in the design.

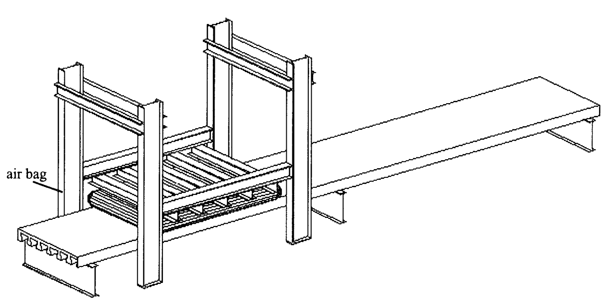


Fig. 8. Test set-up schematic.

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 Figures 9 and 10. In Figure 10, cracks indicated by x are cracks that were developed during the test of the adjacent span.

Flexural cracks in the positive moment regime appearing 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 versus mid-span deflection response from the tests and analyses of LSS1 and LSS2 are compared in Figures 11 and 12. It can be observed from these figures, that the response of the second test of each LSS was relatively weaker and more flexible compared to the first. This may be caused by damage that occurred in the adjacent span (first test), so that less (horizontal) restraint resulted. In LSS2, the occurrence of the negative cracks before the test on the second span may have increased this effect.

Predicted responses using the iterative method, as shown in Figures 11 and 12, 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,

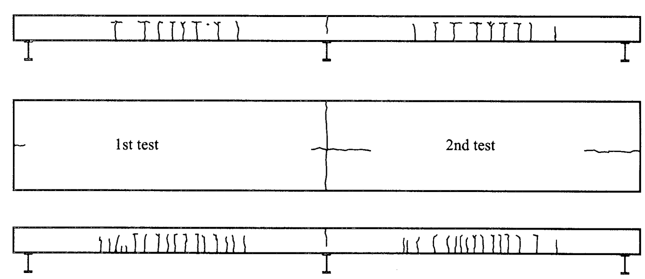


Fig. 9. Map of cracks in LSS1.

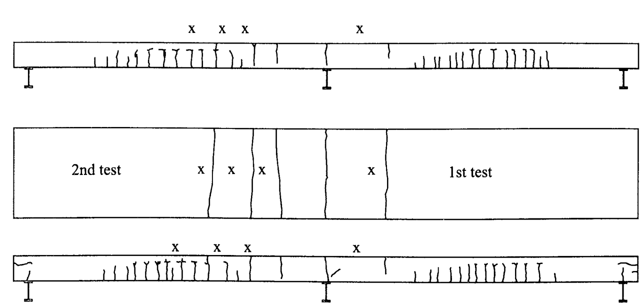


Fig. 10. Map of cracks in LSS2.

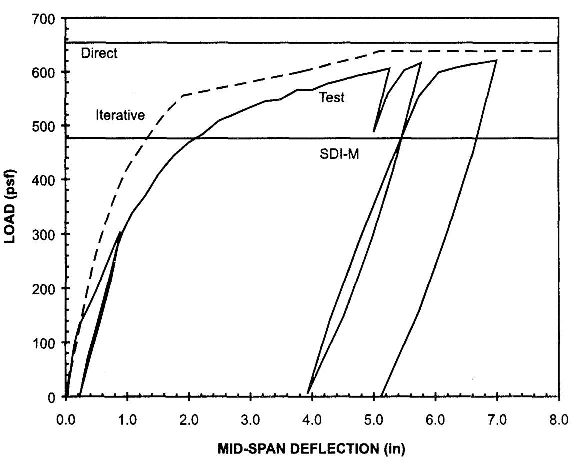
Table 3. Summary of Maximum Test Load and Permissible Load Based on Allowable Deflection						
Slab #	Maximum Test Load (psf)	Load at Allow. Deflection* (psf)	Maximum Test Load/50 psf	Load at Allow. Deflection*/50 psf	Maximum Test Load/150 psf	Load at Allow. Deflection*/150 psf
LSS1a	621	245	12.42	4.90	4.14	1.63
LSS1b	559	210	11.18	4.20	3.73	1.40
LSS2a	498	163	9.96	3.26	3.32	1.09
LSS2b	455	121	9.10	2.43	3.03	0.81
* based on L/360						

predicted rather low strength (very conservative). This is due to the very low values of the reduction factor, *R,* which is a function of the ratio of provided to required anchorage forces determined in the SDI-M method. These were 0.545 and 0.447 for LSS1 and LSS2, respectively. Finally, a summary of the maximum test load and permissible load based on the allowable deflection is given in Table 3.

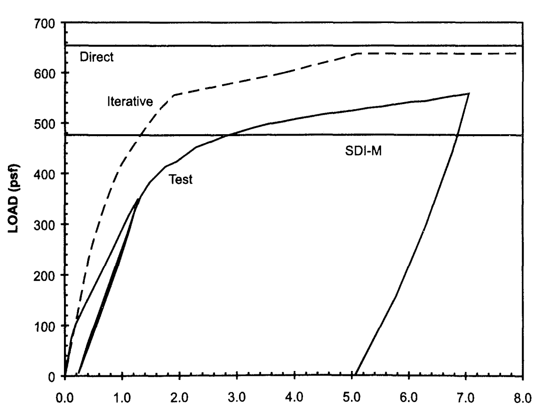
From Table 3, it can be noted that for LSS2, the permissible loads based on the allowable deflection are relatively low compared to those of typical span slabs and LSS1. Therefore, in the case of long span composite slabs, it is important to pay close attention to the deflection limit state. In general, all the results indicate that the profiles show real promise for composite deck applications and therefore warrant further development.

Preliminary Evaluation of the Floor Vibration Limit State

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

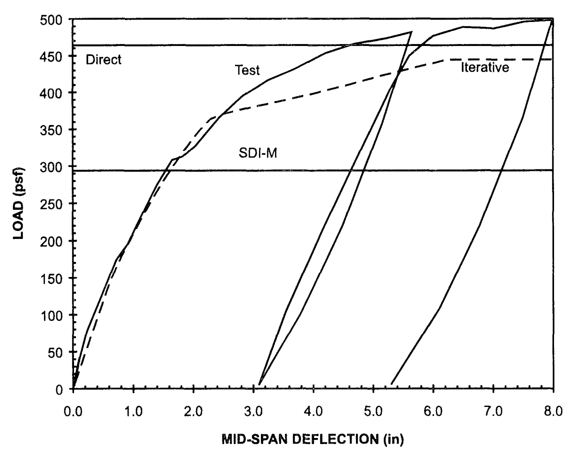


(a) 1st test

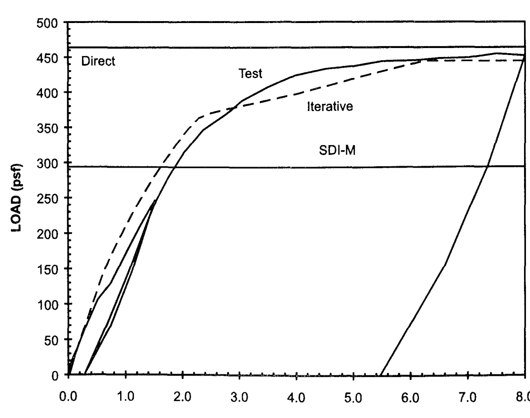


(b) 2nd test

Fig. 11. Load vs. mid-span deflection of LSS1.



(a) 1st test



(b) 2nd test

Fig. 12. Load vs. mid-span deflection of LSS2.

of the frequency spectra in terms of the normalized relative power versus the frequency resulting from the tests are shown in Figures 13 and 14.

Analytical calculations were made to determine if the frequencies satisfy the acceptance criteria for human comfort (Murray, Allen, and Ungar, 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 occupant load of the slab influence the vibration performance. 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 latter case, the slab can be considered satisfactory. These estimations, however, are rather approximate, and further investigation is necessary.

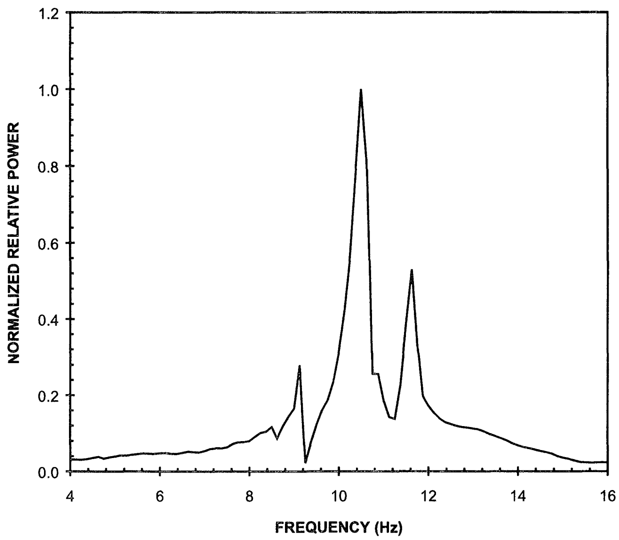


Fig. 13. Normalized relative power vs. frequency of LSS1.

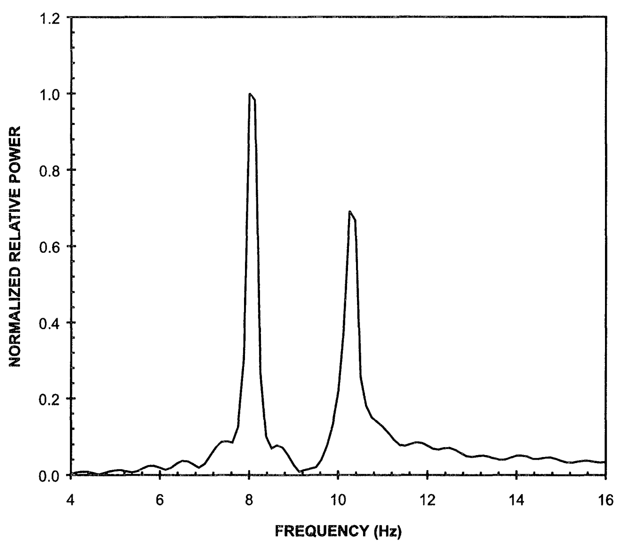


Fig. 14. Normalized relative power vs. frequency of LSS2.

The vibration response of LSS2 was not as good as that 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.

CONNECTION DETAIL

The increase in total depth of the composite floor system using steel deck profiles as described in this study may be an issue of concern. In comparison with the 3 in. trapezoidal deck profile using the 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 connections 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 similar to that shown in Figure 15 or Figure 16.

A preliminary study of the configuration shown in Figure 16 is currently underway at Virginia Tech. This connection,

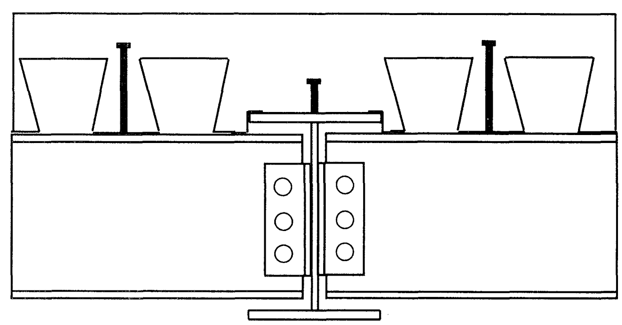


Fig. 15. Beam-to-girder connection to reduce slab-beam height.

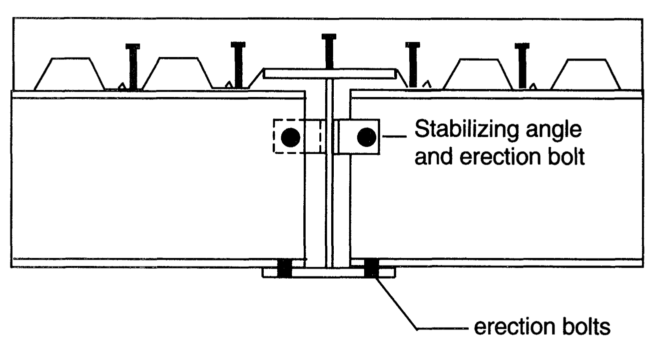


Fig. 16. Direct bearing beam to girder connection.

should it be shown to have adequate strength and stiffness, offers an attractive alternative to typical beam to girder connections, regardless if used with existing deck profiles or deep deck profiles. The overall depth of the floor system should be reduced by the difference in elevation between the top flanges of the beam and girder. It is suggested that this be one nominal section depth, thus on the order of 2 to 3 in. Thus, if the connection were used along with a typical 2 or 3 in. steel deck profile, then the result would be a decrease in the overall depth compared to using a framing connection in which the top flanges of the beam and girder are at the same elevation. If a deep deck profile were used, then the increase in depth due to the deck would be offset by the decrease in depth created by the beam to girder connection.

CONCLUDING REMARKS

A study on long span composite slab systems has been presented and two steel deck profiles have been investigated. The study, supported by analytical calculations and experimental tests, has shown very promising results for the use of long span deck profiles filled with almost the same concrete volume as used in typical deck profiles. Therefore, the span is increased without a significant increase in weight. With the proposed beam to girder connection, the floor depth may be reduced to one comparable with currently used floors.

The design method for the development of the deck profile by generating charts of the steel deck weight versus 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. 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.

In summary, the potential economic advantages offered by increasing the slab spans in steel-framed buildings, over those typically used, warrant further attention. This study has shown this to be true. The increase in slab span will result in competitive advantages for steel-framed construction over other alternative systems.

ACKNOWLEDGMENTS

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REFERENCES

Heagler, R. B., Luttrell, L. D., and Easterling, W. S. (1997), *Composite Deck Design Handbook*, Steel Deck Institute, Canton, OH.

Hillman, J. R. (1990), *Innovative Lightweight Floor System for Steel Framed Buildings*, M.S. Thesis, Virginia Polytechnic Institute and State University.

Hillman, J. R. and Murray, T. M. (1990), "Innovative floor systems," *Proc. 1990 National Steel Construction Conference*, Kansas City, MO, AISC, pp. 12/1-12/21.

Hillman, J. R. and Murray, T. M. (1994), "An innovative cold-formed floor system," *Proc. 12th. International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Ed.:W. W. Yu, pp. 513-521.

Lawson, R. M. and Mullett, D. L., (1993), *Slimfloor Construction Using Deep Decking,* Steel Construction Institute, Berkshire, UK.

Lawson, R. M., Mullett, D. L., and Rackham, J. W. (1997), *Design of Asymmetric "Slimflor" Beams Using Deep Composite Decking,* Steel Construction Institute, Berkshire, UK.

Mullett, D. L. (1992), *Slim Floor Design and Construction,* Steel Construction Institute, Berkshire, UK.

Murray, T. M., Allen, D. E., and Ungar, E. E. (1997), *Floor Vibrations due to Human Activity,* AISC and Canadian Institute of Steel Construction, Chicago, IL.

Ramsden, J. A. (1987), "A Light Gauge Structural Panel for Composite Flooring," *Composite Construction in Steel and Concrete*, Proceedings of an Engineering Foundation Conference, ASCE, Eds.:C. D. Buckner, I. M. Viest, pp. 374-386.

Terry, A. S. and Easterling, W. S. (1994). *The Effects of Typical Construction Details on The Strength of Composite Slabs*, Report No. CE/VPI-ST 94/05, Department of Civil Engineering, Virginia Polytechnic Institute and State University.

Widjaja, B. R. (1997), *Analysis and Design of Steel Deck-Concrete Composite Slabs,* Ph.D. Dissertation, Virginia Polytechnic Institute and State University, Blacksburg, VA.

Widjaja, B. R. and Easterling, W. S. (1995), *Evaluation of Composite Slabs Using Versa-Dek Profile,* Report No. CE/VPI-ST 95/02, Department of Civil Engineering, Virginia Polytechnic Institute and State University.

Widjaja, B. R. and Easterling, W. S. (1996a), "Strength and stiffness calculation procedures for composite slabs," *Proc. 13th. International Specialty Conference on Cold-Formed Steel Structures*. Ed.:W. W. Yu, pp. 389-401.

Widjaja, B. R. and Easterling, W. S. (1996b), *Vulcraft 3VLI Composite Deck Tests,* Report No. CE/VPI-ST 96/18, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA.

Widjaja, B. R. and Easterling, W. S. (1997), *Vulcraft 2VLI Composite Deck Tests,* Report No. CE/VPI-ST 97/09, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA.

Errata

Beam-Column Base Plate Design
Paper by RICHARD M. DRAKE and SHARON J. ELKIN
(1st Quarter, 1999)

On Page 35, revise Equations 44, 46, and 47 as follows:

$$f_p = \frac{T_u + P_u}{BY} \tag{44}$$

$$t_{p(req)} \geq 1.49c\sqrt{\frac{T_u + P_u}{BYF_y}} \tag{46}$$

$$t_{p(req)} \geq 2.11\sqrt{\frac{(T_u + P_u)\left(m - \frac{Y}{2}\right)}{BF_y}} \tag{47}$$

On Page 36, bottom of the left column, revise the last two lines as follows:

$$t_{p(req)} = 2.11\sqrt{\frac{(8.92\text{K} + 130\text{K})\left(5.24\text{ in.} - \frac{2.27\text{ in.}}{2}\right)}{(20.0\text{ in.})(36\text{ ksi})}} \tag{47}$$

= 1.88 in. *controls*

On Page 37:

In the left column, under Item 5, revise the solution of Equation 47 as follows:

$$t_{p(req)} = 2.11\sqrt{\frac{(17.3\text{K} + 87.6\text{K})\left(5.24\text{ in.} - \frac{2.27\text{ in.}}{2}\right)}{(20.0\text{ in.})(36\text{ ksi})}} \tag{47}$$

=1.35 in. *controls*

In the left column, under Item 6, 4th and 8th lines, revise the base plate size as follows:

Line 4: Select: Base Plate 1 3/8 × 14 × 1'-2

Line 8: Select: Base Plate 1 3/8 × 14 × 1'-2