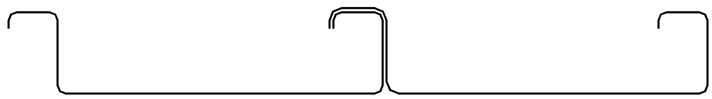
CHAPTER 5

LONG SPAN COMPOSITE SLAB SYSTEMS

5.1. General

The maximum span length of unshored single span composite slabs used in the U.S. based on available steel deck floor in the market is around 13 ft. The choice of unshored systems is very common because these systems can save construction cost and time. If the span length can be increased by a factor of, for example, 1.5 or 2, significant cost savings can be expected from elimination of some intermediate beams and their connections to the girders. These potential advantages have motivated research in the area of long span slab systems. In this case, long span slab systems that do not cause any significant increase in the depth and weight of the slabs compared to regular span slabs are particularly attractive. This has been one of the main objectives of this part of the study.

Research in this area has been carried out by other researchers. Notable among these are, the investigations by Ramsden (1987), the innovative lightweight floor system by Hillman and Murray (Hillman 1990, Hillman and Murray 1990, 1994) and the *slimflor* system (British Steel, Lawson et al. 1997). Ramsden (1987) conducted a study on two new prototypes of deck profiles that can span a distance up to about 24 ft (7.5 m). The prototypes 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. These two prototypes are shown in Fig. 5-1. 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 the construction was provided.



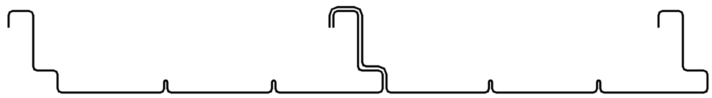


Figure 5-1. Prototype 1 and prototype 2 of Ramsden (1987) deck profiles

An innovative composite slab floor system design was developed and reported by Hillman (1990) and Hillman and Murray (1990, 1994). The floor system developed was not only lightweight but also able to span up to 30 ft without any intermediate beams. Figure 5-2 shows schematically the design of the composite slab.

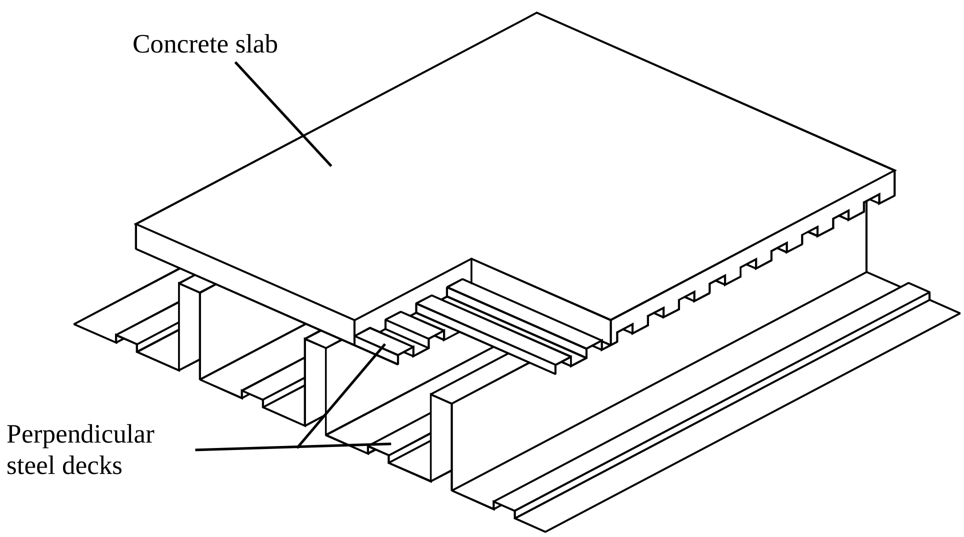


Figure 5-2. Innovative light weight and long-span composite floor
(Hillman 1990, Hillman and Murray 1990, 1994)

The *slimflor* system, marketed by the British Steel, utilizes deep deck sections of *ComFlor* 210 (210 mm, approximately 8.25 in. deep) and *SlimDek* 225 (225 mm, approximately 8.86 in. deep) sections. With lightweight concrete, the *ComFlor* 210 deck section can span up to 6 m (approximately 19.7 ft). Figure 5-3 shows schematic view of this *slimflor* system.

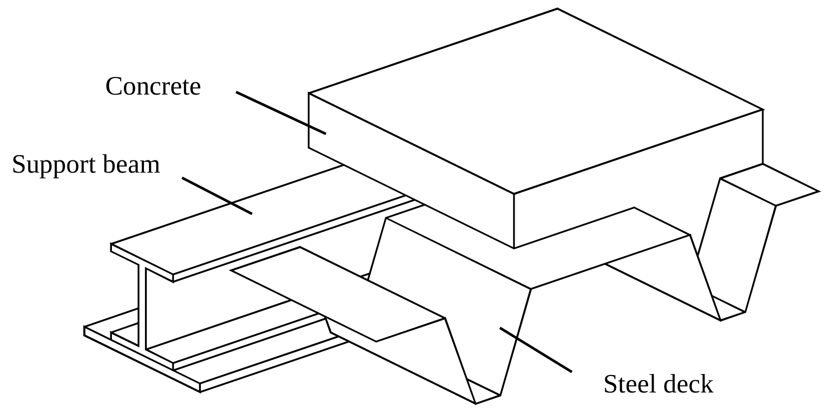


Figure 5-3. *Slimflor* system (British Steel, Steel Construction Institute 1997)

In the current study, two 16 ga, deep steel deck profiles are investigated. The first profile, referred to as profile 1, has a 6 in. rib height. The profile is currently not available in the market 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, i.e. profile 2, is a currently available roof deck profile whose stiffness, as discussed later in this chapter, satisfies the requirements for a long span slab in a double span configuration. This section was manufactured through a cold-rolling process. Profile shapes of these sections are shown in Fig. 5-4. Note that neither of these shapes incorporated embossments. This is because neither are currently available *composite* deck profiles. For comparison, a 3 in. deep trapezoidal section is also included in the figure.

Two design phases have to be considered in the development of these new deck profiles for long span composite slab systems, 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 becomes very crucial.

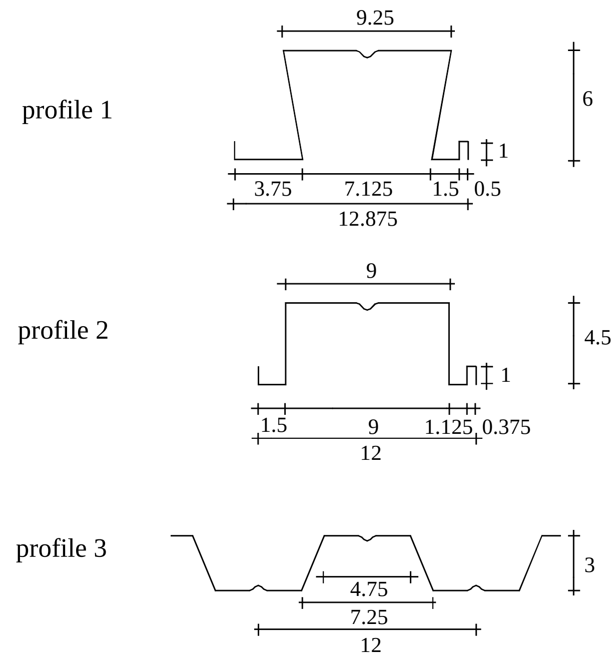


Figure 5-4. 6 in., 4.5 in. and 3 in. deep profiles

The service phase deals with a composite section of steel deck-concrete slab that is subject to occupancy loads. Studies on composite slabs with typical span lengths (Terry and Easterling 1994, Widjaja and Easterling 1995, 1996, 1997) revealed that the actual load capacity of the slabs are very high compared to the standard design live loads (50 to 150 psf). Table 5-1 shows that the ratios of actual load capacities (from the tests) to a 150 psf design live load range from 2.45 to 7.90. At these (ultimate) load capacities, however, the slabs have undergone excessive deflections. If the allowable deflection is limited to L/360 (SDI 1992), then, the permissible loads based on this allowable deflection will be much lower than the ultimate load capacities. The ratios of these permissible loads to a 150 psf design live load, as shown in Table 5-1, range from 1.37 to 3.11.

These ratios suggest that the service phase rarely governs the design of composite slabs. However, this is not always the case for long span composite slabs as latter shown by the analysis and test results. For long span slabs, both the construction and service phase have an equal change to govern the design.

Table 5-1. Ratios of actual load capacities and permissible load based on allowable deflection to 50 and 150 psf design live loads.

slab #	ultimate load capacity (psf)	load at allow. deflection *) (psf)	test load / 50		test load / 150	
			ultimate load capacity	load at allow. deflection	ultimate load capacity	load at allow. deflection
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 L/360

5.2. Construction Phase

As previously mentioned, this design phase considers the strength and stiffness of steel deck due to the fresh concrete weight. For typical span lengths, the flexural strength limit state is generally the governing condition in the design. For a longer span length, the governing condition is shifted toward the stiffness or deflection limit state. This condition is schematically shown in Fig. 5-5. The deflection is limited to l/180, as required in the SDI Composite Deck Design Handbook (Heagler et al 1997). The 0.75 in. maximum deflection limitation was not used because it is considered to be to restrictive for long span slabs.

For the purpose of this study, the construction phase is utilized to determine the profile shape of the steel deck that can be used for a desired span length. This was performed by generating charts of steel deck weight vs. span length as shown in Fig. 5-4, for various types of profile shapes. For the profiles shown in Fig. 5-4 with a 2.5 in. concrete cover in a single span system, Fig. 5-6 gives the plots of the steel deck weight vs. span length. Figure 5-7 shows similar plots for double span (continuous) condition. The steel deck weight was calculated based on the deck thickness that corresponds to the required moment of inertia for a certain span length with a given concrete self-weight plus the construction load.

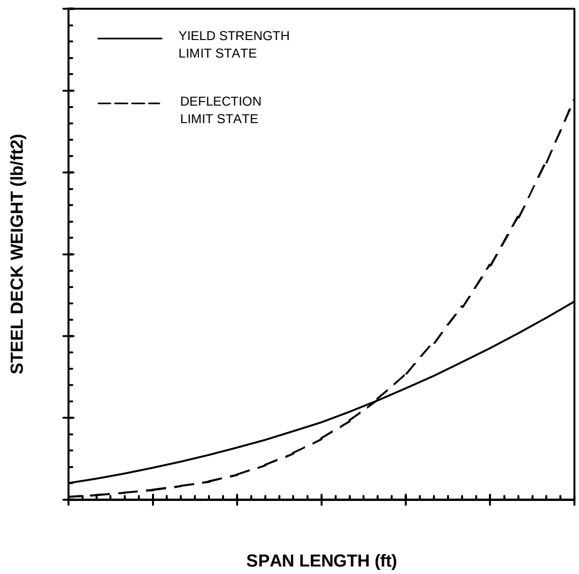


Figure 5-5. Yield strength and deflection limit states
of the construction (non-composite) phase

It can be observed from Figs. 5-6 and 5-7, that for a same weight of steel deck, profiles 1 and 2 allow one to have a longer span than that of profile 3. This indicates that profiles 1 and 2 are more efficient than profile 3. Therefore, for a long span slab system of 20 ft, only the 4.5 in. (profile 2) and 6 in. (profile 1) sections are considered in this study.

Table 5-2. Section properties of profiles 1, 2 and 3

Profile #	Thickness (in)	Area (in2/ft)	Inertia (in4/ft)	yp (in)	weight (lb/ft2)	slab weight (lb/ft2)
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

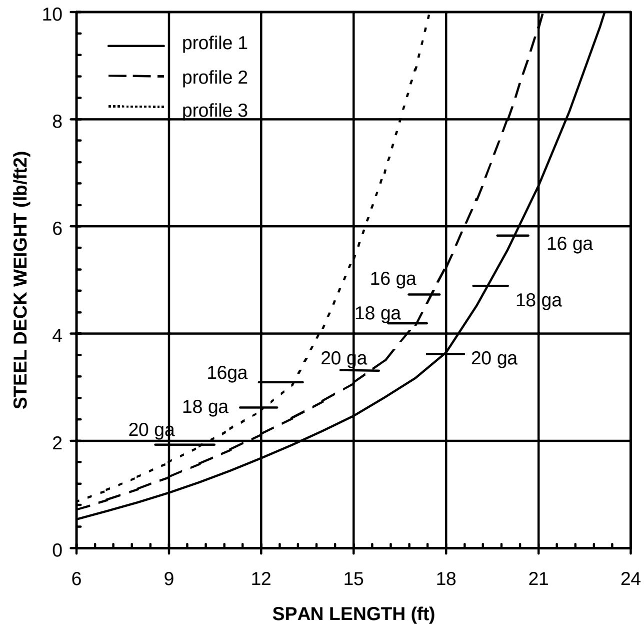


Figure 5-6. Steel deck weight vs. span length of single span systems

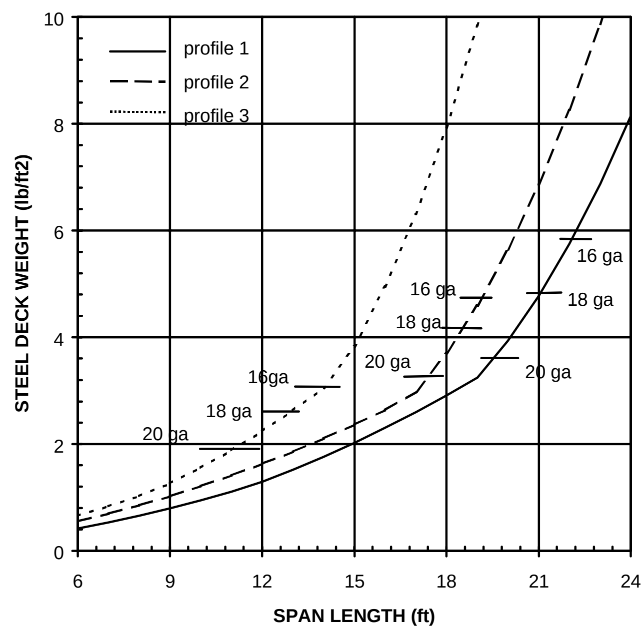


Figure 5-7. Steel Deck weight vs. span length of double span systems

From Table 5-2, by comparing values of profiles 1 and 3, it can be seen 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. The steel deck self-weight is almost double the one of profile 3. For a 20 ft long piece of deck with only one typical rib of profile 1, the piece weighs about 116 lb and it can be handled by two people in the construction site.

For profile 2, the increase of the moment of inertia is about 3 times of 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 as the steel deck.

5.3. Service Phase

In the service phase, predicted maximum test loads of composite slabs can be calculated using various ways. The iterative, direct, SDI-M and finite element methods were used to predict the capacities of the specimens built using profiles 1 and 2 in this study. However, only the iterative and finite element methods can provide response histories of load vs. deflection of the slabs. In the SDI-M and direct methods, I_{avg} is used with an elastic analysis to obtain permissible loads based on deflection limit state. The analysis was performed in the same ways as those with typical span length.

5.4. Specimen Descriptions and Instrumentation

Long span slab 1 (LSS1) has a configuration of two single deck spans of 20 ft each and 1 ft cantilever as shown in Fig. 5-8 (a). The total slab depth was 8.5 in. (2.5 in. concrete cover above the 6 in. deck rib height). Six 3/4 in. diameter, 8-3/16 in. tall shear studs were used at each end of the slab, spaced at 1 ft on center as shown in Fig. 5-9.

For long span slab 2 (LSS2), a two-span system was used with 20 ft span lengths. The configuration is shown in Fig. 5-8 (b). The total depth of the slab was 7 in. (2.5 in. concrete cover above 4.5 in. rib height). Six 3/4 in. diameter, 6-3/16 in. tall shear studs were used at the support, spaced at 1 ft on center as shown in Fig. 5-10.

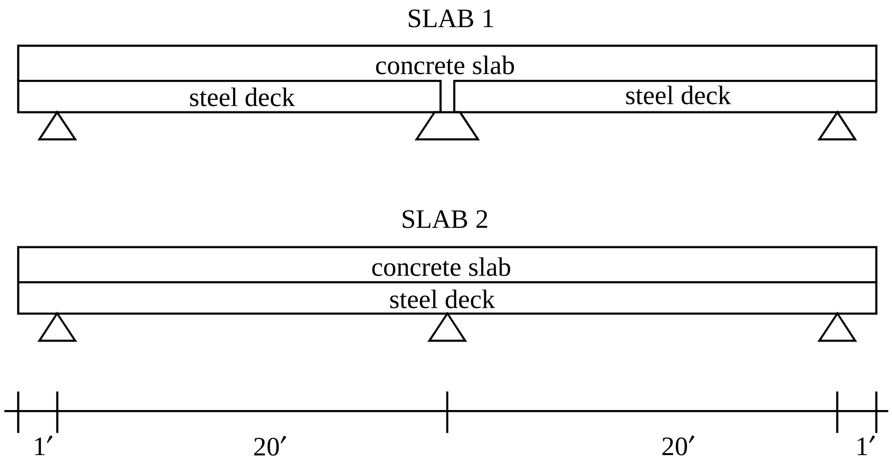


Figure 5-8. System configurations of LSS1 and LSS2

Strain gages were placed at the bottom surface of the deck to measure the steel deck strain during concrete casting and the load test. Three cross sections were monitored in each span of the slab: the exterior support, interior support and mid-span. A set of six strain gages was used at each of those cross sections. The schedules of these strain gage and shear stud locations are shown in Figs. 5-9 and 5-10 for LSS1 and LSS2, respectively. In addition to these strain gages, potentiometers were also placed at each end of the slab to measure the slip between the concrete and the deck. Several displacement transducers were also used to measure vertical displacements.

No shoring was provided during the construction of the slabs. The measured mid-span deflections of the steel deck during concrete casting were 0.695 in. and 0.685 in. for LSS1 and LSS2, respectively. Concrete compressive strength at 28 days were 3060 psi and 2330 psi for LSS1 and LSS2, respectively.

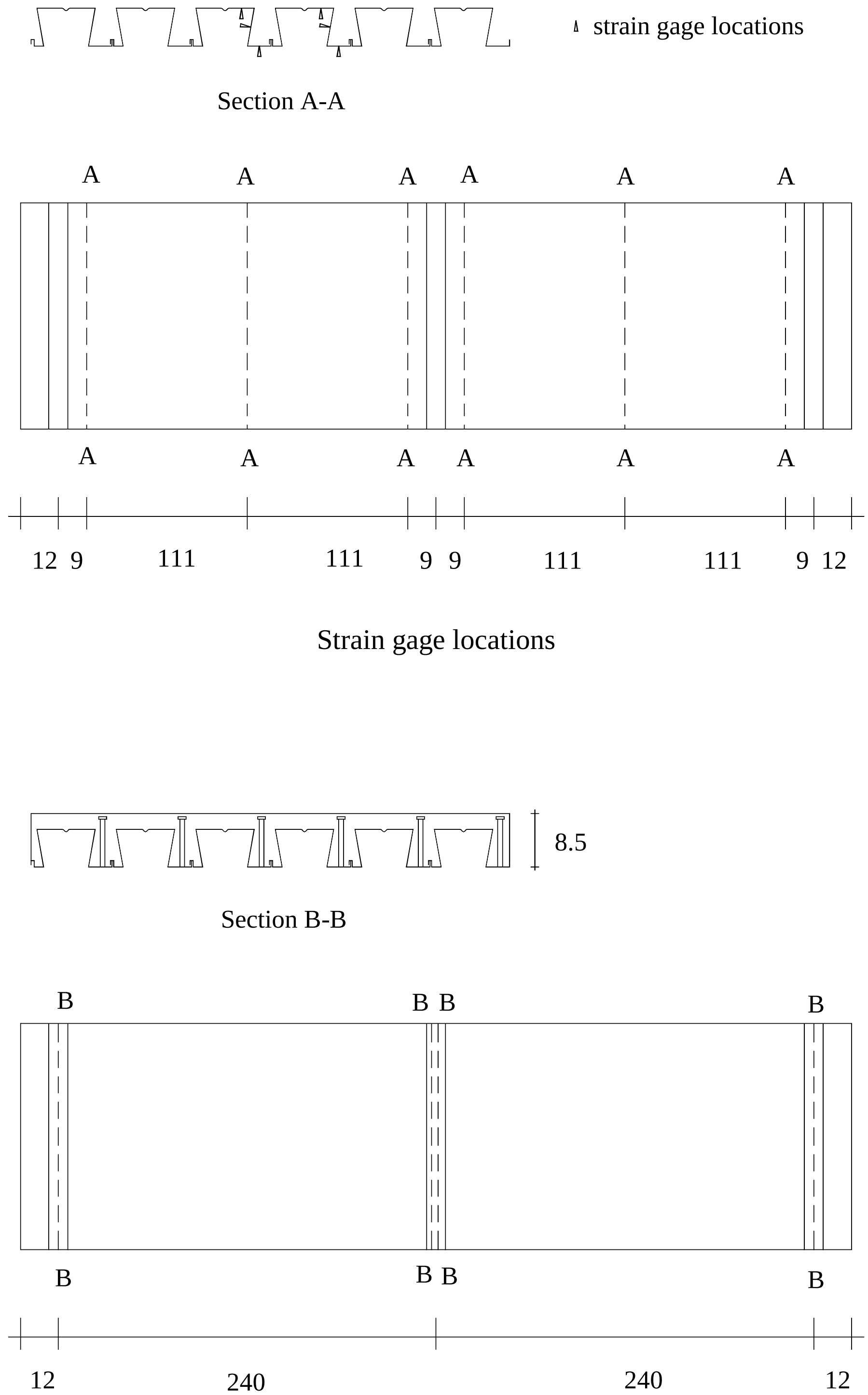
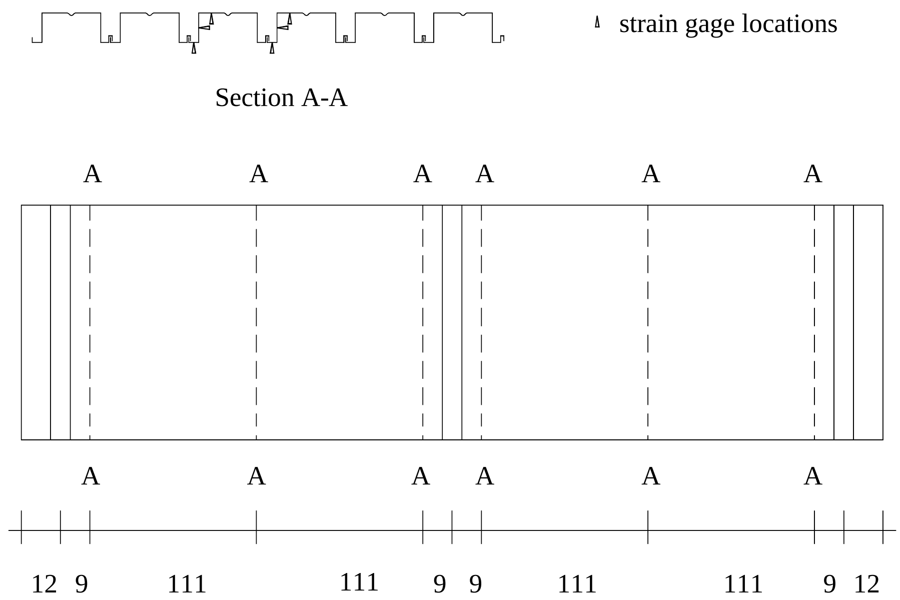
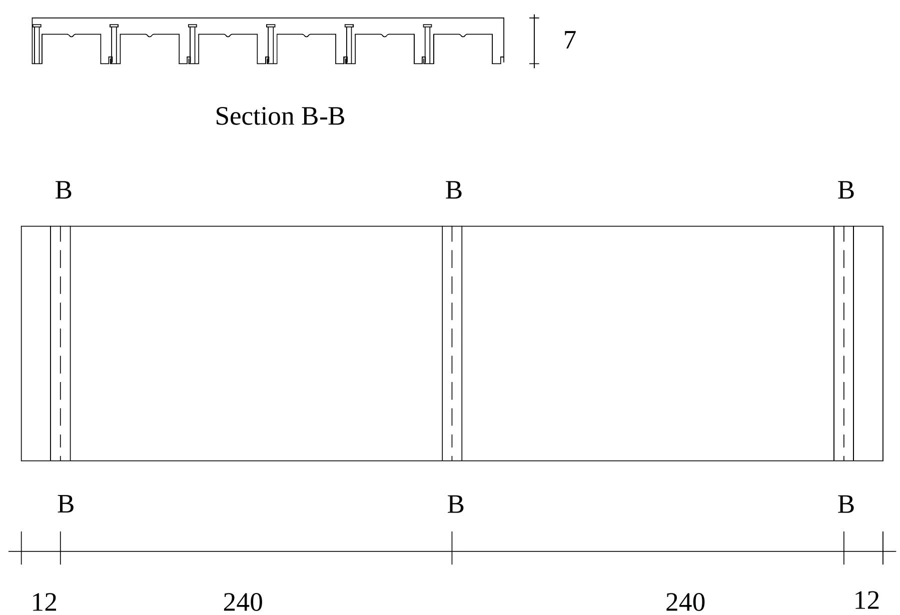


Figure 5-9. Strain gage and shear stud schedules of LSS1



Strain gage locations



Shear stud locations

Figure 5-10. Strain gage and shear stud schedules of LSS2