1 Introduction

1.1 Definition and advantages

A composite slab is defined by ASCE (1992) as “a slab system comprising normal weight or lightweight structural concrete placed permanently over cold-formed steel deck in which the steel deck performs dual roles of acting as a form for the concrete during construction and as positive reinforcement for the slab during service.” Deck profile, strength and thickness of the steel sheeting, span length and construction details influence the strength and behavior of composite slabs, and they determine whether the steel deck must be shored or unshored during construction.

Advantages of composite slab systems as mentioned in ECCS (1995) and Evans and Wright (1988) are:

- Once in position and prior to concrete placement, the steel deck immediately provides a platform to support construction loads and a safe sturdy surface for the work crew.
- The steel deck acts as the form for the in-situ cast concrete, thus eliminating the time consuming construction of costly removable forms.
- Once in service the steel deck acts as the tensile reinforcement, thereby eliminating the time-consuming placing and fixing of reinforcing bars for the slab.
- The shape of deck profile which can be made very effective can result in a reduction of about 30% in the amount of concrete fill required for the floor. The

consequent significant reduction in dead weight leads to lighter superstructures and reduced foundation loads.

- The cellular geometry of the deck permits the formation of ducting cells within the floor so that services can be incorporated and distributed within the floor depth. This increases headroom or reduces building height.
- Because steel deck is formed from thin gage steel sheeting, it is lightweight and easy to handle during both transport and placement.
- By using a composite slab, the construction time can be reduced and this will increase the economy of the construction considerably.

With the above mentioned advantages, composite slabs have greatly enhanced the competitiveness of steel-framed construction. At the present time, composite slabs are used in virtually all steel frame buildings in the US.

1.2 Composite slab behavior

It has been recognized that composite slabs under bending can exhibit three major modes of failure: flexure failure at section 1-1, vertical shear failure at section 2-2 and horizontal shear failure at section 3-3 as shown in Fig. 1.1 (Johnson, 1994).

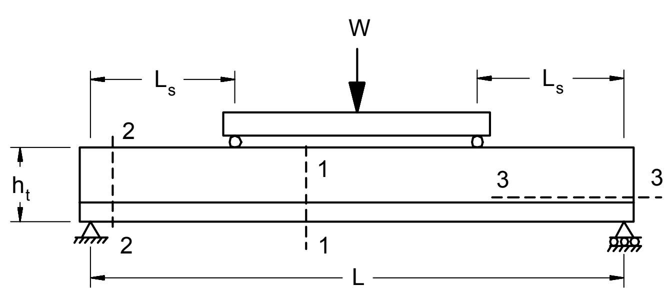


Fig. 1.1 Modes of failure of composite slab (Johnson, 1994)

The flexural failure (mode 1) occurs when complete interaction at the interface between concrete and steel is achieved. This type of failure usually occurs in long thin slabs. Analysis for this type of failure is quite easy, in which case ordinary reinforced concrete procedures can be followed (ASCE, 1992; Easterling and Young, 1992). The flexural failure however is not a dominant design criterion because the steel and concrete interaction is usually incomplete and the slab length is always limited by the serviceability (deflection) limit.

The characteristic of the second mode, which is the vertical shear failure, has been studied by Patrick and Bridge (1992). The slab has to be very short and thick with a high concentrated load near the supports for the mode 2 failure to be dominant. This is not common in construction practice therefore it has not been the subject of much research. The effect is typically ignored in design.

Failure mode 3, which is a horizontal shear failure or shear bond failure as it is commonly referred to, is the mode more likely to occur for most composite slab systems subjected to vertical loads (Porter and Ekberg, 1978; Schuster, 1970). This is characterized by the development of an approximate diagonal crack under or near one of the concentrated loads just before failure, followed by an observable end-slip between the steel deck and the concrete, within the concrete shear span, L_s, as illustrated in Fig. 1.2.

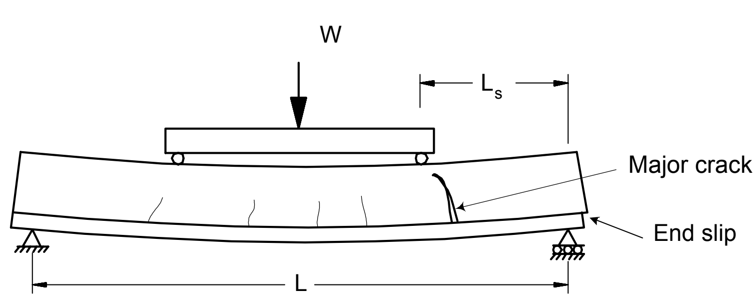


Fig. 1.2 Horizontal shear failure

The strength and behavior of composite slabs depend on several major factors such as shear transfer devices, steel thickness and slab slenderness. The shear transfer devices are usually a combination of steel profile shape, indentations or embossments on the steel surface and end anchorages. Some of the shear transfer devices normally employed in composite slabs are illustrated in Fig. 1.3.

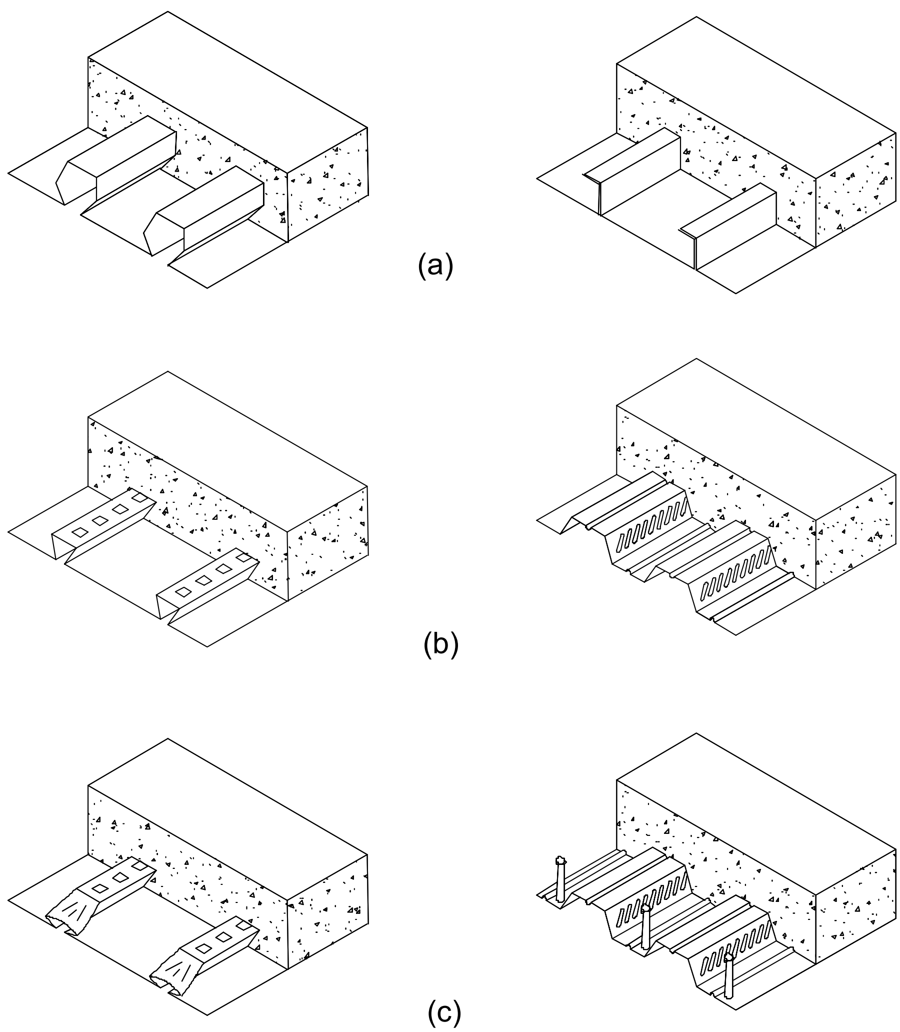


Fig. 1.3 Examples of shear connection devices in composite slabs
(a) Frictional interlock; (b) Mechanical interlock; (c) End anchorage combined with mechanical interlock (An, 1993; Eurocode 4, 2001)

Other factors that influence the slab performance include surface finish, steel strength, and concrete density, strength and curing age. Seleim and Schuster (1985) however reported that neither the reinforcement ratio nor the concrete compressive strength had a significant influence on the shear bond resistance but steel thickness was the governing parameter. Other researchers also confirmed that the concrete strength did not affect the slab performance significantly (Bode and Sauerborn, 1992; Daniels, 1988; Luttrell, 1987; Veljkovic, 1995).

Shear bond strength can be divided into three components namely chemical bond, frictional bond and mechanical bond. As explained by Burnet (1998), the chemical bond is a bond resulting from the chemical adherence of cement paste to the steel sheeting. Such bond exerts shear resistance with no slip at the interface. Once this bond is broken, slip is initiated and the chemical bond strength reduces to zero and does not reform. The frictional bond is a direct result from the application of active normal forces, which act perpendicular to the steel-concrete interface. This bond is directly proportional to the normal force, so that if the normal force is zero then the frictional force is zero.

Mechanical bond exists due to physical interlocking between the steel sheeting and the concrete. The interlocking is developed as a result of clamping action caused by the bending of steel deck, and from the friction between the steel sheeting and the concrete due to surface roughness such as indentation or embossment on the steel surface.

The interaction between the steel deck and the concrete is complex and difficult to model mathematically. As a result the design and analysis procedures available today have to rely on test data to account for the interaction parameters.

1.3 Objective

The primary objective of this research was to develop an efficient, simple and economical small scale testing procedure for composite slabs which was able to provide parameters needed for all design and analysis methods. The aims were to reduce the dependency on the full scale bending test which is required by the current design specifications, and to eliminate the need for elemental push off tests.

The small scale test referred to in this dissertation is a bending test conducted on a specimen whose width is one deck rib, which is equivalent to 1 ft wide for the VL type deck. The span length and concrete thickness are within the range of typical construction practice. The full scale test is a bending test conducted on a slab specimen whose minimum geometry is in accordance with either ASCE (1992) or Eurocode 4 (1994). Elemental push off tests are usually conducted to determine shear interaction relationship between the steel deck and the concrete for use in numerical analysis, to develop a new product, to study the behavior of the interaction property as a function of slab geometry, and to improve present design tools.

1.4 Scope

The scope of this research consisted of a laboratory test program, development of a shear bond stress calculation procedure, analytical and finite element (FE) modeling of shear bond, and study of composite slab behavior. The laboratory test program included full scale performance tests of composite slabs utilizing trapezoidal deck profiles that are commonly available in the US market, and the development of a new type of small scale test procedure. The development of a shear bond stress calculation procedure was necessary for obtaining the interaction property of a composite slab from the small scale test for use in the FE modeling and analysis. Finite element modeling and analysis was carried out to study the slab behavior focusing on the effect of the slenderness of the slab on the interaction property and slab performance.

1.5 Organization of this report

This dissertation is organized in seven chapters. Following Chapter 1, details of the full and small scale specimens are described in Chapter 2. Results of all tests are presented in Chapter 3. In Chapter 4, analysis according to present specifications, namely the ASCE (1992) and Eurocode 4 (1994), are presented. Then the proposed procedures to calculate the shear bond property[1] are developed. Comparison of results from the PSC method and the proposed procedures are also provided. The FE modeling and analysis is presented in Chapter 5. The issue of modeling the shear bond property for various slab slenderness and the effect of the slenderness on the slab behavior is presented in Chapter 6. Lastly summary, conclusion and recommendation are made in Chapter 7.

[1] The shear bond property referred to in this report is the graph of the horizontal shear stress between concrete and steel deck surface against end slip (horizontal movement of the concrete relative to the steel deck).

2 Experimental Program

2.1 Introduction

There are two composite slab design methods that are currently the most widely used. These are the shear bond method, also known as the *m-k* method, and the partial shear connection (PSC) method. The *m-k* method has been in the ASCE (1992), CSSBI (1996), BS-5950 (1994), Eurocode 4 (1994) and many other specifications around the world for many years. The PSC method, being the newer method, was implemented in Eurocode 4 (1994) (Annex E) as an alternative to the *m-k* method. The newer version of Eurocode 4 (2001) considered the PSC method as the major design procedure.

Both design methods however, still suffer a major drawback, in which the required parameters have to be obtained from full scale bending tests that are expensive and time consuming. This is obviously so when dealing with many types of steel deck profiles where composite slabs built with each of them have to be tested separately because of their different characteristics. Because of that, a smaller, simpler and more economical test has been needed and has been the main theme of studies for many researchers (Airumyan et al., 1990; An, 1993; Burnet, 1998; Daniels, 1988; Patrick and Poh, 1990; Plooksawasdi, 1977; Porter and C. E. Ekberg, 1978; Stark, 1978; Tremblay et al., 2002; Veljkovic, 1995; Zubair, 1989). The small scale test procedure developed in this research was one of the kinds that have potential for use as an alternative to the full scale tests.

2.2 Objective

The primary objective of the experimental program was to develop an efficient, simple and economical small scale testing method and procedure for composite slabs with the ability to provide parameters needed for all design and analysis methods; empirical, analytical or numerical. The second objective was to apply the result of the tests into two design methods, namely the *m-k* and the PSC methods, so that the need for full scale tests as required by these two methods can be eliminated. These two methods were specifically chosen for detail study along with the experimental results because of the fact that they are the most established and acceptable design methods for composite slabs to date.

In addition to replacing the full scale tests, the small scale test procedure developed here was also intended for other purposes. For example, deck manufacturers can use the tests as a tool to evaluate the performance of their steel decks, especially for development of new and more efficient profiles. The test procedure can also be used for parametric studies, where many tests and therefore cheaper tests are always needed. In numerical analysis, the shear bond stress-slip relationship is typically required and can be determined from the same tests, thus eliminating the need for separate elemental tests, such as push off or pull out tests as those proposed by previous researchers (An and Cederwall, 1992; Daniels, 1988; Patrick and Poh, 1990; Veljkovic, 1995). With its multipurpose and *small scale* characteristics, the test procedure developed here will be more economical as well as simpler and easier to perform than the full scale tests.

2.3 Review of the elemental and small scale tests

The purpose of elemental tests conducted in the past was mainly two-fold. First, elemental tests have been used as a means for evaluating the many parameters that affect the performance of composite slabs. These evaluations have resulted in refined design procedures or the development of more efficient profiles and embossment types (Airumyan et al., 1990; Jolly and Zubair, 1987; Shen, 2001; Tremblay et al., 2002). The second purpose was to obtain design parameters such as the shear bond property, friction coefficient, ductility characteristic, etc. for use in design and analysis (An, 1993; Burnet, 1998; Daniels, 1988; Patrick and Bridge, 1994; Patrick and Poh, 1990; Veljkovic, 1995; Zubair, 1989).

2.3.1 Schuster (1970)

One of the earliest elemental tests was the push out type performed by Schuster in 1970, as illustrated in Fig. 2.1 and reported by Porter and Ekberg (1978). The tests were carried out together with beam tests to investigate the shear bond characteristic of several types of steel decks available at that time.

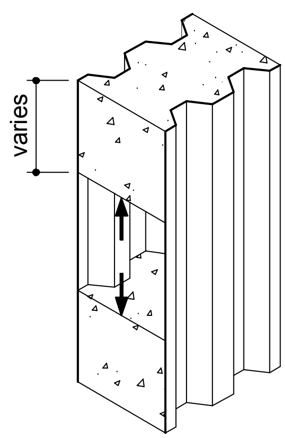


Fig. 2.1 Push out test by Schuster (1970)

The test was used in an attempt to establish the relationship between the maximum push out force and moment capacity of slabs with the same shear span length. The push out test results did not offer a good correlation with the bond strength for composite slabs in flexure. The test results were then abandoned and the full scale test was adopted as the preferred method to evaluate the composite slabs. The results from full scale tests later became the basis for the development of the *m-k* design procedure (Schuster, 1970; Porter and Ekberg, 1978).

2.3.2 Plooksawasdi (1977)

In 1977, Plooksawasdi developed a mathematical model for predicting the ultimate moment capacity of composite slabs that failed in shear bond. The model required the total pull out force, which was obtained from pull out test as shown in Fig. 2.2. The specimen dimensions were varied according to the size of one corrugation of the profiled deck. The relationship between shear stress and the slip were not reported. The mathematical model did not attract enough attention for further application.

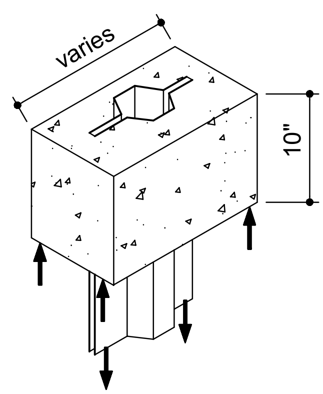


Fig. 2.2 Pull out test by Plooksawasdi (1977)

2.3.3 Stark (1978)

Stark (1978) used a push out test configuration as shown in Fig. 2.3, to determine the influence of concrete quality on the maximum load per embossment (burl). The push out test was conducted to confirm the results of full-scale bending tests that he used in the

PSC method. The concrete block was sandwiched between steel sheets, and the steel sheets were clamped against the concrete block but the clamping force was not measured. It was found that the ultimate shear load per embossment from a push test differed by 15% from the bending test. Stark concluded that the difference was due to the edge webs in the bending test, which were unrestrained and free to curl, but were clamped in the push test. Another factor that might have influenced the test results but was not addressed by Stark is the applied lateral force. The lateral force can induce frictional resistance at the interface and also can prevent the separation of the sheeting from the concrete.

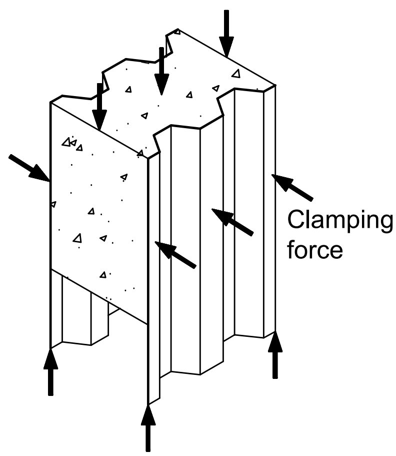


Fig. 2.3 Push out test by Stark (1978)

2.3.4 Jolly and Zubair (1978)

Jolly and Zubair (1987) experimentally evaluated the behavior of various types of indentations, pressed manually mostly on the web of deck profile, using a push off test configuration as shown in Fig. 2.4. The main objective of the tests were to evaluate the effect of different indentations on the shear bond strength, so that improvement to the sheeting profile could be made through modification of embossment shape, size, depth and spacing. Among the findings in these tests were that the discontinuous embossments appeared to enable the steel sheeting to distort and ride over concrete more easily. Embossment faces orthogonal to the direction of slip were more effective than the inclined. There appeared to be no advantage in more numerous and smaller indentations. Minimum width and maximum height and depth of embossment gave better results. The results were also used in the theoretical method for predicting the load carrying capacity of the corresponding composite slabs (Zubair, 1989).

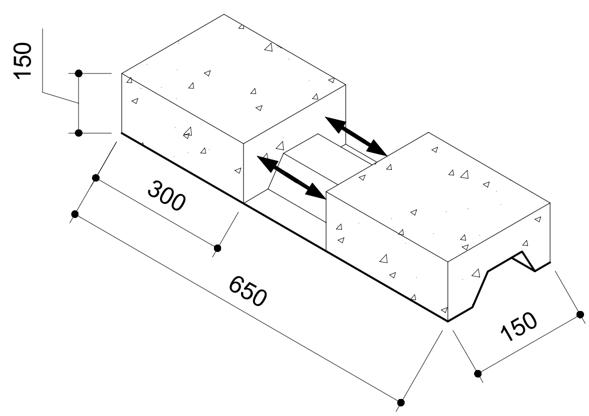


Fig. 2.4 Push off test by Jolly and Zubair (1987)
Note: Dimensions shown in mm

2.3.5 Daniels (1988)

Daniels (1988) developed a pull out test arrangement, as shown in Fig. 2.5, to determine the interaction properties of profiled sheeting. In this test, longitudinal force was applied to the sheeting in the direction of the rib with the concrete blocks held against a rigid frame. At the same time lateral force was applied onto the concrete block to simulate normal pressure exerted by the concrete weight at the sheeting-concrete interface. The resulting stress-displacement relation was then determined by dividing the applied load by total surface area of the steel sheeting in contact with the concrete. As expected, the final shear stress level increased as lateral force increased. The results tended to overestimate the shear resistance of the profiled sheeting. Daniels also reported that, contrary to the findings of Porter and Ekberg (1978), there was no appreciable change in bond stress with an increase in the specimen length. According to Daniels, while the chemical bond was still intact, the distribution of interfacial stress was non-uniform with peak stress occurring near the loaded edge.

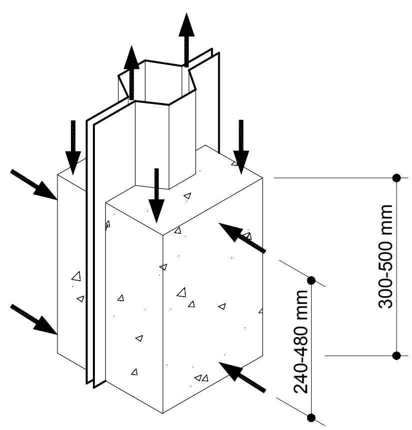


Fig. 2.5 Pull out test by Daniels (1988)

Overestimation of shear resistance can be expected from this specimen because the application of lateral force can produce additional frictional resistance at the interface. Another reason for overestimation is that the steel decks were held against each other thus preventing them from separating from concrete block naturally.

2.3.6 Patrick and Poh (1990)

The slip block test, as illustrated in Fig. 2.6, was developed and reported by Patrick and Poh (1990) and was used to investigate and quantify components of longitudinal slip resistance, namely adhesion bond, mechanical interlock and frictional resistance between concrete and sheeting. The mean shear stress per unit horizontal area and coefficient of friction between the concrete and the sheeting were obtained from this test. The parameters were applied in a rigorous analysis method developed by Patrick (1990).

It can be seen that this test setup is essentially similar to that of Daniels (1988), thus may suffer similar drawbacks. Because the sheeting was welded to the rigid base plate and lateral load was applied against the concrete block, vertical separation[2] between the sheeting and the concrete was most likely restricted. This could result in an overestimation of the shear bond stress. It should also be noted that the slip block test was only suitable for profile that exhibit ductile shear connection (Patrick and Poh, 1990).

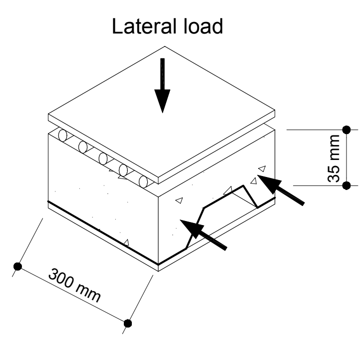


Fig. 2.6 Slip block test by Patrick and Poh (1990)

[2] Vertical separation is the separation between the concrete and the steel deck in the direction of lateral load (perpendicular to the deck flange surface).

2.3.7 Airumyan et al. (1990)

Airumyan et al. (1990) investigated the effect of various embossment shapes on the behavior of shear bond using a push test as shown in Fig. 2.7. The steel sheeting was flat and no corrugation was made. This type of push test had no mechanism to keep the sheeting in contact with the concrete block. Therefore it can be expected that the sheeting would have a tendency to pull away from the concrete as soon as chemical bond was broken (Burnet, 1998). This type of test is not suitable for determining the true shear bond stress involving the profiled sheeting.

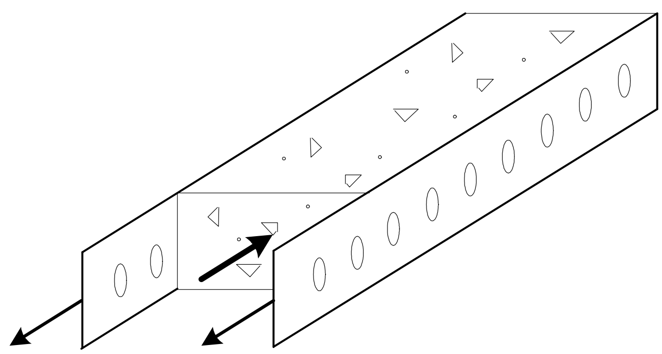


Fig. 2.7 Push test by Airumyan et al. (1990)

2.3.8 An (1993)

In an effort to include the effect of bending curvature in small scale tests, An (1993) developed a block bending test, as shown in Fig. 2.8, to determine the interfacial shear stress between the steel and the concrete. The parameters studied were concrete type (normal and light weight concrete) and the shear span to depth ratio. The test results were used as input in the finite element analysis. Two types of test setups were employed. The first shown in Fig. 2.8(a), had the steel sheeting in direct contact with the support so that the effect of frictional force on the slip resistance could be studied. The second setup shown in Fig. 2.8(b), did not have the steel in direct contact with the support, thus was used to obtain the shear strength without support friction. The shear stress was determined by calculating tensile force in the sheeting using analytical methods. The results were verified with the strain in the sheeting whose values were measured during the test. The shear resistance of specimen with the sheeting extended into the support was found to be 20-30% higher than that without sheeting. This clearly indicated the presence of shear resistance at the support.

Although the block bending test was not exactly similar to the actual slab, there was major improvement of the elemental test where it included the effect of bending curvature and vertical separation which were lacking in other type of tests discussed so far.

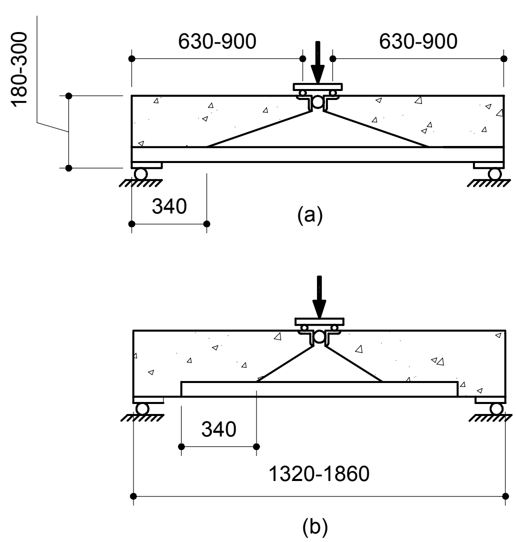


Fig. 2.8 Block bending test by An (1993)
Note: Dimensions shown in mm

2.3.9 Veljkovic (1996)

Veljkovic (1996a, 2000) studied the behavior of composite slab using the finite element (FE) method. The parameters used in the FE model were mechanical interlocking (shear bond-slip) resistance, a friction coefficient and a reduction function. These parameters were obtained from three types of elemental tests, namely push test (Fig. 2.9a), slip block test (see Sec. 2.3.6), and tension-push test (Fig. 2.9b). The push test was used to obtain the mechanical interlocking resistance. The slip block test was used to determine the friction coefficient at supports. The tension-push test was used to determine the reduction function of the shear bond strength. From the tension-push test, Veljkovic found that tensile strain could stretch and flatten the embossments which in turn would lower the shear bond resistance and increase the corresponding slip. Hence a reduction function was established to make correction to the mechanical interlocking resistance obtained from the push test. Further discussion on Veljkovic's model utilizing the reduction function is given in Sec. 5.7.

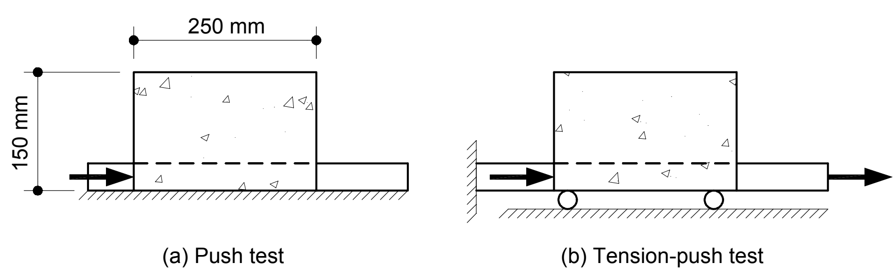


Fig. 2.9 Push test and tension-push test by Veljkovic (1995)

2.3.10 Burnet (1998)

In an attempt to overcome the drawback of most of the elemental tests discussed so far, where sheeting deformation was mostly restricted, Burnet (1998) used push test similar to Stark (1978) except that no lateral pressure was applied to the specimen and the sheeting was free to move laterally, as illustrated in Fig. 2.10. Using this test, Burnet studied the bond characteristics of variable shape profiles ranging from re-entrant to trapezoidal shape. He observed that for profiles with embossment, the longitudinal slip was accompanied by lateral separation of the sheeting from the concrete surface. For re-entrant ribs, the separation was less than the trapezoidal rib. Profiles with a larger rib opening (ie. trapezoidal profile) had lower shear resistance than that with a smaller rib opening. This was true for all sheeting thickness, for all profiles with or without embossments, and also for profiles with or without a debonding agent. Burnet also determined the limit of rib opening beyond which the shear capacity was completely lost after breaking of chemical bond. The results from this test were also used to study the performance of profiled composite beams.

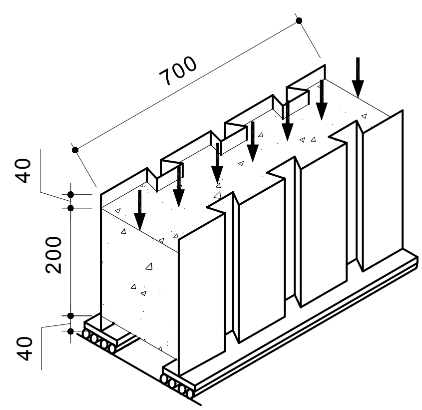


Fig. 2.10 Push test by Burnet (1998)
Note: Dimensions shown in mm

Although the sheeting was free to separate from the concrete, one factor that Burnet's specimen failed to address is the curvature and clamping force that arise due to bending as naturally happens in the actual slab. This factor may be significant for large curvature, especially for the re-entrant profiles.

2.3.11 Tremblay et al. (2002)

The latest push out test was reported by Tremblay et. al. (2002) as shown in Fig. 2.11. In this setup, a lateral load of 6 kN was applied at the end of the specimen to provide containment and, according to the authors, to simulate an end condition typical of simply supported slab. The authors used the test to study the effect of steel thickness, steel grade, surface coating, deck position (normal or inverted), concrete curing age and the presence of electrical conduit in the slab. As expected the shear strength was higher for stronger and thicker steel and for longer concrete curing age. Different surface coating produced different chemical bond strength. The presence of electrical conduit reduced the performance of the slab whereas deck position either normal or inverted had no effect on the shear bond strength.

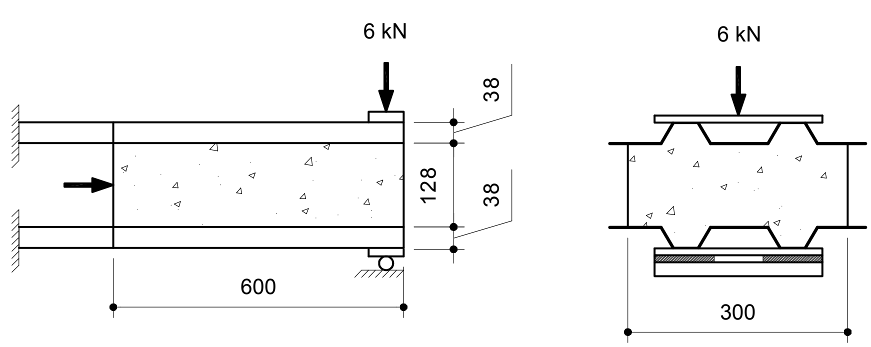


Fig. 2.11 Push out test by Tremblay (2002)
Note: Dimensions shown in mm

2.4 Critical issue pertaining to elemental tests

With the exception of the test by An (1993) all elemental tests discussed in the previous sections were conceptually similar in that direct shear loading was applied. These types of tests have a similar shortcoming because the complex interactive behavior of slab bending and shear that occur in actual slabs is not reflected in the test. The effect of curvature and clamping that induced by bending (see Fig. 2.12), slab slenderness, tensile strain in sheeting, natural frictional resistance at supports and other phenomena associated with bending could not be simulated in the direct shear loading tests. Furthermore, the fixing of steel sheeting to the test bed or to the opposite deck as in Plooksawasdi (1977), Daniels (1988), Jolly and Zubair, (1987), Patrick and Poh (1990), and Veljkovic (1996) can exert constraint to the movement of steel sheeting and hindered the tendency of the sheeting to separate from the concrete naturally.

As for the block bending test by An (1993), the bending, as in the actual slab, still could not be simulated accurately because of the shortness of the contact components and the test setup. The specimen is relatively complicated and perhaps the most difficult one to construct.

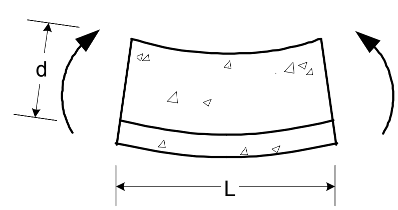


Fig. 2.12 Portion of slab under bending

Application of artificial lateral load such as in Stark (1978), Daniels (1988) and Tremblay et al. (2002) might induce or increase the frictional resistance and eventually may result in overestimation of the shear bond resistance. It was found also that slab slenderness (Tenhovuori and Leskela, 1998; Tenhovuori et al., 1996) and loading arrangement (Veljkovic, 1998) greatly influenced the behavior and strength of composite slabs. These factors were not and could not be included in the existing elemental tests.

At present, an elemental test has not been standardized and no such test is included in any specification. No attempt has been made to utilize elemental test data into present specifications. As a result design methods offered in the present major specifications such as ASCE (1992), BS-5950 (1994), CSSBI (1996) and Eurocode 4 (1994) still have to resort to full scale bending tests.

2.5 Test program

Three series of tests were carried out in this study. The first series, which consisted of 24 full scale specimens, were built and tested in accordance with the sponsor's details to determine the behavior and load capacity of the composite slabs under investigation. The data from the full scale tests was used to compare the performance of the newly developed small scale tests. Following the full scale tests, a series of 16 preliminary small scale tests were conducted. The small scale test was designed to behave and resemble the full scale test as closely as possible. The purpose of the preliminary test was to determine factors that significantly affect the performance of the small scale specimens. Once these factors had been recognized, the final details were chosen for the third test series, which comprised of 32 small scale specimens. The performance of these tests was then compared with the full scale results.

2.6 Full scale test

Twenty-four full scale specimens with twelve different configurations were built in a three-span setup. Three separate sheets of deck were used to form the three spans, thus a simple span configuration was used for the flexural evaluation. Both exterior spans were tested to failure while the intermediate spans were intended to provide constraint to slip and movement of the exterior spans. The slab geometries and details were chosen by the sponsor. The main idea was to create slabs specimens that can represent as closely as possible the composite slabs normally found in actual construction practice. Slab strength, vertical deflection, relative end slip, failure mode and strain response in the steel deck were investigated and recorded.

2.6.1 Steel deck properties

Light gage profiled steel decks that were used to build the slab specimens were made of structural quality steel sheets conforming to ASTM A653-94. The steel sheeting was finished with galvanized coating class of G60 (Z180) as defined in ASTM A653-94. The decks were manufactured and supplied by Vulcraft, a division of Nucor Corporation. The profile was a newly developed type known as "VL" where the shape was trapezoidal with two way embossments (up and down) skewed opposite on adjacent webs. Two and three in. deep decks with 20, 18 and 16 gage thickness for each were chosen for building the composite slab specimens. The deck section properties and embossment details are shown in Fig. 2.13. The corresponding values are presented in Table 2.1. The deck designation is in the form of "*i*VL*j*" where *i* and *j* denote the deck depth and gage thickness respectively.

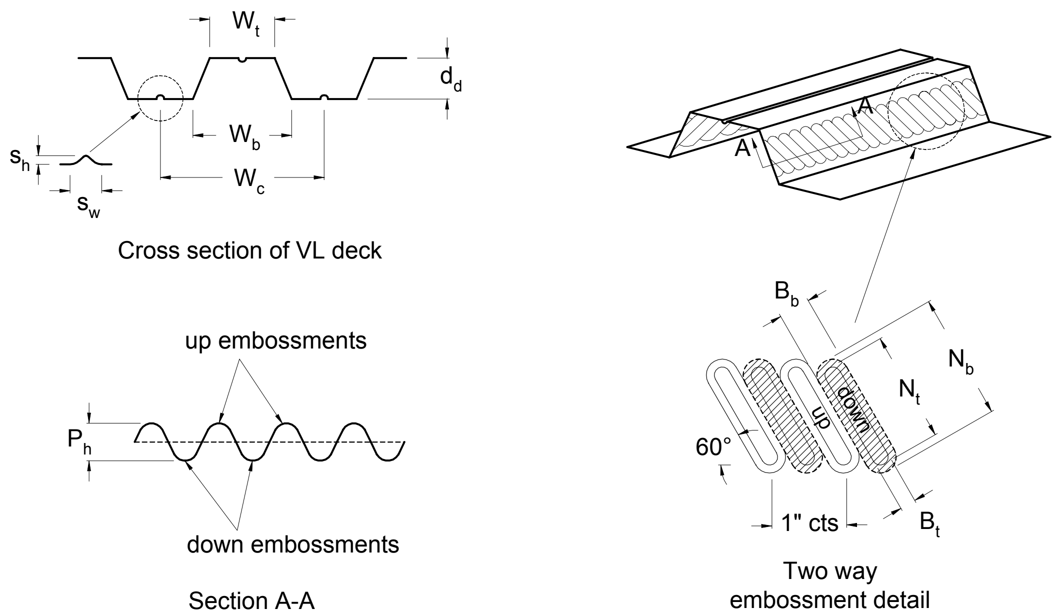


Fig. 2.13 VL deck cross section and embossment details

Table 2.1 Deck section dimensions and properties

Type	W_t (in.)	W_b (in.)	W_c (in.)	d_d (in.)	t (in.)	Weight (psf)	B_b (in.)	B_t (in.)	N_b (in.)	N_t (in.)	P_h (in.)
3VL20	4.75	7.25	12.0	3	0.0358	2.14	0.433	0.36	2.598	2.309	0.274
3VL18	4.75	7.25	12.0	3	0.0474	2.84	0.433	0.36	2.598	2.309	0.274
3VL16	4.75	7.25	12.0	3	0.0598	3.58	0.433	0.36	2.598	2.309	0.274
2VL20	5.00	7.00	12.0	2	0.0358	1.97	0.433	0.36	1.443	1.155	0.274
2VL18	5.00	7.00	12.0	2	0.0474	2.61	0.433	0.36	1.443	1.155	0.274
2VL16	5.00	7.00	12.0	2	0.0598	3.29	0.433	0.36	1.443	1.155	0.274

Type	s_h (in.)	s_w (in.)	A_s (in.^{2}/ft)	I_p (in.^{4}/ft)	I_n (in.^{4}/ft)	S_p (in.^{3}/ft)	S_n (in.^{3}/ft)	F_y (ksi)	F_u (ksi)
3VL20	0.5	0.8	0.594	0.938	0.937	0.553	0.572	54	64
3VL18	0.5	0.8	0.787	1.251	1.251	0.795	0.803	48	64
3VL16	0.5	0.8	0.993	1.580	1.580	1.013	1.013	51	60
2VL20	0.4	1.25	0.546	0.418	0.415	0.355	0.360	52	62
2VL18	0.4	1.25	0.723	0.557	0.557	0.512	0.518	49	59
2VL16	0.4	1.25	0.912	0.704	0.704	0.653	0.653	47	58

Note: Values for dimension and section properties were supplied by the deck manufacturer. I_p and S_p were used in all calculations

Coupon tests were carried out on the flat portion of the steel sheets according to ASTM E8-00b. Three specimens were cut from each bundle of each deck type. The stress-strain curves obtained from coupon tests are shown in Appendix A.1. The mean value of the test results was taken as the representative yield strength and ultimate strength for each deck type. The values are listed in the last two columns of Table 2.1.

2.6.2 Concrete properties

The concrete used for the specimens was of normal weight, designed for 3000 psi and supplied by a local concrete supplier. The maximum aggregate size was 3/8 in. Before placing, slump tests were performed to ensure that the mix was of good and consistent workability. The recorded slumps were in the range of 4.5 to 6.0 in. The concrete compressive strengths were determined from compressive test of 4 in. x 8 in. cylinders according to ASTM C39-96 procedures. Nine cylinders were prepared for each batch of concrete. They were prepared and cured in the same manner as the slabs. Three cylinders were tested on the same day as the slab test to determine the concrete compressive strength. The mean values of the cylinder strengths were taken as the representative compressive strength of each slab. Before carrying out the compression tests, the cylinders were weighed to determine the concrete dry weight. Because each slab specimen was prepared and cast at different times, the strengths and weight varied. The concrete strengths and dry weight are presented in columns 4 and 5 of Table 2.2.

Table 2.2 Slab parameters and concrete properties

Test number	Test ID representing the specimen parameters	Shear span measured on centers (in.)	Concrete	
			Compressive Strength, f'_c (psi)	Weight (pcf)
1	3VL20-8-7.5	*	3200	133
2	3VL20-11-5	40	5000	153
3	3VL18-8-7.5	32	2900	142
4	3VL18-13-5	52	5000	150
5	3VL16-8-7.5	32	3600	146
6	3VL16-14-5	52	3300	146
7	2VL20-7-6.5	28	4500	150
8	2VL20-9-4	38	4600	146
9	2VL18-7-6.5	28	2800	133
10	2VL18-11-4	40	4100	142
11	2VL16-7-6.5	28	4500	153
12	2VL16-12-4	46	4300	155

Note: * Slab #1 was tested using airbag. The major crack was formed at 41 in. and 36 in. from center of support for test A and B respectively

2.6.3 Details of test specimen

In addition to deck labeling presented in Sec. 2.6.1, the specimens were labeled in the form of "*i*VL*j*-*k*-*l*" where *i*, *j*, *k* and *l* are variables indicating deck depth (in.), steel deck thickness (gage), span length (ft) and total slab thickness (in.) respectively. Hence, "2VL20-7-6.5" refers to the specimen using 2 in. deep VL type 20 gage deck (see also Sec. 2.6.1), 7 ft span measured on centers of support beams, and 6.5 in. total concrete thickness. Specimen configurations and details are shown in Fig. 2.14 and Fig. 2.15 and test parameters are presented as ID labels in the second column of Table 2.2.

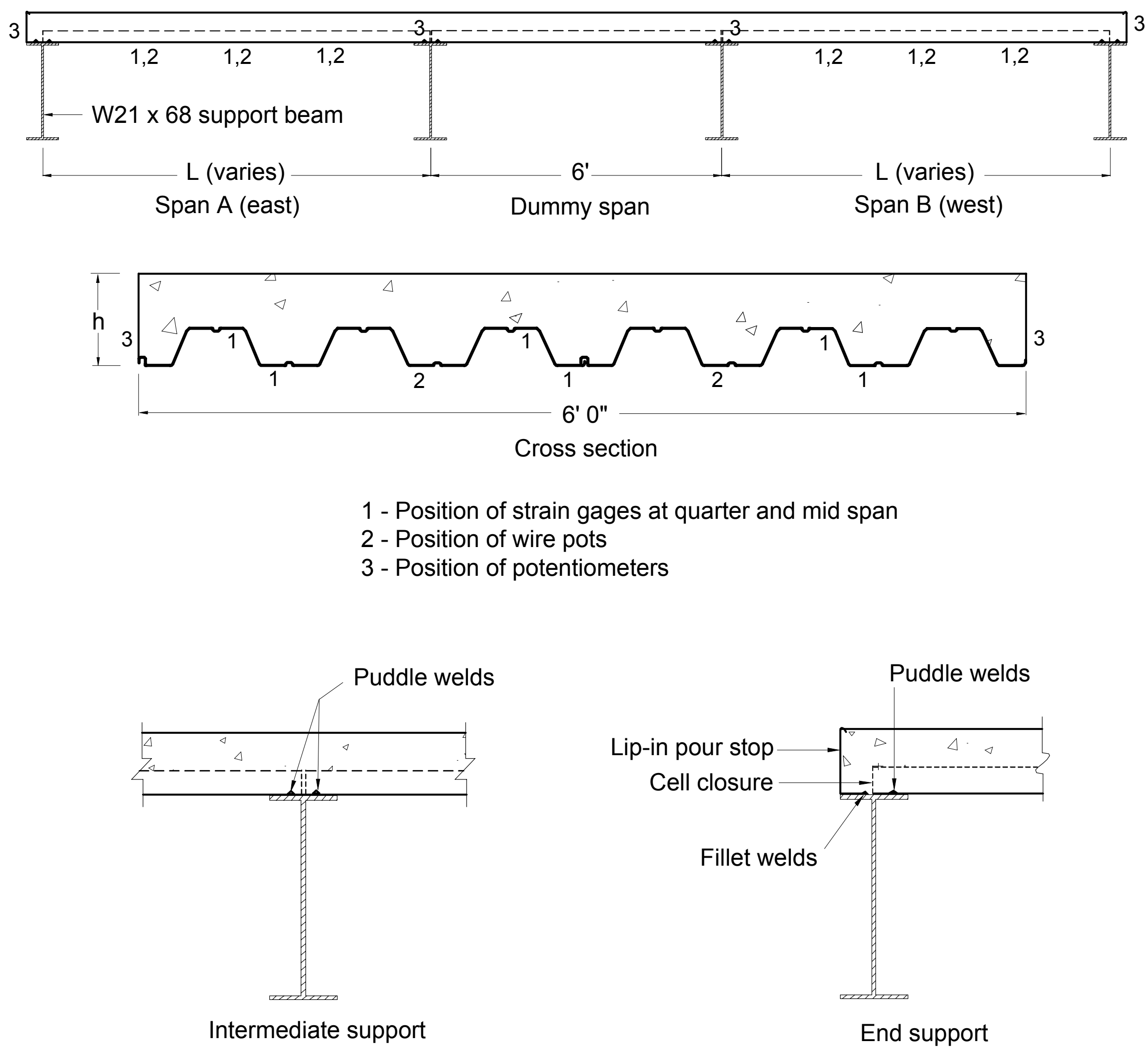
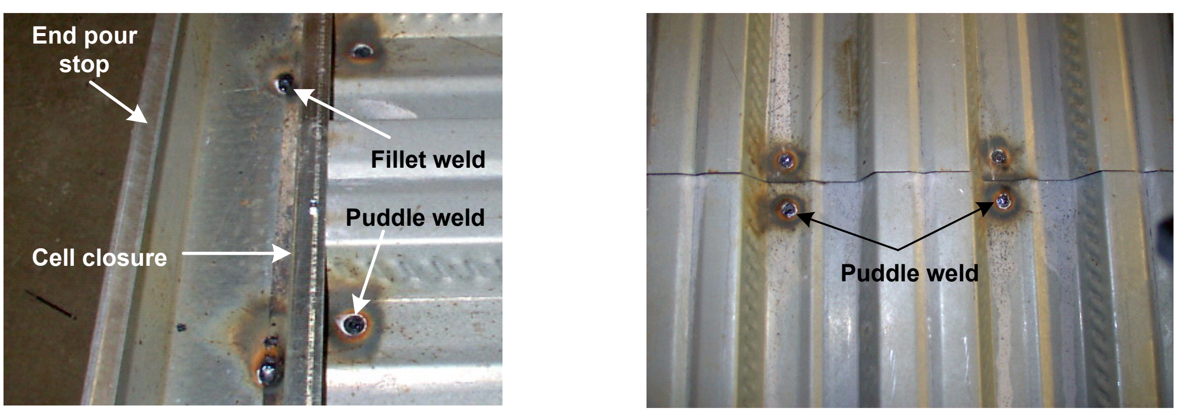


Fig. 2.14 Details of full scale specimen



(a) **(b)**

Fig. 2.15 Photographs showing steel decks at supports
(a) End of slabs (b) Intermediate support

2.6.4 Preparation of slab specimens

Each slab specimen was prepared in a similar manner. First, strain gages were attached to the undersides of the decks. For each tested span, nine strain gages were on the bottom and top flanges respectively with three gages each at first quarter point, mid-span and third quarter point. The strain gage locations are shown in Fig. 2.14. After strain gaging, the decks were then placed on the W21 X 68 support beams to form three-span simply supported slabs as shown in the figure. The interior spans for all specimens were fixed at 6 ft long and were constructed using 18-gage decks. All specimens were 6 ft wide, which were formed with two pieces of 3 ft wide decks attached and button punched at 6 in. interval along their side edges. At the support, the deck bottom flanges were puddle welded to the support beams. The nominal puddle weld size was $^5/_8$ in. However, it was difficult to maintain a $^5/_8$ in. maximum while still ensuring a quality weld. The actual visible diameter was typically between $^5/_8$ in. and ¾ in.

Cold-formed pour stops were used to form the sides of slabs. The return lip was bent away from the slab and the pour stops were attached to the bottom flanges along the side edges with screws from below to facilitate removal prior to testing. Small threaded rods (¼ in. diameter) were fastened at the opposite sides of the pour stops to prevent them from deflecting laterally when the concrete was placed. One or two rods were used depending on the length of the particular span (see photograph in Appendix B.1(b)).

At the exterior ends, pour stops with return lips bent into the slab were fixed to the support beams by ¾ to 1 in. nominal length fillet welds at 12 in. intervals. The fillet weld size was not controlled because of difficulty welding on too thin steel sheet. The pour stops were left in place and served as anchors at the slab ends. Welded wire fabric was placed in all slabs as shrinkage and temperature control reinforcements as specified in the ASCE (1992) specification. Mesh size 6 x 6-W1.4 x W1.4 was used in 4 and 5 in. thick slabs (slender slabs) and 6 x 6-W2.1 x W2.1 was used in 6.5 and 7.5 in. thick slabs (compact slabs). Photographs showing puddle and fillet welds, and end and intermediate support details are shown in Fig. 2.15. See also photograph in Appendix B.1(a).

Following preparation of the decks, concrete was then placed, vibrated and finished. The steel deck was not shored between supports. The deck strains and deflection due to fresh concrete were recorded. Wire pot displacement transducers were used to record the deflection at mid-span and quarter points. When the concrete began to set, a ¼ in. wide and ½ in. deep groove was made on the top surface for controlling cracks at the interior supports along the width of the slab. The concrete was then cured by covering it with plastic sheets and was kept moist for seven days. After seven days, the plastic cover and pour stops along the sides of the slabs were removed. The specimens were then left cured under normal room temperature for a minimum of 28 days. Concrete test cylinders were also prepared and cured in the same manner as the slab specimens. Details of the concrete properties were discussed in Sec. 2.6.2.

2.6.5 Instrumentation for composite slab testing

When the specimens were ready to be tested, potentiometers were fixed at each corner of the slabs to measure the relative slip, which is the horizontal movement of the concrete with respect to the deck. The potentiometers were fastened to the sides of the concrete slab with their plungers held against metal angles which were glued to the underside of the steel deck. Photographs in Appendix B.1 (e) and (f) show the potentiometers used for recording end slip.

Wire pot displacement transducers were used to measure vertical deflections. Two transducers were fixed at the first quarter point, mid-span and third quarter point. An air bag was used to apply uniform load for the first three tests as explained in the following section. A hydraulic actuator and load distribution frame was used for the remaining tests. A pressure transducer was used to record the air bag pressure. A 250 kips capacity load cell was used for the remaining tests.

2.6.6 Test procedure

In this experimental program, only static load was applied. Cyclic loading prior to static loading as recommended by Eurocode 4 (1994) testing procedure was not followed. According to Wright and Evans (1987), composite slabs were negligibly affected by the cycling loading. The ASCE (1992) specification also does not specify the need for cyclic loading.

Specimens 3VL20-8-7.5-A and B were tested by loading uniformly with a 6 ft x 10 ft rubber airbag similar to the procedure used by Shen (2001). The load was exerted by a power operated hydraulic ram against a reaction frame. The air bag was also used for specimen 3VL18-8-7.5-A. However this specimen could not be loaded to failure because the airbag capacity was reached. The test for 3VL18-8-7.5-A was then continued with two symmetrical line loads. For reasons of consistency, it was decided that the rest of the slab specimens would be tested similarly using line loads as illustrated in Fig. 2.16. The line load test procedure and the selection of shear span length were in accordance with ASCE (1992). Static load was applied incrementally first by load control and after cracking was significant, the load was increased by displacement control. Each load increment was held for at least 2 minutes to ensure that the slab was stabilized before the reading was recorded.

After completing the test, crack patterns were measured and recorded at both sides of the slabs.

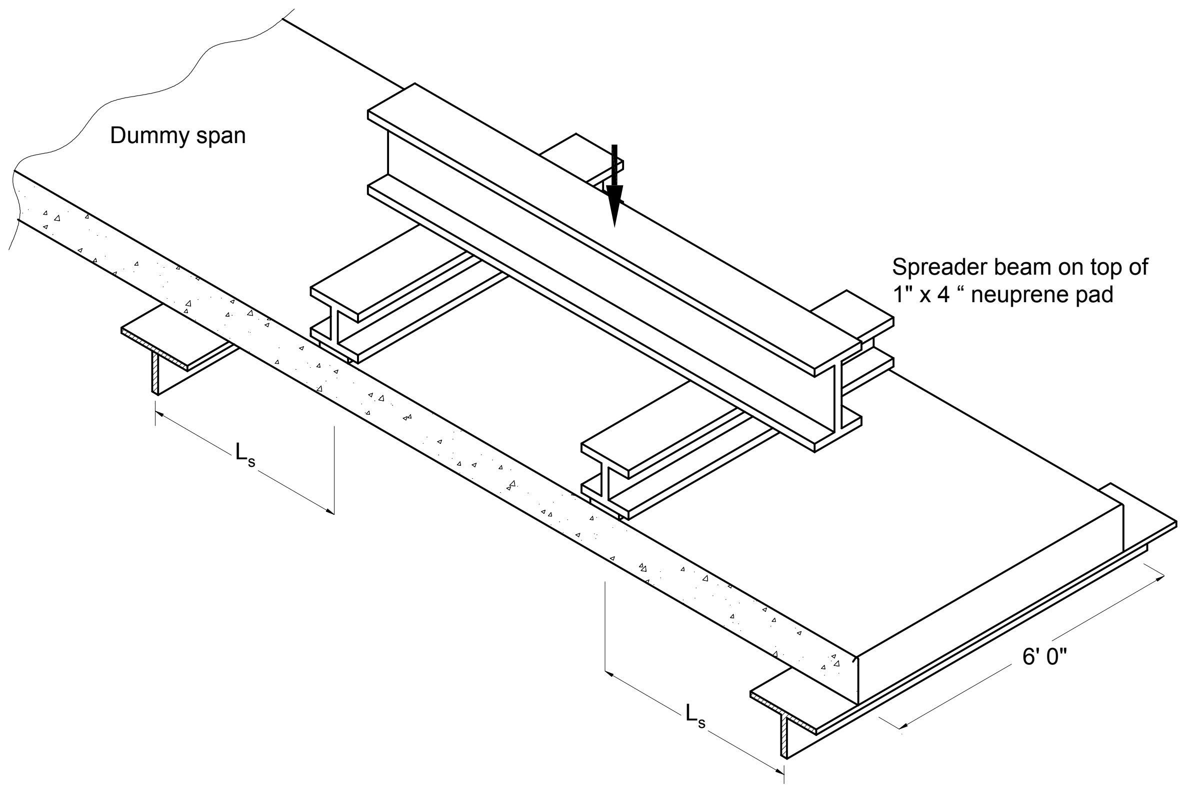


Fig. 2.16 Full scale test diagram

2.7 Small scale test

As mentioned in the beginning of this chapter, development of a new type of small scale test procedure is the central objective of this research. The tests were conducted in two separate series which are referred to as series #1 and series #2. Series #1 consisted of 16 specimens with 8 different parameters and was constructed to study the effect of constraining the webs of the deck against curling. Specimens with a number of different thicknesses and shear spans were also constructed to determine their impacts on the load capacity. Test results from series #1 were used as a basis for the selection of specimen details for series #2, so as to create small scale specimens whose behavior was comparable with those of the full scales tests. Hence, comparison between the small scale and full scale tests can be made. Thirty two specimens were tested in series #2, of which two tests were conducted for each different parameter.

In the following section, the discussion is first focused on the major factors that influence the performance of small scale specimens and then followed by the details of the specimen and test procedure.

2.7.1 Development of small scale specimens

There are two critical factors that were known to influence the results of small scale specimens. The first is web curling as recognized earlier by Stark (1978), which may occur in edge webs as depicted in Fig. 2.17(a). The presence of embossments in the webs can create a reaction force that pushes the webs away from the concrete once the slip occurs. Thinner steel sheets and deeper webs are more vulnerable to flexing or curling. Because of this flexibility, the overriding resistance between the concrete and steel sheet can be significantly reduced when horizontal shear slip is taking place in the small scale specimen. This effect is less significant in full scale specimen because the webs are interconnected and restraining each other as shown in Fig. 2.17(b).

It should be noted that the so called *full scale test* done by researchers in the past were actually not too wide. Test specimens reported in Porter and Ekberg (1978) for example, were mostly made of one panel deck with width ranging from 12 in. to 36 in. depending on different designs of profile geometries by different manufacturers. As reported by Prasannan (1983), tests conducted at West Virginia University for varying numbers of ribs had shown that the load carrying capacity increased as the number of ribs of the test specimen increased. The capacity of the slab started to stabilize when the numbers of ribs increased to 14. This indicated that if the full-scale test was built with the minimum width as prescribed in the ASCE (1992) and Eurocode 4 (1994) specifications, which is the larger of 2 ft wide or one steel deck panel, the full-scale result may still be well below the actual slab. Therefore if the webs could be stiffened and prevented from becoming too flexible, the performance of small scale specimen, though not necessarily equal to the actual slab, may have come closer to the full scale tests and hence should be admissible for design purpose.

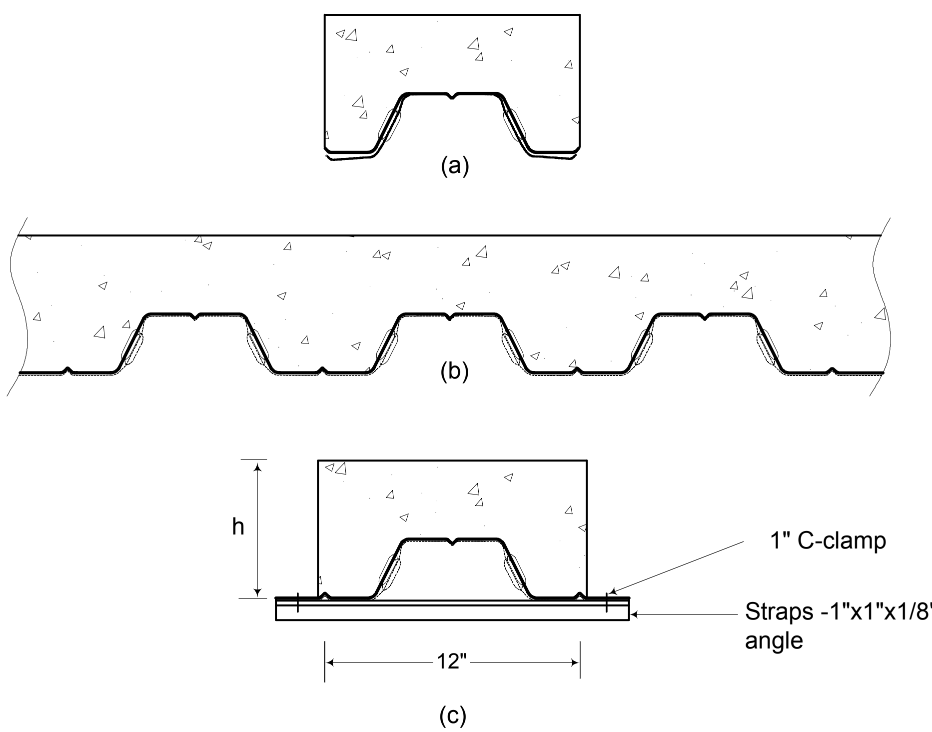


Fig. 2.17 Actual slab and small scale specimen cross section
(a) Web curling of small specimen (b) Actual slab condition (c) Strengthening the web using angle straps

To reduce web curling on small scale specimen, angle straps as shown in Fig. 2.17(c) were fixed using 1 in. C-clamps (also shown in photographs in Appendix B.2(b), (d) and (e)) to the bottom flanges of the test specimen along the span to simulate the restraining effect of adjacent webs in full scale condition. The strap members were light but stiff enough so that they did not buckle when both webs were trying to curl due to concrete overriding[3]. To compare the effect of strapping the webs, specimens without straps were also tested.

The second critical factor that can influence the slab behavior and strength either for test specimens or for actual slabs in the field is the construction detail at the supports. The ASCE (1992) and Eurocode 4 (1994) standard test procedures specify the test specimen should be simply supported without providing any restraint to concrete and deck movement at the supports. However in practice, the slab ends are usually anchored, typically by welding of the decks and pour stops to the support beams which are then permanently left in place after construction and become part of the system. Elimination of these details in test specimens can reduce their stiffness and load carrying capacity which may produce too conservative results and eventually uneconomical design. As such, these factors were addressed in the small scale specimen developed in this study so that the specimens were as close to the field condition as possible. Specimens without fixing the deck to the support beams were also built and tested to show the effect of end details (see notes for Table 2.4 and Sec. 2.7.3).

Other factors that are known to affect the specimen strength and behavior are shear span, and concrete thickness. These factors were also considered in the development of

[3] Concrete overriding carries the meaning of the concrete slips against uneven deck surface (e.g. embossment) causing a reaction that pushes the deck away from the concrete.

small scale specimens and were evaluated in series #1. It should be noted that the influence of shear studs, which are usually used in composite beam construction, was not considered in this research.

2.7.2 Description of series #1 specimens

All specimens in series #1 were built with 3VL16 steel decks. Eight parameters were investigated with two tests conducted for each detail. The decks used in series #1 specimens were of lower strength steel compared to other specimens. This was indicated by the coupon test results as shown in Appendix A.1(c). The average 0.2% offset values for the yield strength was 34 ksi and the ultimate strength was 50 ksi. The concrete compressive strength was 4200 psi and the dry weight was 142 pcf. Test parameters and material properties for series #1 are shown in Table 2.3.

Each slab specimen was prepared in similar manner. First, one rib of the steel deck was cut from a deck panel as shown in Fig. 2.18. The bottom flanges were cut slightly longer than 12 in. wide to facilitate fixing of angle straps. All decks were cut such that the embossments on one web were skewed opposite of the embossments on the other web except for specimen #15 in Table 2.3, where the embossments on both webs were skewed towards the same direction.

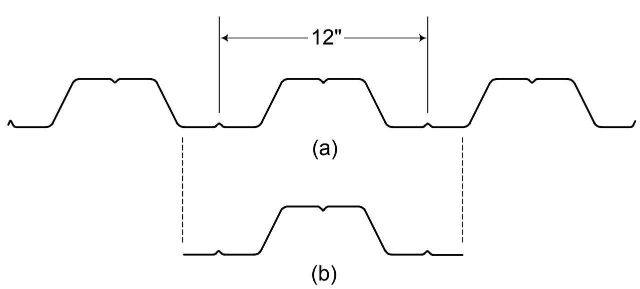


Fig. 2.18 Steel deck cross section
(a) One deck panel (b) A cut from the middle rib

Table 2.3 Test parameters for small scale specimens in series #1

Test number	Test parameters			Concrete		Steel deck	
	ID representing the specimen parameters	Shear span (in.)	Straps	Comp. strength f'$_c$ (psi)	Weight (pcf)	Yield strength F_y (ksi)	Ultimate strength F_y (ksi)
13	3VL16-8-7.5	22	at 6" interval along shear spans only	4200	142	34	50
14	3VL16-8-7.5	22	No	4200	142	34	50
15	3VL16-8-7.5	26	at 4" interval along shear spans and 8" in the constant moment region	4200	142	34	50
16	3VL16-8-7.5	26	No	4200	142	34	50
17	3VL16-8-7.5	30	at 4" interval along shear spans and 8" in the constant moment region	4200	142	34	50
18	3VL16-8-7.5	30	No	4200	142	34	50
19	3VL16-8-6.5	22	at 4" interval along shear spans and 8" in the constant moment region	4200	142	34	50
20	3VL16-8-5	22	at 4" interval along shear spans and 8" in the constant moment region	4200	142	34	50

Note: Test number 1-12 are for full scale specimens

The specimens were constructed in single spans with no shoring. The deck ends were not anchored and no permanent pour stop was provided. This was done by using lip-in pour stop facing away from the concrete. Except for the width, other details were similar to the ASCE (1992) specification. Welded wire fabric was also placed in all specimens as shrinkage and temperature reinforcement. The construction process and quality control was in the same manner as the full scale specimens. The deck deflection and strain due to fresh concrete was not recorded. This was decided considering the fact that the side form stiffness was relatively large compared to the narrow deck in the small scale setup. As such, it was assumed that if deflection and strain were measured it would not reflect the true response of the deck and therefore they would not be useful for data analysis.

When the slab specimens were ready to be tested, one end of the specimen was slowly lifted using an overhead crane and a pin support was put underneath the slab end. The lifting was done very slowly and carefully to prevent damaging the specimen during the process. For lifting the slab specimen, a steel bar was put under the slab as close to the slab end as possible and the bar was hung from both ends by a sling to the overhead crane. The same procedure was repeated on the other end to install a roller support. Potentiometers were fixed at both ends of the specimen and LVDTs were placed at the sides near load points to record the relative slips. Wire pots were attached to the underside of the slab at mid-span, and at the load points to record the deflection. The locations of potentiometers, LVDTs and wire pots are indicated by point 1, 2 and 3 in the test setup diagram shown in Fig. 2.19(a). The specimen cross section is shown in Fig. 2.19(b) and the isometric view of the test setup is shown in Fig. 2.19(c). Photographs showing the potentiometers and LVDTs are available in Appendix B.2(c) and (d).

Angle straps were then fixed to the underside of deck bottom flanges at a predetermined interval along the length of the slabs to simulate the restraining effect of adjacent deck ribs. The straps were made of 1 in. x 1 in. x 1/8 in steel angle and were fixed with 1 in. C-clamps. The clamps were hand-tight but the clamping force was not measured. The frequencies of straps used for specimens in series #1 are given in Table 2.3. A view of angle straps in the final position and C-clamps for holding the straps are shown in Appendix B.2(b), (d) and (e). The test was then carried out by applying line loads similar to the line load test procedure for the full scale specimens. The actual test in progress is shown Appendix B.2(a).

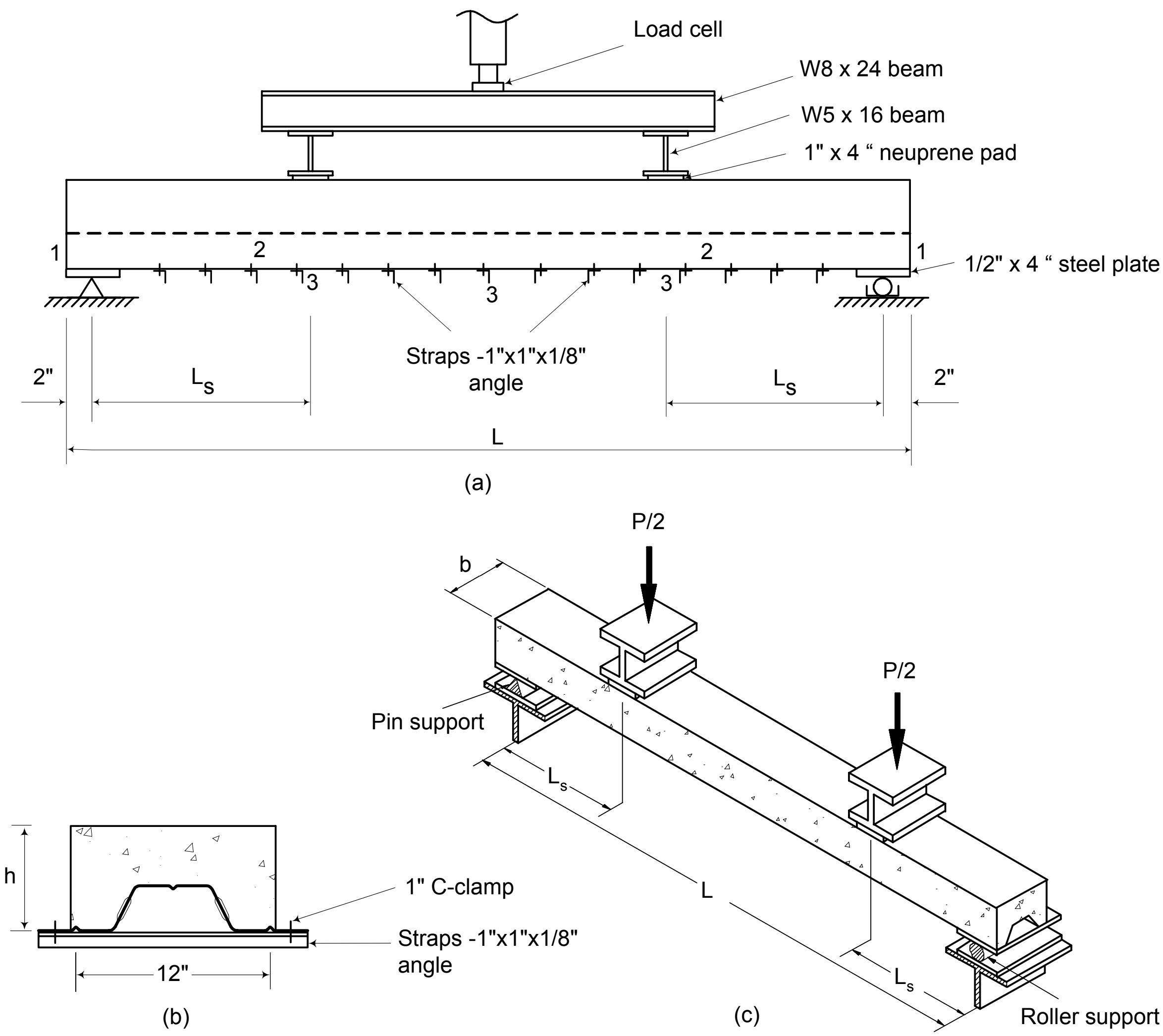


Fig. 2.19 Small scale test setup for series #1 specimens
(a) Side view. Number 1, 2 and 3 indicate the locations of wire pots, LVDTs and potentiometers respectively. (b) Cross section. (c) Isometric view

2.7.3 Description of series #2 specimens

Following the results and observations of the tests in series #1, it was decided that the specimens in series #2 were to be built in single span with end conditions, construction, and test procedure similar to the full scale tests as discussed in Sec. 2.6. The test setup and end details are depicted in Fig. 2.20 while the test parameters are listed in Table 2.4. To facilitate comparison between simple and welded supports, two otherwise identical specimens (3VL16-8-7.5 labeled as #26 in Table 2.4) were constructed and tested in simple support. For these two specimens the decks and the pour stops were welded to ½ in. thick by 4 in. wide steel plates at both ends which then rested on pin and roller supports as discussed in series #1 and depicted in Fig. 2.19.

All specimens in series #2 were built using the steel decks obtained from the same bundle as those in the full scale tests and therefore their strengths were identical. Because of space limitation, the specimens in series #2 were cast and tested in two separate batches. They can be recognized by the different concrete compressive strengths and densities as listed in Table 2.4.

The construction process and quality control was similar to the full scale and series #1 specimens. After casting, the specimens were left undisturbed until the test was carried out. The instrument locations were as shown in Fig. 2.19(a). The applied loads were recorded using 50 kips capacity load cells while all vertical deflections and end slips were recorded using wire pot displacement transducers. In series #2 tests, wire pots were used to measure the end slip as shown in Appendix B.2(e) and (f) instead of LVDTs and potentiometers because an interface problem was encountered between data logger and potentiometers.

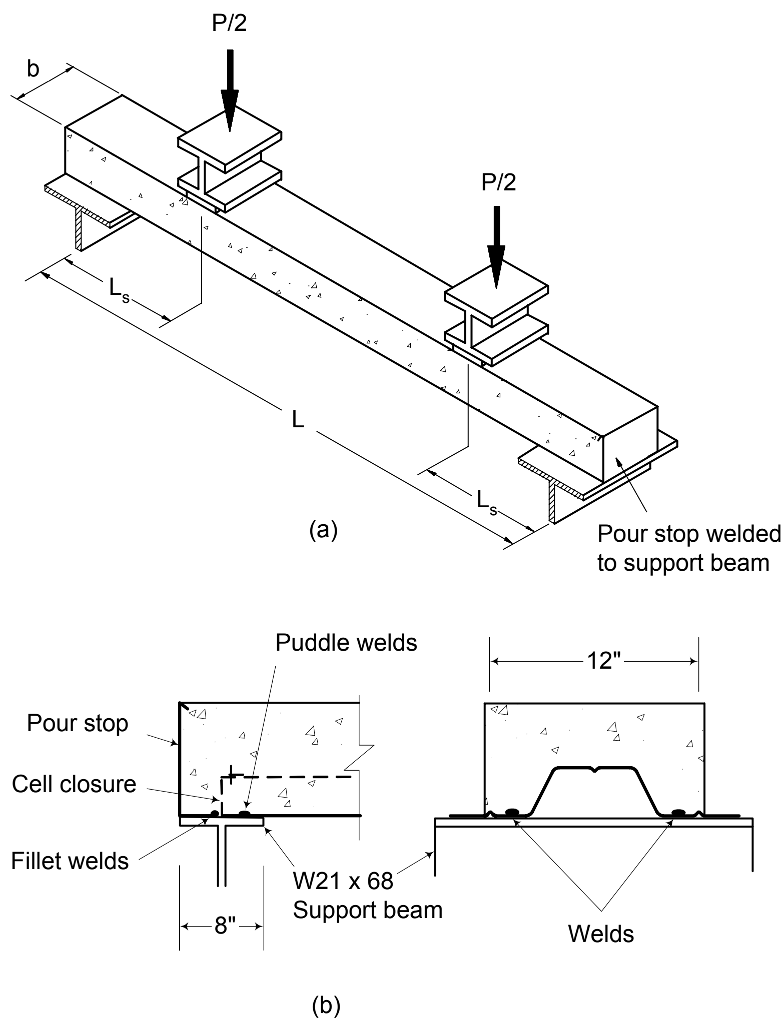


Fig. 2.20 Series #2 specimens
(a) Isometric view of test setup (b) Details at supports

Table 2.4 Test parameters for specimens in the series #2

Specimen number	Test ID representing the specimen parameters	Shear span measured on centers, (in.)	Concrete		Steel deck	
			Comp. strength, f'_c (psi)	Weight (pcf)	Yield strength, F_y (ksi)	Yield strength, F_y (ksi)
21	3VL20-8-7.5	32	5100	155	54	64
22	3VL20-11-5	40	5100	155	54	64
23	3VL18-8-7.5	32	5100	155	48	64
24	3VL18-13-5	52	4500	154	48	64
25	3VL16-4-7.5	16	5100	155	51	60
26	3VL16-8-7.5*	32	4500	154	51	60
27	3VL16-8-7.5	32	4500	154	51	60
28	3VL16-10-7.5	38	5100	155	51	60
29	3VL16-12-5	44	5100	155	51	60
30	3VL16-14-5	52	4500	154	51	60
31	2VL20-7-6.5	28	5100	155	52	62
32	2VL20-9-4	38	4500	154	52	62
33	2VL18-7-6.5	28	5100	155	49	59
34	2VL18-11-4	42	5100	155	49	59
35	2VL16-7-6.5	28	5100	155	47	58
36	2VL16-12-4	46	4500	154	47	58

Note :*Specimen #26 was supported by pin and roller.

2.8 Concluding remarks

The small scale specimens were very simple and easy to construct. The side pour stops, angle straps and C-clamps were reusable, which made the small scale experiment economical. Four small scale specimens could be setup on the same space needed for one full scale specimen.