prefabricated sandwich panels for bridge decks

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Based on the generally recognized superior strength-weight characteristics of sandwich panels, a study program was carried out to test a new type of compositely acting steel-concrete sandwich panel for use in bridge decks. Basically, the sandwich panel consists of two thin-faced plates of steel joined by a series of round shear-spacer studs welded to their inner surfaces. The core between the plates is made of lightweight concrete using expanding cement to induce a small prestress into the system. Laboratory tests were made on 10 small-scale panels, some loaded with a concentrated load and others with a uniformly distributed load. Mathematical theories developed for this type of panel show a generally satisfactory correlation between tests and theory. A number of solutions that take into account practical fabrication and erection problems are offered to illustrate how such panels can be bolted or welded, either longitudinally or transversely, across standard steel bridge girders. A comparative investigation indicated that these panels are substantially stronger and stiffer than normal reinforced concrete slabs that use the same quantity of concrete and steel.

•Since 1968, the Virginia Highway Research Council, with the cosponsorship of the Federal Highway Administration, has been engaged in the study and development of a new type of sandwich panel as might be used for prefabricated bridge decks. From the experience gained with sandwich construction in airframe and building industries, the use of sandwich construction to improve the strength-weight characteristics appeared promising. Inasmuch as bridge decks have requirements distinct from those of airframes and buildings, sandwich solutions appropriate to these requirements had to be developed. In particular, bridge decks must sustain unusually large uniform and concentrated loads, be durable under exposed conditions, and be relatively economical to construct. For this study, an additional requirement was imposed: The deck should lend itself to prefabrication to minimize the need for field construction.

After consideration of all these factors, a sandwich panel configuration (Fig. 1) was conceived. The details of fabrication and erection will be discussed later in this paper; at this time only a brief description of the panel will be presented. The top and bottom faces are thin steel plates connected intermittently by steel studs welded between them. (The protrusion of the studs on one side was done for ease of fabrication for the experimental test panels only. As will be explained later, this need not be done for prototype panels.) Side plates are welded around all edges of the panel. In the hollow between the face plates, lightweight concrete is placed to form a rigid core. This concrete can be pumped into place through small holes in the panel at either the fabricating plant or the construction site. Additionally, small bleeder holes should be provided in the top plate to ensure that during pumping all voids will be filled. (After filling, all holes in the plates would be sealed with welded steel cover plates.)

Although not absolutely essential, it is desirable to use an expanding cement in the concrete mix to induce a compressive prestress into the concrete (to reduce cracking) and a tension prestress into the steel face plates (to reduce plate buckling).

The purpose of the studs is to provide composite action between the concrete core and the steel face plates.

STRUCTURAL INVESTIGATIONS

Before the panels were tested, it was necessary to make a study of the expanding lightweight core concrete, inasmuch as no relevant information was available on the subject. In all, 170 test specimens were cast by using 10 concrete mix designs and seven percentages of steel reinforcing. The reinforcing steel simulated the elastic restraining effect achieved by the face plates in the actual panel as the concrete tends to expand. The aggregates used were expanded shale and sand. Basically, the test specimens were concrete prisms 3 by 3 by 11 in. (76 by 76 by 280 mm), with reinforcing steel positioned along the 11-in. length. Expansion readings were taken periodically for 8 weeks after casting. All curing was under autogenous conditions to simulate the moisture-sealed conditions of the prototype panels (1, 2).

The empirical equation found to express the percentage of expansion, r, is

$$r = K(p + q)^{-m} \tag{1}$$

where

$K = 0.012\ (w\text{-}c)^{-2.783}$,
$m = 0.488\ (w\text{-}c)^{-0.609}$,
$q = 0.003\ (w\text{-}c)^{-3.109}$,
p = percentage of steel reinforcing based on gross area, and
$w\text{-}c$ = water-cement ratio.

Figure 2 shows a typical curve relating expansion to percentage of steel restraint.

After the study of the core concrete material, 10 test panels were fabricated and tested. Their dimensions are given in Table 1, and their various stud configurations are shown in Figure 3. All panels were 25 in. (635 mm) square and were simply supported on all four sides by bearings placed 23.5 in. (597 mm) apart. The first seven panels were loaded by a concentrated load at the center by using a 2-in. (50.8-mm) square bar as shown in Figure 4. Figure 4 also shows the instrumentation used—dial gauges for deflection and electrical strain gauge rosettes for strain.

Panels loaded with the concentrated force characteristically failed by shear punching as shown in Figure 5. The steel face plates bent plastically but did not rupture. Removal of the face plates revealed that the concrete core did rupture by diagonal tension in the standard failure cone pattern. The studs near the failure zone underwent some distortion but did not rupture.

Given that the object of the tests was to determine the nature of the service load or elastic behavior, instrumental data were recorded only for such linear conditions. Plots of a typical load-strain relationship are shown in Figure 6. Strain readings are given for a point on the bottom of the panel under the load.

Figure 7 shows the test setup for loading the panel with a uniformly distributed force. Note that a steel box filled with dry sand was placed above the test panel in the hydraulic loading machine. The head of the loading machine bore on a stiff plate on top of the sand, which distributed load on the test panel. Three panels were tested in this manner.

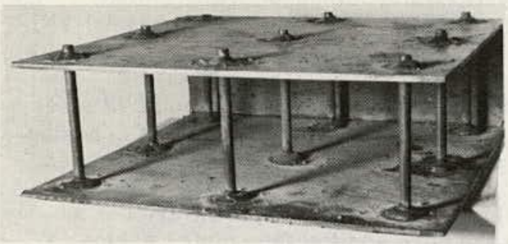


Figure 1. Cutaway of sandwich panel without concrete core.

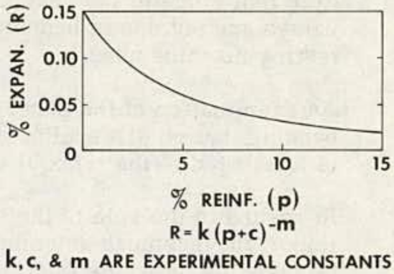


Figure 2. Typical expansion curve for expanding lightweight concrete.

Table 1. Dimensions of test panels.

Panel No.	Thickness (in.) Face	Core	Total	No. of Studs[a]
P_1	0.140	2.51	2.79	36
P_2	0.076	1.348	1.50	36
P_3	0.75	1.038	1.19	36
P_4	0.076	1.118	1.27	1
P_5	0.076	1.120	1.27	10
P_6	0.074	1.032	1.18	4
P_7	0.076	1.078	1.23	33
P_8	0.075	1	1.15	36
P_9	0.075	1	1.15	16
P_{10}	0.075	1	1.15	0

[a]Diameter of the studs was 0.25 in. except for panel P_3 with a diameter of 0.125.

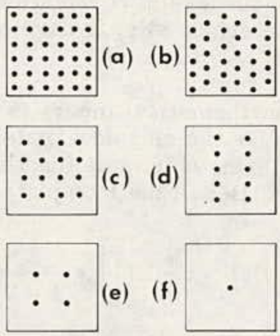


Figure 3. Pattern of studs in test panels

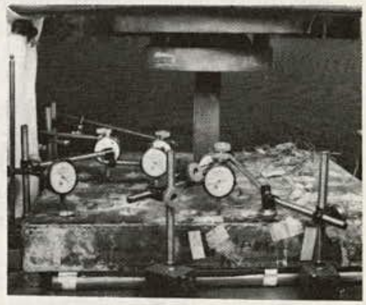


Figure 4. Test panel under concentrated load showing instrumentation.

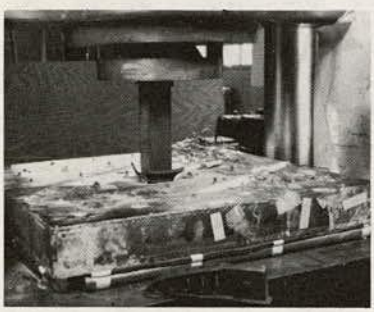


Figure 5. Failed test panel.

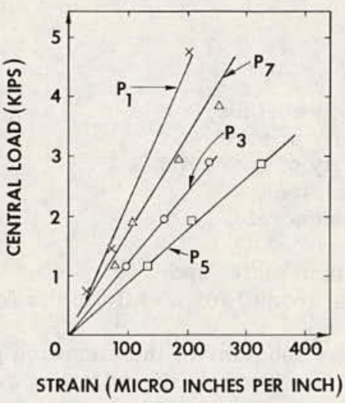


Figure 6. Load-strain readings for centrally loaded panels.

Figure 7. Test panel under uniform load.

A plot of the load-strain readings at the bottom center of the panel is shown in Figure 8. A related plot of load-deflection readings at the center of the panel is shown in Figure 9. Note that some of the readings are beyond the elastic limit, although ultimate load values are not shown because they went beyond the capacity of the 300,000-lbf (1300-kN) testing machine used.

An examination of the panel beyond the elastic limit showed some small amount of steel bending, but no discernible cracking or rupturing in the concrete. This lack of cracking is attributed to the triaxial restraint offered the concrete core by the steel on all sides.

In regard to the role of the studs at low loads, their number and placement are not too important inasmuch as composite action between the core and the face plates seems to be taken by interface friction. At loads beyond the elastic limit, interface friction breaks and the studs take over as the shear transfer mechanism. At higher loads, an increase in the number of studs causes an increase in the strength and stiffness characteristics.

Following the laboratory testing program, the sandwich panel was analyzed mathematically by using linear theory. Successful correlation of theory and test results depends on the assumption that some interface slip, S, develops between the core and the face plates (Fig. 10).

A complete derivation of the mathematical theory is given elsewhere (1, 2). Of interest is the fact that Eq. 2 is not unlike the classical plate equation used for homogeneous plates, except for a modifier term, which is a function of the slip and the respective dimensions and material properties of the face plate and core.

$$\left\{\frac{E_s}{1 - V_s^2}\left[t/2(T + t)(T + t - kT)\right] + \frac{E_c}{1 - V_c^2}(T^3/12)\right\}\nabla^4 W = P \tag{2}$$

in which

$$\nabla^4 W = \frac{\partial^4 W}{\partial x^4} + 2\frac{\partial^4 W}{\partial^2 x \partial^2 y} + \frac{\partial^4 W}{\partial y^4}$$

and where

- W = deflection,
- x, y = plate coordinates,
- P = normal load per unit surface,
- E_s = modulus of elasticity of steel,
- E_c = modulus of elasticity of concrete,
- V_s = Poisson's ration of steel,
- V_c = Poisson's ratio of concrete,
- T = thickness of core,
- t = thickness of each face plate, and
- k = slip factor (varying from 0 for no slip to 0.5 for full slip).

By using Eq. 2 to compare the behavior of the sandwich panels described with that of standard reinforced concrete deck slabs, the following results were found. Given the same working stress in the steel and concrete and the same quantity of steel and concrete, the sandwich panel is found to be 41 percent stronger than the concrete slab, yet it deflects 23 percent less, even at the greater load. The conclusion is that sandwich panels are substantially stronger and stiffer than comparable reinforced concrete panels, which bears out the original assumption that sandwich construction offers superior strength to weight characteristics, even for bridge decks. Inasmuch as one of the major weight factors in a normal bridge is the dead load of the concrete floor, any method of materially reducing this dead load is highly desirable.

BRIDGE APPLICATIONS

After demonstrating that sandwich panels are structurally valid for bridge decks, we propose practical means of fabricating and erecting such panels. Several such methods are suggested. In all cases, the following design conditions are assumed:

1. The panel is attached to steel stringers or girders;
2. The top surface of the panel is skid-proofed by either an epoxy mortar overlay or a thin layer of bonded asphalt;
3. The exterior steel used is either weathering steel or normal structural steel painted for corrosion resistance;
4. The panels are prefabricated in large sections for ease and rapidity of field erection;
5. Rails, poles, curbs, and so on are made so that they can be welded or bolted to the panels; and
6. The panels are adequately braced or supported in shipment to avoid damage.

Method I: Full-Length Longitudinal Panels Bolted to the Girders

Figure 11 shows how a longitudinal panel might be fabricated in a plant by using modern welding techniques. Note that the whole panel can be assembled in one position with all welding done downhand. Actually the panel is fabricated in a position opposite to the one it will have on the bridge. It is estimated that the steel plates will be on the order of $\frac{1}{4}$ in. (6.4 mm) thick, the studs $\frac{3}{8}$ in. (9.5 mm) in diameter and about 12 in. (305 mm) apart, and the core 4 to 6 in. (102 to 152 mm) thick. The entire panel is about 6 to 8 ft (1.8 to 2.4 m) wide and the full length of the span, or up to about 100 ft (30.5 m) maximum.

The sequence of fabrication is (a) automatic welding of the studs to the bottom plate (as with Nelson studs), (b) welding of the bolts to the bottom plate, (c) fillet welding of the side plates, (d) plug welding of the top plates to the studs in prepunched holes (the wide head on the studs allows for small fabrication errors), (e) pumping of the concrete through holes in the top plate, and (f) welding of the small cover plates over the pumping and bleeder holes.

Figure 12 shows how these panels are attached to the bridge girders by simple bolting. Such bolting not only secures the panel to the girder but also provides composite action between the girder and the deck panel. Should transverse continuity of the deck panels be desired (although not really necessary) the top face plates could be field-welded at their junctures as shown.

The voids between the panels can be hot sealed or pressure sealed with any number of materials such as bitumen, grout, urethane, or neoprene gaskets.

Method II: Full-Length Longitudinal Panels Welded to the Girders

In Figure 13, the fabrication shown is similar to that in method I except that the top plate is bent to form the sides as well, which avoids some welding. If a press or brake is not available to bend a very long plate, the edge strip can be cut at suitable intervals for accommodation by a short press. The cut can then be welded back together after bending.

Figure 14 shows how these panels can be attached in the field by slot welding from above. Semiautomatic welders are available for continuous straight run welding of this type, which requires little manual welding. By attaching the panels to the girders in this fashion, partial composite action can be effected between the girder and the panels. If composite action is not required, simple tack or intermittent welding of the panel to the flange can be done, with a savings in field welding.

Figure 8. Load-strain readings for uniformly loaded panels.

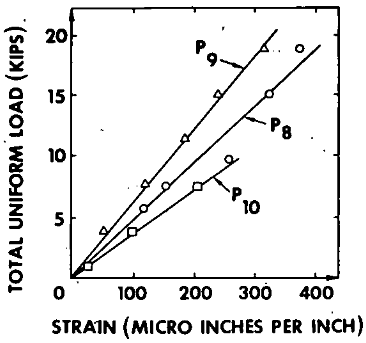


Figure 9. Load-deflection readings for uniformly loaded panels.

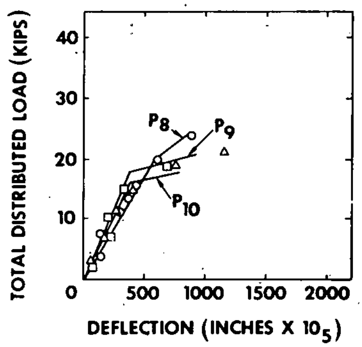


Figure 10. Slip at interfaces.

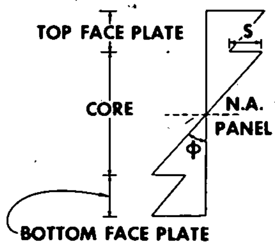


Figure 11. Method I longitudinal panel.

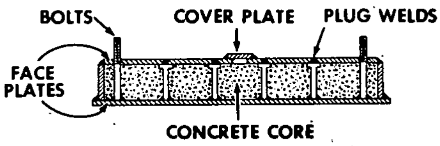


Figure 12. Attachment of panels to bridge (method I).

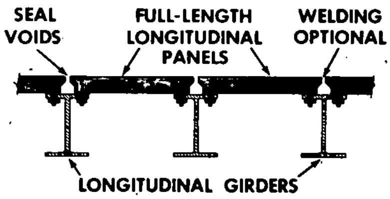


Figure 13. Method II longitudinal panel.

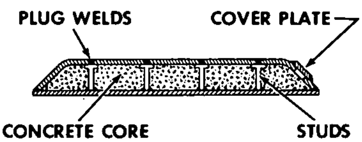


Figure 14. Attachment of panels to bridge (method II).

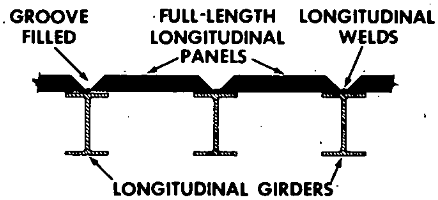


Figure 15. Method III transverse panel.

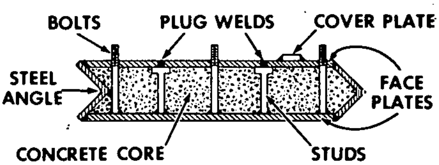
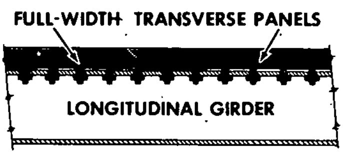


Figure 16. Attachment of panels to bridge (method III).



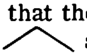
If the panels are topped with a wearing surface of asphalt, the grooves between panels can be filled with the same material at the same time to level out the roadway surface.

Method III: Full-Width Transverse Panels Bolted to the Girders

The basic method of fabrication for transverse panels shown in Figure 15 is similar to that described in method I except that the long side pieces are rolled angle sections rather than flat plates. The angles provide a shear transfer joint when the panels are erected on the girders. (Note that the sharp leading edge of the angle is to be ground off so that it can fit snugly with its mating angle as shown in Fig. 16.) Field welding of this joint is optional.

Pumping and bleeder holes are located in regions away from the bolts so that they will not interfere with attachment to the girders.

These panels can be 4 to 8 ft (1.2 to 2.4 m) wide and up to 100 ft (30.5 m) long, or the full width of the bridge.

Figure 16 shows how these transverse panels are mated and attached to the girders by bolts. For ease of erection, holes in the girder flanges should be enlarged or slotted. It is recommended that the transverse cross slope (for drainage) be along an arc of a circle rather than  so that the natural flexure of the panel can adjust to the curvature more easily. However, even with an arc, tapered washers would probably be needed to secure a good connection between the panel and the girder flange. The panels, if properly gripped, provide for composite action between the girder and the deck. Transverse continuity of the deck panels would, of course, be automatically established.

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