Development and Testing of a New Shear Connector for Steel Concrete Composite Bridges

Wayne S. Roberts and Robert J. Heywood, *Queensland University of Technology, Australia*

A new cross section has been developed for steel concrete composite bridges that eliminates the top steel girder flange. This is made possible by utilizing a recently developed shear connector known as the Perfobond Strip. This shear connector provides a stiff connection between steel and concrete and reportedly has excellent resistance to fatigue. Because the success of this new cross section and numerous other applications depends on the performance characteristics of the shear connector, the present design models were investigated. Some inconsistencies were found between current design models and experimental results; the results of a series of shearbox tests that have led to the development of a new design model are included. This new design model is compared with current models. Some details of a full-scale bridge test are also included to examine the fatigue behavior and overall performance of the new bridge cross section.

Steel concrete composite construction is re-emerging as a competitive form of construction for bridges. This is because of the availability of higher yield strength steels, automated fabrication methods, and the development of new coating systems to resist corrosion (*1*). Permitting tensile stresses in the concrete deck provided crack widths are controlled and research into the behavior of shear connectors have encouraged production of more cost-efficient designs (*2*). The focus of current research in composite structures is to increase the understanding of shear connector behavior and to develop designs that are more cost-effective through efficient use of materials.

A new shear connector known as the Perfobond Strip has been developed by Leonhardt and Partners in Germany (*3*). It consists of a steel strip with holes punched in it that can be welded to the top flange of steel I-sections. Concrete is cast through the holes, forming a series of concrete dowels that resist the shear flow.

The top steel girder flange contributes little to the strength of the composite section because of its proximity to the neutral axis. Its main function is to provide stability during the construction process and a location for the attachment of shear studs. Although designers have minimized the size of the top flange, it would be more efficient for fabrication if it were eliminated. Knowles has stated that this would be possible if researchers could find a suitable method of shear connection that did not require a top flange for the shear connection to function effectively (*4*).

The Perfobond Strip shear connector can be incorporated into a composite cross section that eliminates the top steel girder flange and the shear studs (*5*). This

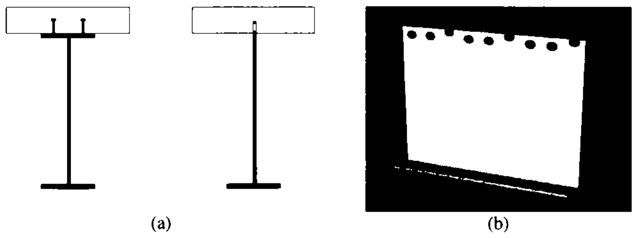


FIGURE 1 *(a)* **Comparison of traditional and new composite cross sections;** *(b)* **inverted steel T-section used in the new cross section.**

cross section, illustrated in Figure 1, leads to savings in steel fabrication costs because there is no top flange and no weld between the top flange and web plate. Although the web plate is a little deeper in the new cross section, fabrication time is significantly reduced. Instead of shear connectors being welded to the top flange, holes are punched in the top of the web. Figure 1 compares the traditional composite cross section with the inverted T cross section.

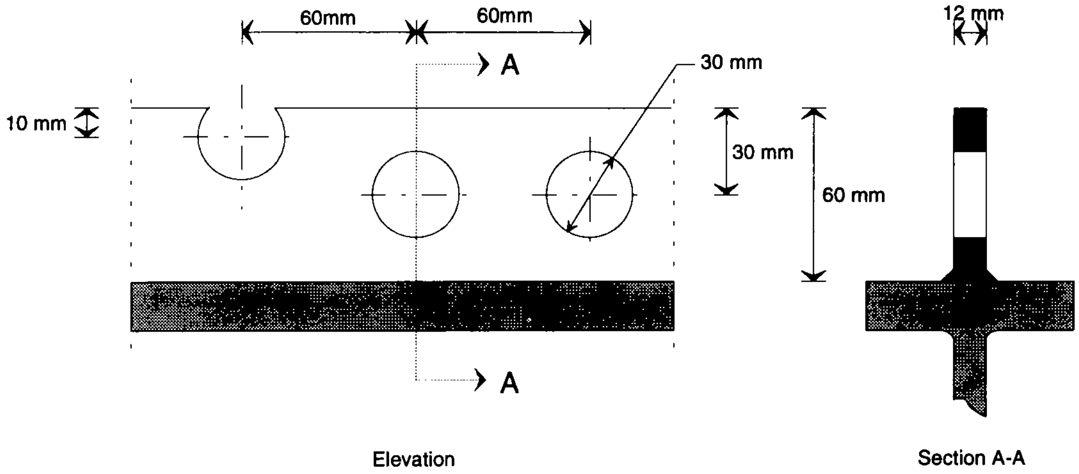
The inverted T cross section requires appropriate handling and construction techniques because the T section is unstable before the concrete deck is complete. While the inverted T alone is sufficiently stable during handling, it is not able to carry construction loads. Construction methods involving erecting the section with the concrete deck precast onto the steel girder have been investigated and found to be feasible. A new bridge cross section has been developed using precast steel and concrete composite T-beams (*6*). Incremental launching with the concrete deck cast in place is also a feasible option.

One disadvantage of the inverted T cross section is that deck replacement would be more difficult. This is not a problem in Australia as the climatic conditions are not as severe as they are elsewhere and deck replacement is unusual.

For the inverted T cross section to be successful, the performance of the shear connection in this application is vital. This paper reports research aimed at verifying the performance of this new method of shear connection in the absence of the steel girder top flange.

PERFOBOND STRIP

Figure 2 illustrates the typical dimensions of the Perfobond Strip. The advantage of the Perfobond Strip shear connector is that it behaves like a rigid connector at working stress levels and it does not deform like shear studs. A comparison of the static performance of shear studs and the Perfobond Strip, both with steel girder flanges, is presented in Figure 3 (*7*). The comparison is based on pushout test results using six shear studs 19 mm in diameter and ten holes 30 mm in diameter at 80-mm centers. The strip provides a slightly stiffer connection before the ultimate load is reached and maintains up to 80 percent of its load after 15 mm of slip. The load for the shear studs begins to fall off after 10 mm of slip as individual studs shear off. The fatigue performance of the Perfobond Strip has also been investigated by Leonhardt et al. (*3*). After 2 million load cycles at 40 percent of ultimate load, the recorded slip for the Perfobond Strip shear connector was 0.14 mm, whereas the slip for the shear stud was significantly higher at 1.5 mm. At working stress levels the Perfobond Strip shear connector does not deform and is therefore a rigid means of shear connection. Therefore, it is not as prone to fatigue problems, which limit the



Elevation

Section A-A

FIGURE 2 The Perfobond Strip.

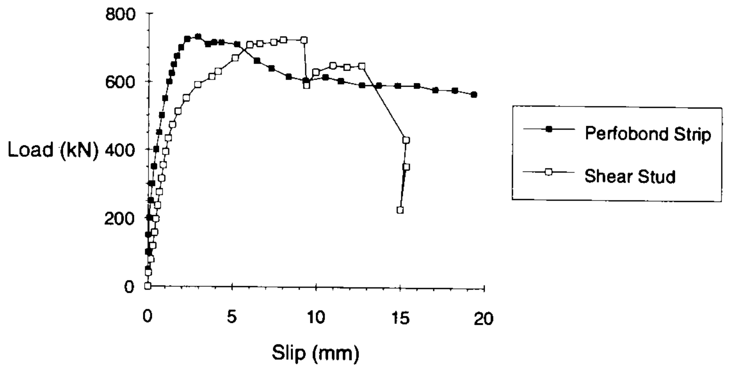


FIGURE 3 Comparison of shear studs and the Perfobond Strip (*7*).

service loads carried by shear studs. Figure 4 illustrates the fatigue test results.

Present Design Equations

Leonhardt et al. have presented design equations for the Perfobond Strip (*3*). The desired failure mode is by shearing of the concrete dowels, and the resulting ultimate capacity is given by Equation 1. *SF* is the ultimate shear force per hole.

$$SF = 2 \times \frac{\pi D^2}{4} \times 1.6 f'_c \qquad (1)$$

The equation is essentially the hole area where *D* is the diameter, multiplied by the shear strength developed in the concrete, which is given by the constant 1.6 (shear strength parameter) times the concrete strength (f'_c). This value was given as 1.3 in Leonhardt's paper but has been converted to 1.6 so that the cylinder strength can be used instead of the cube strength. This is multiplied by two as there are two shear planes per hole. A strength reduction factor $\phi = 0.7$ is applied to calculate the design shear force SF^*. Other modes of failure include failure of the concrete dowels by bearing and shearing of the steel strip between the holes; these can be avoided by ensuring appropriate hole sizes and spacings for the plate thickness used.

There is also a requirement that reinforcing steel be provided transverse to the strip to confine the concrete around the strip to ensure that the concrete in the hole is confined in three dimensions. The requirement for this transverse steel is further illustrated using a strut tie analogy in Figure 5. The amount of reinforcing required is calculated using Equation 2 where A_{st} is the area of steel required, SF^* is the design shear force per concrete dowel, and f_{sy} is the yield strength of the reinforcing:

$$A_{st} \geq \frac{0.8 \times SF^*}{f_{sy}} \qquad (2)$$

Oguejiofor and Hosain have also presented a design equation (Equation 3) for the Perfobond Strip (*8*). It is based on an application for beams using 375-mm

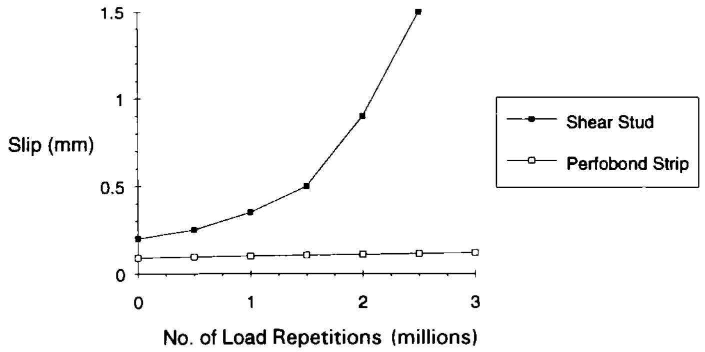


FIGURE 4 Fatigue comparison between shear studs and the Perfobond Strip (*3*).

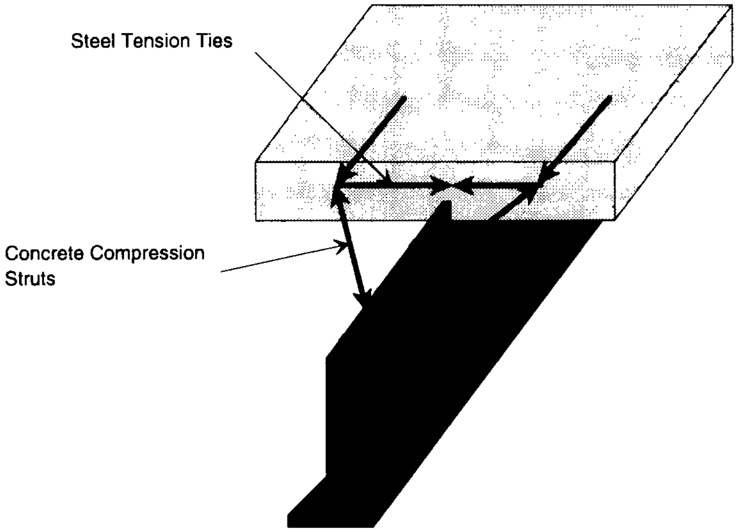


FIGURE 5 Internal forces associated with the Perfobond Strip shear connector.

lengths of strip. Consequently it allows for end bearing of the strip on the concrete. Their equation is based on the results of pushout tests that failed because of splitting of the slab along the line of the connector. This failure mode resulted from the use of lighter transverse reinforcing typical of building applications. The first term is related to the splitting of the concrete where A_{cc} is area of concrete in the plane of the connector. The second term accounts for the degree of confinement from the transverse reinforcement where A_{rt} and f_y are the area and yield strength, and the last term gives the shear strength of the concrete dowels where A_{bs} is the total area of the dowels in shear. In this case SF is the ultimate shear force per connector, as follows:

$$SF = 0.6348A_{cc}\sqrt{f'_c} + 1.1673A_{rt}f_y + 1.6396A_{bs}\sqrt{f'_c} \qquad (3)$$

The differences between the methods of Leonhardt (*3*) and Oguejiofor and Hosain (*8*) revolve around the various failure modes that result from the differences in the transverse reinforcing used. Leonhardt et al. use the concrete strength, whereas Oguejiofor and Hosain use the square root of the concrete strength. Clearly there are some inconsistencies in the methods available for the design and prediction of the Perfobond Strip capacity. This is to be expected as the current theories are based on the results of pushout tests of varying configurations.

Pushout Testing

A series of pushout tests was conducted to verify the performance of the Perfobond Strip and to investigate the performance of the connector without a top steel girder flange. The configurations used in the program are illustrated in Figure 6. Tests 1 and 2 consisted of two strips with holes 30 mm in diameter at 80-mm centers with the strips welded to a top flange in Test 1 and with no flange in Test 2. Test 3 consisted of holes 30 mm in diameter at 50-mm centers with no flange. The transverse reinforcing was typical of Leonhardt's requirements. The strain in the transverse reinforcing was recorded during the tests.

The results of the testing program are illustrated in Figure 7. The initial data recorded were load slip data, which have been converted to shear strength parameter versus slip. The shear strength parameter was outlined in reference to Equation 1. Plotting the shear strength parameter permits the comparison of test results with various concrete strengths and hole diameters. The shear strength parameter is representative of the shear strength being developed in the concrete dowels. The straight line on the graph represents Leonhardt's value of the shear strength parameter at ultimate, which is used in Equation 1.

A number of conclusions can be drawn from the results of this test series. By comparing the results of Tests 1 and 2 it is clear that the Perfobond Strip remains functional without the top flange. The initial stiffness is similar, but there is some reduction in ultimate strength that could be allowed for in the design process. This is because the concrete is not confined around the strip by the flange. Comparing the results of Tests 2 and 3, in which the same number and size of holes were used, showed that there was a drop-off in load when the holes were closer together. However the area of steel in the

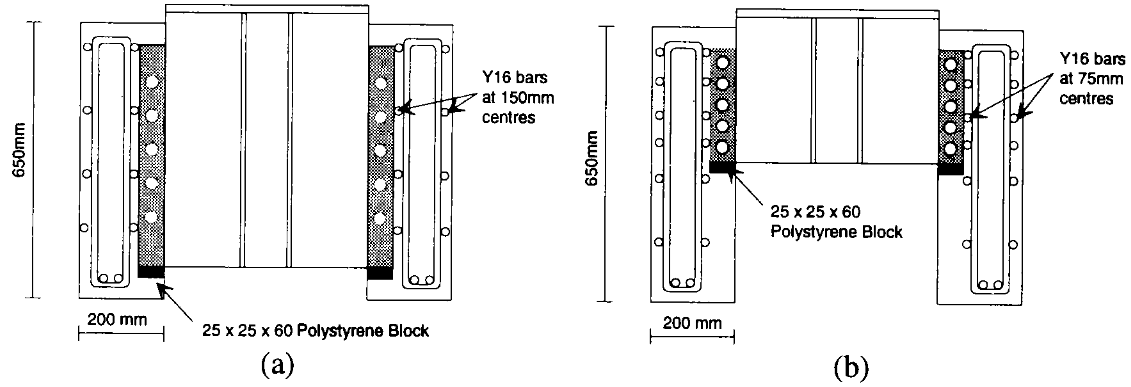


FIGURE 6 Pushout test configuration (*a*) for Test 1 (with flange) and Test 2 (without flange) and (*b*) for Test 3.

TABLE 1 Pushout Test Failure Loads and Calculated Failure Loads

			Calculated Failure Load (kN) (Experimental Failure Load as a percentage of calculated load)		
Test No.	Concrete Strength (MPa)	Experimental Failure Load (kN)	Leonhardt et al	Oguejiofor and Hosain	Shearbox Equation
1	30	732	678 (107%)	2686 (27%)	471 (155%)
2	33.5	640	758 (84%)	2724 (23%)	451 (142%)
2A	34	640	769 (83%)	2729 (23%)	361 (177%)
3	36	470	814 (57%)	2866 (16%)	384 (122%)

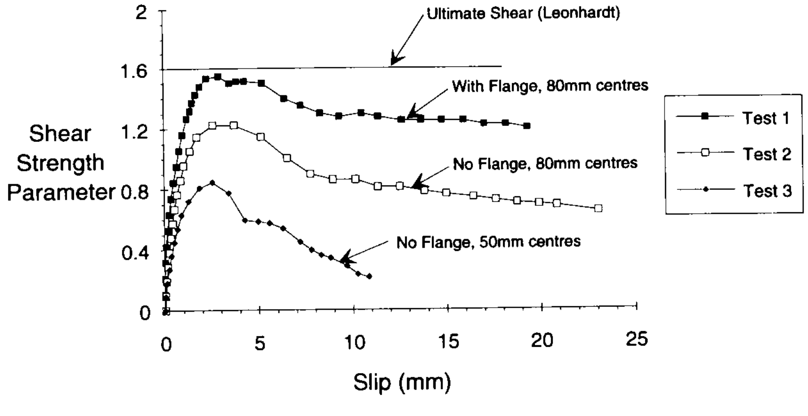


FIGURE 7 Graph of shear strength parameter versus slip for Perfobond Strip pushout tests.

strip without holes in it also decreases. This result highlights a problem with both of the currently used equations in that they do not consider the effect of friction between the surrounding plate and concrete.

Shear Box Testing

To investigate some of the inconsistencies highlighted by the pushout testing program, a series of shear box tests was undertaken. The principal aims of these tests were to investigate whether friction between the plate and concrete contributed to the strength of the shear connection, the effect of varying hole diameters, and the influence of the confining force provided by the transverse reinforcement. The test specimens consisted of plates 12 mm thick with varying hole diameters cast into concrete. The specimens were then sheared along one interface between the plate and the concrete. Figure 8*a* illustrates the different plates used in the investigation. The shaded area indicates the area of plate in contact with the concrete. Figure 8*b* illustrates a test specimen.

The investigation involved a total of 60 samples in five sets. Hole diameter was varied between 0 and 40 mm. The confining force was varied between 0 and 60 kN (0 and 4 MPa). Some typical shear force (load) versus slip curves are presented in Figures 9 and 10. The legend code describes the sample and test characteristics. D30CF0S3 denotes a diameter of 30 mm, a confining force of 0 kN, and that the test was from Series 3. Figure 9 illustrates tests with a common diameter of 30 mm with varying confining forces, whereas Figure 10 presents the results for varying hole diameters with constant confining force. In general, shear strength increases with increasing hole diameter and increasing confining force. The bond between the plate and concrete contributes to the strength indicated by result D0CF40S5 and the connector produces some strength with no confining force, which indicates a cohesion effect.

Analysis of Results

The factorial method of analysis is a method used to evaluate experiments in which interaction between variables is expected in experiments (9). A factorial analysis on the shear box test data indicated that both confining force and hole diameter influenced the results but no interaction occurred between the two.

A suitable strength model was required to describe the results from the shear box testing. The Mohr Coulomb soil shear strength model summarized by Equation 4 was chosen because it describes the shear strength of materials that have cohesion and friction components. This model allows the shear strength to vary with the applied stress normal to the shear plane, which was typical of the results from the shear box tests.

$$SF = C + \sigma_n \tan \phi \tag{4}$$

The results were segregated into the four different hole diameters so that the principal variable was the stress normal to the shear plane. To account for various concrete strengths the shear strengths have been divided by the square root of the concrete strength. The square root was used as shear failures and is related to the tensile strength of the concrete, which is related to the square root of the concrete strength. The data were plotted with shear force on the vertical axis and normal stress on the horizontal axis. Using linear regression

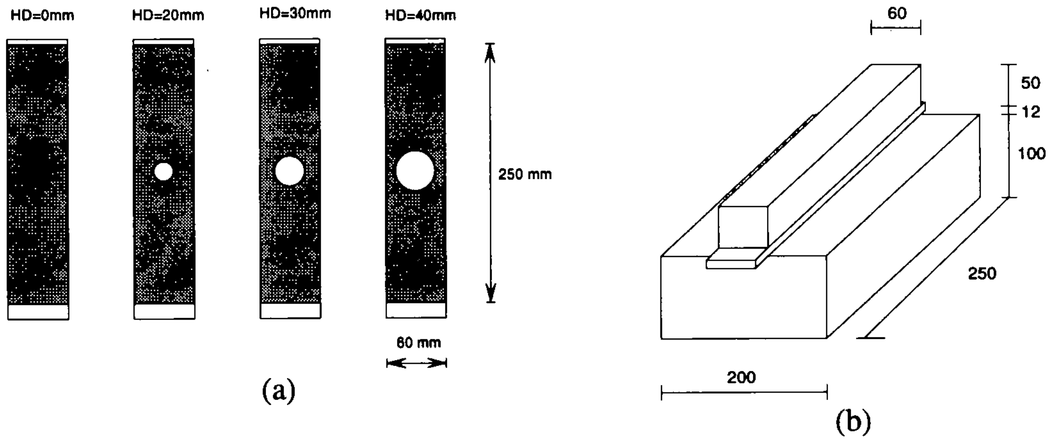


FIGURE 8 *(a)* **Steel plates with varying hole diameters;** *(b)* **sample use in shear box investigation.**

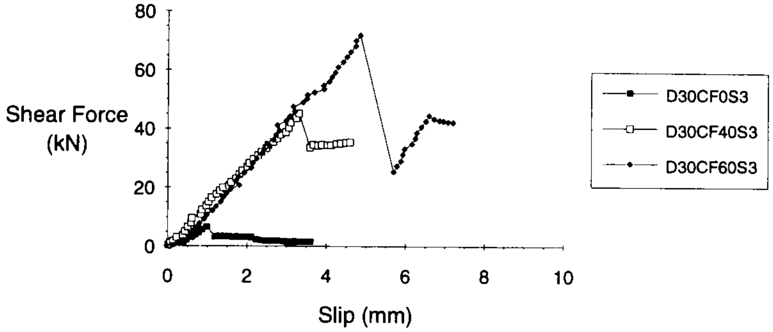


FIGURE 9 Typical shear force versus slip curves for Series 3.

analysis, four lines were plotted for each of the hole diameters. These lines are illustrated in Figure 10. Although there is some variation in the data, all regression lines fitted the data with an r^2 value of 0.9 or higher. The result for the hole 20 mm in diameter appears to be inconsistent with expected results. This inconsistency may be related to the influence of concrete aggregate in the hole. More testing would confirm the influence of aggregate in the hole. The ratio of hole area to plate area is also not typical of applications using the Perfobond Strip. A smaller plate size should be used for any subsequent testing.

Equation 5 represents the results of the tests on the basis of the regression lines in Figure 11 except for the 20-mm-diameter hole. The equation consists of a cohesion and friction angle component for the bond between the concrete and the steel plate and for the concrete to concrete interface in the concrete dowel as follows:

$$SF = \sqrt{f'_c}[A_p(0.046 + 0.15\sigma_n) + A_h\,\{(2.1 - 0.00055A_h) + (-0.079 + 0.00029A_h)\sigma_n\}] \quad (5)$$

here

A_h = the hole area;
A_p = the plate area in contact with the concrete less the hole area;
σ_n = the stress normal to the plate; and
f'_c = the concrete strength.

The quantities A_h and A_p are illustrated in Figure 12. *SF* in this case is the shear force per shear plane. The units are in millimeters, megapascals, and newtons. This equation allows the calculation of the shear strengths for hole sizes between 30 and 40 mm at any spacing. Further verification is required before this equation could be used for any size hole. The effect of hole spacing on the shear strength has not been considered in this investigation.

To equate the results of the shear box testing program back to the pushout testing results, it is necessary to quantify the stress normal to the plate (σ_n), which in the case of pushout tests is developed by the transverse reinforcing (see Figure 5). Using the average strain measured in the reinforcing bars, a stress normal to the connector was calculated and used in Equation 5 to derive a failure load for the pushout tests. The results are included in Table 1 along with calculated loads from Leonhardt's Equation 1 and Oguejiofor and Hosain's Equation 3. Equation 3 has been used out of context in this situation as the tests in this study used polystyrene blocks to prevent end bearing. Test 2A was identical in configuration and failure load to Test 2 but lower strains were recorded in the reinforcing bars. With the

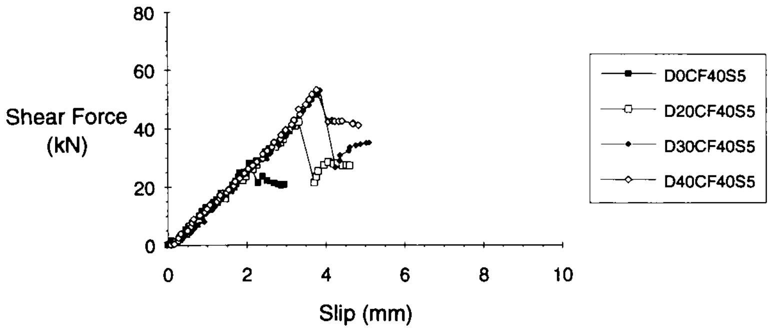


FIGURE 10 Typical shear force versus slip curves for Series 5.

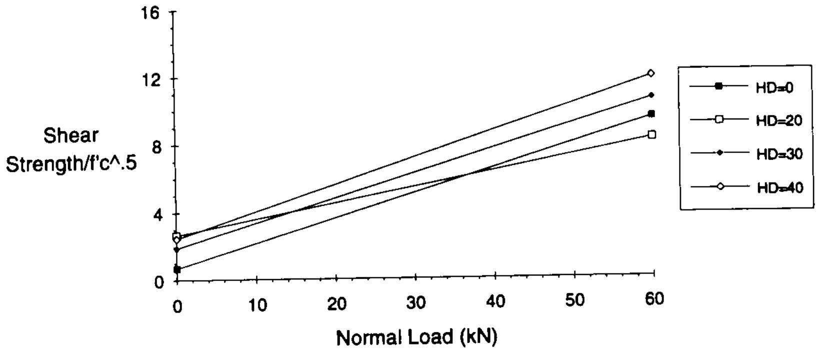


FIGURE 11 Results of linear regression.

shear box method this leads to a lower calculated value for failure load. The actual failure load as a percentage of the calculated load is indicated in brackets.

These results indicate that Equation 1 is inadequate for Test 3, and Equation 2 should not be used for heavily reinforced applications typical of the tests in this paper. The results from Tests 2 and 3 are encouraging for the shearbox equation. In all cases the shearbox equation underestimates the failure load. Further investigation into the relationship between confining force and reinforcement quantities should lead to a more accurate method for the design of the Perfobond Strip.

This work has illustrated the difficulty of extrapolating the results of shear connection test data outside the limits of experimental data. In this exercise methods have been compared with less accuracy than is desired, and it has shown the importance of operating within the confines of experimental data when considering shear connector behavior.

Full-Scale Bridge Test

Although the results of pushout tests and other small-scale tests are an ideal method for comparing the performance of various methods of shear connection and for investigating the effect of various parameters on the performance of shear connection, pushout tests do not reproduce the behavior of shear connectors in a structure because of the presence of more complex stress states.

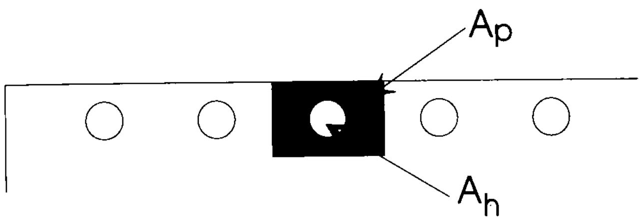


FIGURE 12 Plate and hole areas used in Equation 5.

To investigate this and to answer some questions about the performance of the Perfobond Strip without a top flange in a structure, a full-scale bridge section was constructed. The principal aims of this test are to verify the claimed fatigue performance of the Perfobond Strip and to examine the effects of stress concentrations in the tension zone of the girder because of the strip holes. Other issues to be examined are the overall performance of the concept, the performance of the deck slab without a top steel girder flange, and the performance of the shear connector particularly in negative moment regions where the slab is transversely cracked. Figure 13 illustrates the bridge that was designed. The deck slab was designed in accordance with the requirements of the Ontario Highway Bridge Design Code (*10*). The shaded section indicates the section that was constructed for testing. Figure 14 illustrates the cross section that represents one design lane of the structure. This section was subjected to 500,000 cycles of loading, with each cycle equivalent to a T44 design truck plus impact allowance, which is the AUSTROADS requirement for fatigue testing (*11*). The bridge section showed

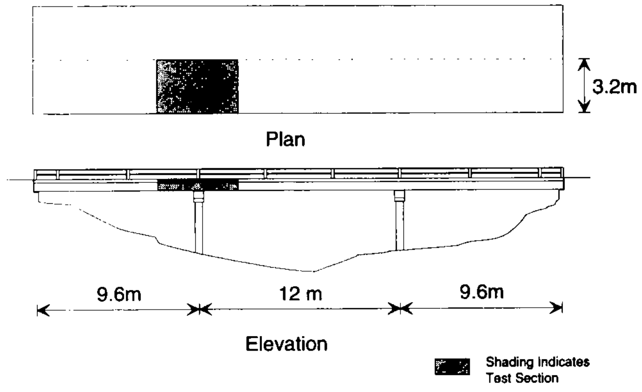


FIGURE 13 Details of full-scale bridge test.

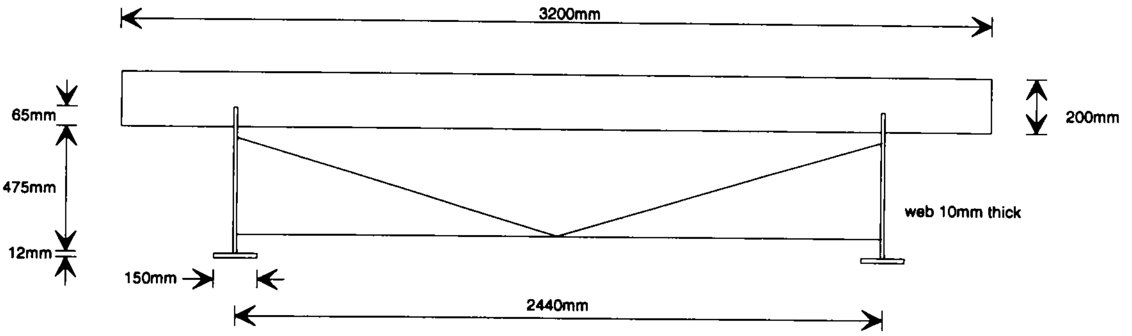


FIGURE 14 Cross section of full-scale bridge test.

no measurable signs of deterioration during the 500,000 cycles. The section also performed satisfactorily under ultimate design loads. An ultimate wheel load was also applied to the slab during testing to investigate load transferral from the slab into the web. No relative displacement occurred between the slab and the girder during this test.

CONCLUSION

This paper has introduced an innovative bridge cross section that utilizes a new shear connector and eliminates the top steel girder flange. An evaluation of the currently existing design theories for the Perfobond Strip has indicated inconsistencies with these design methods related to the transverse reinforcing used and the resulting failure modes. A series of pushout tests concluded that the shear connector was functional without a top steel girder flange and highlighted the fact that the strength of the concrete to plate bond does contribute to the strength of the shear connection.

A series of shear box tests was conducted and a design equation was developed that takes into account the strength of the concrete to plate bond as well as the strength of the concrete dowels in shear. This equation was related to the pushout tests using the recorded strain in the reinforcing bars to calculate a stress normal to the connector, which illustrated that more work is required in relating the reinforcing to the normal stress that it produces.

With the results of the completed work and the findings from the full-scale bridge test, an innovative new steel concrete composite bridge design, as outlined in this paper, should be feasible. This design will lead to more economically competitive composite designs, which should be more competitive in the market against existing prestressed concrete solutions.

REFERENCES

1. Abe, H., H-P. Andra, R. Gruter, J. Haensel, P. Ramondenc, and R. Saul. Steel Composite Railway Bridges. *Structural Engineering International*, Vol. 2, No. 4, 1992, pp. 259–267.
2. Zellner, W. Recent Designs of Composite Bridges and a New Type of Shear Connector. *Proc., Engineering Foundation Conference Composite Construction in Steel and Concrete* (C. Buckner and I. Viest, eds., New Hampshire, 1987, pp. 241–252.
3. Leonhardt, E. F., W. Andra, H-P. Andra, and W. Harre. New Improved Shear Connector With High Fatigue Strength For Composite Structures (in German). *Beton—Und Stahlbetonbau*, Vol. 12, 1987, pp. 325–331.
4. Knowles, P. R. *Composite Steel and Concrete Construction*, Butterworths, London, 1973.
5. Roberts, W. S., and R. J. Heywood. An Innovation To Increase The Competitiveness Of Short Span Steel Concrete Composite Bridges. *Proc., 4th International Conference on Short and Medium Span Bridges. Developments In Short and Medium Span Bridge Engineering '94* (A. A. Mufti, B. Bakht, and L. G. Jaeger, eds.), Halifax, Nova Scotia, Canada, 1994, pp. 1161–1171.
6. Roberts, W. S., and R. J. Heywood. Innovations in Short Span Steel and Concrete Composite Bridges. *Proc., AUSTROADS Bridge Conference on Bridges Essential To Our Economy*, Melbourne, Australia, 1994, pp. 40(1)–40(10).
7. Roberts, W. S., and R. J. Heywood. Shear Connector For Composite Structures. *Physical Infrastructure Centre Digest*, Vol. 3, No. 2, 1992, pp. 1–3.
8. Oguejiofor, E. C., and M. U. Hosain. Perfobond Rib Connectors For Composite Beams. *Proc., Engineering Foundation Conference on Composite Construction in Steel and Concrete II* (W. S. Easterling and W. M. Roddis, eds.). Potosi, Mo. 1992, pp. 883–898.
9. Orr, D. M. F. Factorial Experiments in Concrete Research. *ACI Journal*, Vol. 69, No. 5, 1972, pp. 619–624.
10. Ontario Ministry of Transport and Communication. *Ontario Highway Bridge Design Code*, Toronto, 1983.
11. *Bridge Design Code*. AUSTROADS, Sydney, Australia, 1992.