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Fatigue Strength of Form-Reinforced Composite Slabs for Bridge Decks

Résistance à la fatigue de plaques composites en béton armé pour tabliers de ponts

Ermüdungsfestigkeit von Stahlbeton-Verbundplatten für Brückenfahrbahnen

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Introduction

There are several advantages in using corrugated steel permanent formwork for the construction of concrete bridge decks and floor slabs, rather than conventional timber or steel formwork. Erection is quicker, and no shoring or dismantling are required. If adequate shear connection is provided, the formwork can also act as reinforcement for the deck slab.

In North America, corrugated steel decking is widely used as formwork for floors in buildings, and is designed to act compositely with the concrete. Design is usually based upon safe-load tables supplied by the decking manufacturer, who derives them empirically from his own tests. There is little published information on fundamental research into this type of construction. In the only report on fatigue behaviour known to the authors [1] results of six tests are presented, but insufficient data are given to permit their analysis in terms of bond and bearing stresses on dimples in the shear spans.

An experimental study of the stiffness and static and fatigue strength of composite slabs reinforced with two types of corrugated steel decking is summarised in this paper, and recommendations for design are given. A full report on the work is available [2].

The primary aim of the tests was to find out if "form-reinforced" concrete slabs are suitable for use in the decks of highway bridges. The design of the test specimens was based on criteria laid down in 1970 by the Bridges Engineering Division of the Department of the Environment. Those that influenced the design of the test specimens are:

1. Range of spans, 1.8 to 3.6 m (6 ft to 12 ft).
2. Overall depth of slab not less than 0.18 m (7 in).
3. The metal decking alone must be capable of carrying its own weight, that of the concrete slab, and in addition a load of 9 *kN* (2,000 lb) applied over a circular area 0.6 m (2 ft) in radius.

4. The mature form-reinforced slab must be capable of carrying standard HA and 45 units of HB highway loading as specified in B.S. 153 [3], in accordance with CP 117: Part 2 [4]. The HB loading is equivalent to a 16-wheel truck of total weight 1.8 MN (400,000 lb), for which 25% overstress is allowed. The standard HB (and HA) wheel load is thus 90 kN.

Test Specimens

Metal decking. Initially, the use of pressed steel trough sections with welded steel connectors was studied. If the ratio of connector diameter to sheet thickness is kept below the recommended limit [4] of 1.5 a forest of very small connectors is needed, and the design is not competitive. Where fatigue is a problem, a cheaper alternative can be obtained by using pressed dimples to provide shear connection, as shown in Figs. 1 and 2. Resistance to uplift forces is provided by sloping the dimples on opposite sides of each trough in opposite directions.

In practice, the thickness of the metal decking used in bridges is likely to exceed that determined from considerations of stiffness or strength, in order to provide for some loss of cross-section due to corrosion. It was concluded that the requirements for bridge decking would be met by a galvanised section 4 mm thick and 75 mm deep, with a 'wavelength' of 300 mm.

It was not found possible to have a few pieces of this decking fabricated at an acceptable cost, since special tools would be required to produce the dimples, and time was limited. It had been found so difficult to obtain dimpled sheeting that other designs were studied. Calculations showed that under maximum loading the range of longitudinal stress at the top of the corrugations of the prototype

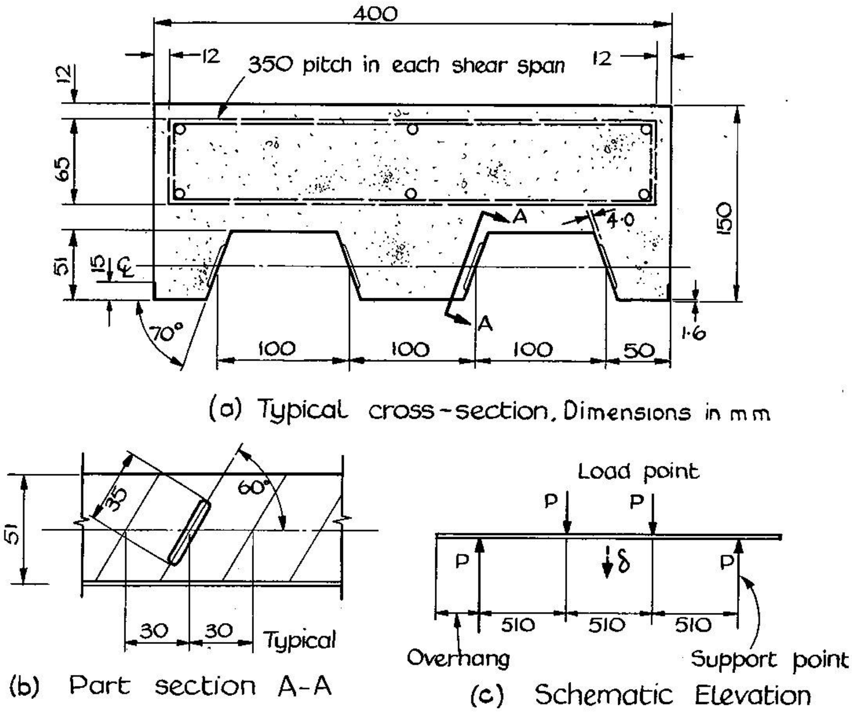


Fig. 1. Typical details of specimens DS1 to DS4.

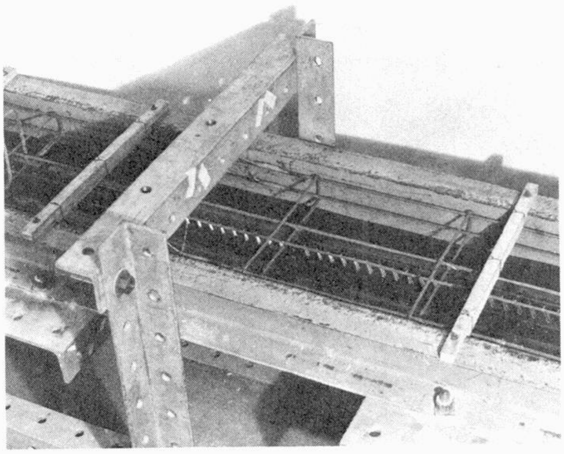


Fig. 2. Specimen DS3 before casting.

is quite small. This suggested that shear connectors could be welded to the tops of the corrugations. Thus the first two profiles tested (Figs. 1 and 3) were determined by the fabrication facilities readily available. All linear dimensions are two-thirds full size, except the sheet thickness (1.6 mm). In the first group of four specimens (DS1 to 4), the dimples (Fig. 2) were pressed individually. The second group of specimens, MS1 to 4, were made to the same design as Group 1, except that welded-bar shear connectors (Figs. 3 and 9) were used in place of dimples.

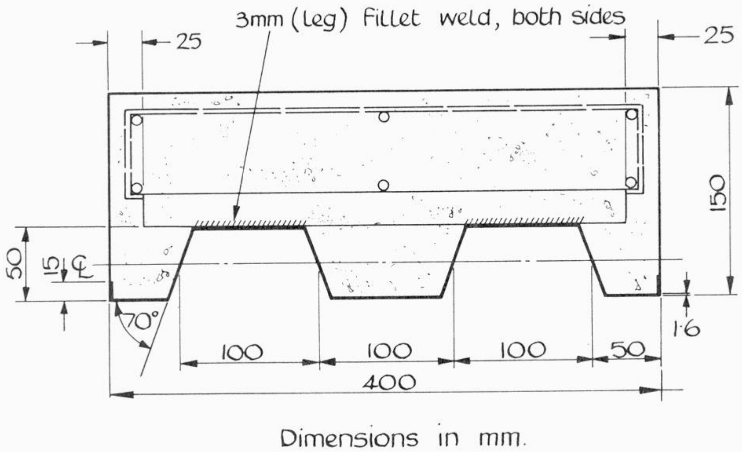


Fig. 3. Typical cross section of specimens MS1 to MS4.

The maximum length of plate that could be corrugated was 1.22 m. Longer units were required for the tests, so 2.44-m lengths were made by butt welding the shorter pieces together. This was not a preferred method of fabricating specimens for fatigue tests, but only one test result, to be described later, was influenced by it.

A third group of specimens (DS5 to 14) using proprietary galvanised decking made in North America was also tested. The corrugated sheets were similar to those of Group 1, except that the dimples projected about 0.9 mm into the concrete, instead of 4.0 mm.

This projection was insufficient to transfer longitudinal shear after breakdown of bond in the fatigue tests, and it was concluded that these sheets were unsuitable for the loading specified. These results are given elsewhere [2].

Design of specimens. It was decided to test simply supported specimens representing a strip of bridge deck spanning transversely between longitudinal girders, and to use symmetrical two-point loading (Fig. 1c).

In designing the specimens of Groups 1 and 2, it was found that the flexural strength of the steel section was determined by the longest span and loading condition [3] above. Maximum longitudinal shear in the composite section is that due to a wheel load. It was assumed that the critical parameter of interaction was the bearing stress, f_g, on the projected area of the dimples or bar connectors. The allowable bearing stress due to a 90-kN wheel load was taken as $0.6u$, where u is the design cube strength (i.e. two-thirds of the specified 'works' cube strength). This led to the dimple size and spacing shown in Fig. 1 and to the use of three bar connectors 19 mm high in each shear span of specimen MS1.

Table 1. Properties of specimens

Specimen No.	Cube strength	Modular ratio, m	Calc. P for HB loading	Calculated P_u	Observed P_{ut}
	N/mm²		kN	kN	kN
DS1	26.1	8.6	20.5	51.8	63.0
2	34.0	8.9	30.0	53.5	65.0
3	26.6	9.0	20.5	51.8	58.5
4	23.6	9.8	20.5	50.8	32.0*
MS1	25.4	9.0	33.0	51.4	33.0*
2	20.1	11.7	27.0	49.2	30.0*
3	25.1	10.1	22.0	51.4	30.0*
4	24.5	9.1	18.0	51.2	33.0*

* Fatigue failure.

Information on the concrete used and on the strength of the specimens is given in Table 1. In Group 1, the design longitudinal shear per unit length of specimen was determined by the chosen dimple size and spacing and the assumed bearing stress, since bond was neglected. Thus the design vertical shear was known for each shear span, the length of which was then chosen from consideration of the flexural strength of the specimen. In Table 1, the column 'Calc. P for HB loading' gives the load on the specimen at which the calculated bearing stress on the dimples in each shear span is 0.6 u, and 'Calculated P_u' gives the load for flexural failure, calculated using full-interaction theory.

No attempt was made to reproduce in the steel the stresses due to 'unpropped' construction, for these are small near the supports, where the highest shear stresses occur.

Details of specimens, and test variables. In Group 1, the only planned variables were the overhang past the support points, which was 0.46 m except in DS4 (0.28 m), and the magnitude and number of the load applications, of which details

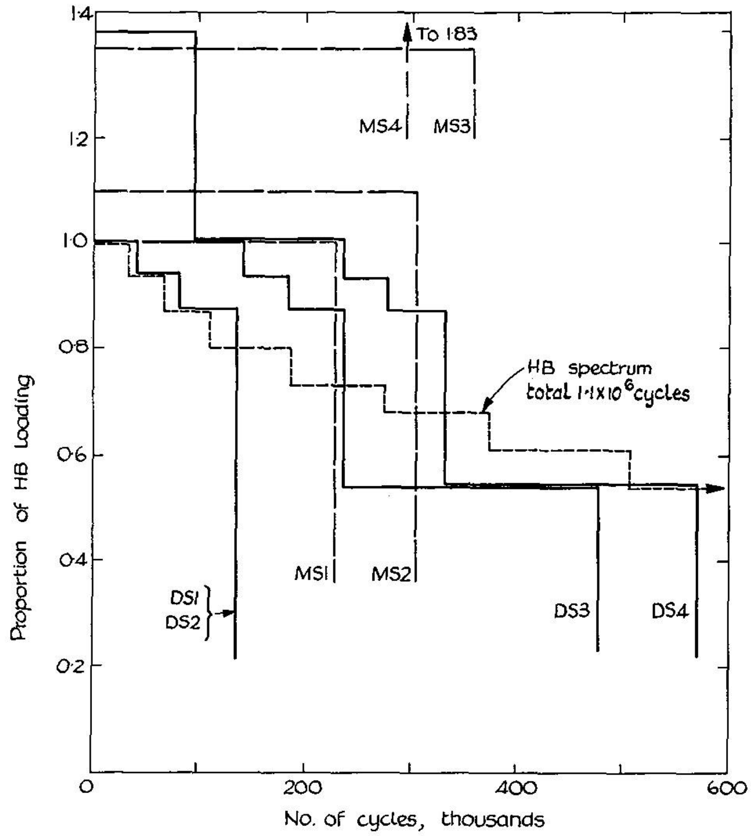


Fig. 4. Repeated loading of specimens.

are given in Table 2. The loadings are compared with the standard HB spectrum in Fig. 4. The values f_g/u in Table 2 are based on measured cube strength, and so do not correspond exactly to 0.6 at the maximum HB wheel load of 90 kN (20,200 lb).

Table 2.

Specimen No.	N_1 $\times 10^3$	f_g/u	N_2 $\times 10^3$	f_g/u	N_3 $\times 10^3$	f_g/u	N_4 $\times 10^3$	f_g/u
DS 1	—	—	55	0.52	41	0.55	41	0.59
2	—	—	55	0.46	41	0.50	41	0.53
3	240	0.32	55	0.51	41	0.55	141	0.58
4	240	0.36	55	0.57	41	0.62	141*	0.66

* Followed by 94,700 cycles at $f_g/u = 0.83$.

In DS1 to 3, the fatigue testing was followed by static tests to failure. In DS4, repeated loading at a mean bearing stress of 0.83 u was continued until failure.

The specimens of Group 2 differed from DS1 to 4 only in the method of shear connection and in the end overhang (0.28 m). The main variable was the intensity of loading on the bar connectors. In designing specimen MS1, it was assumed that the connector would be effective for 130% of the welded length, that f_g should not exceed 0.6 u and that $u = 24.2$ N/mm^2. The weld size was governed by the plate thickness; the design of the welds is discussed later.

These assumptions led to the provision of three connectors at 216 mm pitch in each shear span of MS1 and one in each end overhang. The use of fewer and smaller connectors was studied in the other three tests. In MS2 to 4, the bar in each end overhang was omitted; in MS3 and 4 the height of the bars was reduced from 19 mm to 13 mm, and in MS4, only two bars at 254 mm pitch were provided in each shear span. An additional connector was provided in each specimen at midspan, to prevent uplift.

Table 3. Properties and Results. Group 2.

Specimen No.	Load range	f_g/u	σ_{bd}	$\bar{\sigma}$	N_u	N_c	N_f	$\frac{\log N_c}{\log N_u}$
	kN		N/mm^2	N/mm^2	10^3	10^3	10^3	
MS 1	3-33	0.60	56.1	157	102	230	230	1.17
MS 2	3-30	0.67	43.4	138	175	300	306	1.10
MS 3	3-30	0.82	43.4	115	426	303	360	0.95
MS 4	3-33	1.10	25.5	151	120	197	296	1.10

All specimens in this Group were subjected to fluctuating loads over a single load range (Table 3) until failure. The maximum load on MS1 was that corresponding to 1.0 times HB loading (Table 1). That for MS2 gave a mean bearing stress of 0.67 u, and so corresponded to 1.1 HB. Much higher bearing stresses were used in the last two tests, where the maximum loads were chosen such that the flexural stresses in steel and concrete did not exceed 0.85f_y and 0.65 u, respectively. The columns headed f_g/u and σ_{bd} in Table 3 give the calculated mean bearing stress f_g and the maximum longitudinal flexural stress in the decking at a weld location, σ_{bd}, at these maximum loads.

Casting procedure. During casting, the metal decking was supported at its ends and third points and was coated with mould oil to inhibit bond. About 14 concrete control specimens of various types were cast and cured with each test slab.

Test Procedure

In the fatigue tests, the minimum load was about 10% of the maximum, so the stress ratio was + 0.1. The rate of loading was 200 to 250 cycles/min. Before and during each test, the behaviour of the specimen was monitored by stopping the pulsator and taking sets of readings during a static "run" over the maximum load range.

Static tests to failure were conducted in the same apparatus, and were completed in from 1 to 4 hours.

Curvature, deflection, and longitudinal strains were measured at midspan in all tests. End slip was also measured in each test, and in some tests slip distribution along the shear span was recorded.

Static tests to failure were carried out on five push-out specimens in which the two types of shear connection were reproduced.

Test Results

Typical results are now given; full details are available elsewhere [2]. Data from the static test runs are labelled with the number (N) of cycles of repeated loading completed at that stage. The maximum load per jack applied to each specimen, P_{ut}, is given in Table 1.

In auxiliary tests on materials, the yield strengths in tension of the decking were found to be 257 N/mm^2 for Groups 1 and 2 and 277 N/mm^2 for Group 3; that of the reinforcement was 381 N/mm^2. Table 1 gives results of compressive strength and stiffness tests on the concrete at the mean age of testing of the parent test specimen (29 to 31 days).

Group 1. In Fig. 5, the ratio of the applied load (P) to the calculated load for flexural failure (P_u, Table 1) is plotted against the midspan deflexion, δ, for each of the static-load tests on specimen DS3. The change from a cracked to an uncracked section occurred during the first test ($N = 0$). The crack spacing in the midspan region was about 0.13 m. The curve for the final static test to failure has been zero-corrected. The bands on Fig. 6 show the ranges, for two levels of load, within which midspan strain readings during static tests fell for the whole duration of the fatigue test on this specimen. Typical slip distributions are given in Fig. 7. Results for specimens DS1, 2 and 4 are similar.

The testing of DS4 was designed to study its fatigue strength. The first 377,000 cycles of load (Table 2) simulated the upper part of the HB loading spectrum (Fig. 4). Another 100,000 cycles of loading at $f_g/u = 0.66$ were followed by 94,700 cycles at $f_g/u = 0.83$, at which time fatigue failure occurred in the butt weld in the decking.

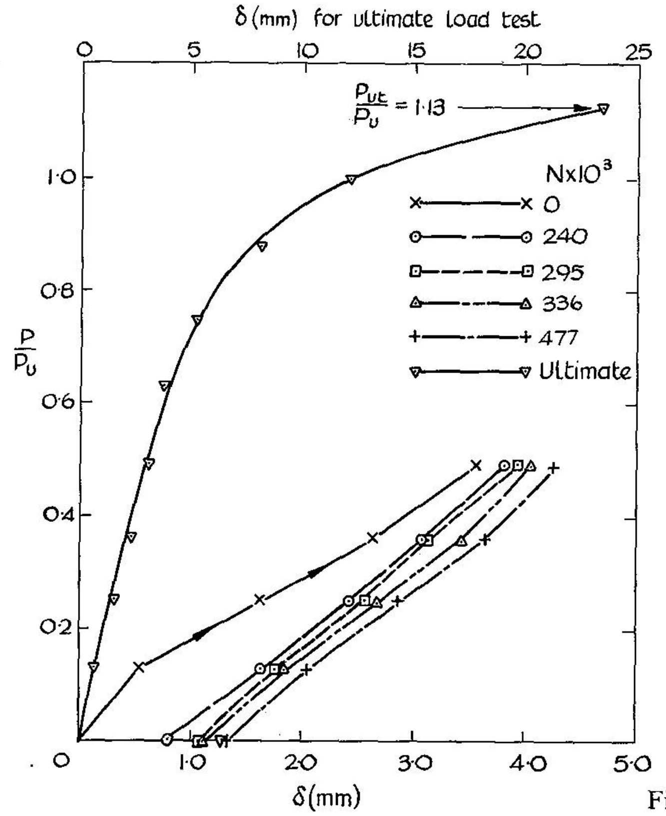
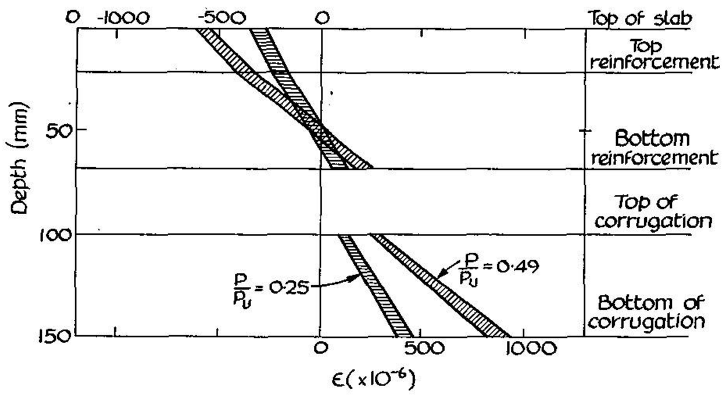
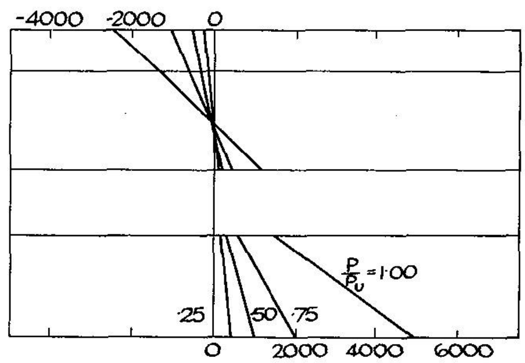


Fig. 5. P-δ curves for DS3.



(a) Strain distributions during fatigue tests



(b) Strain distributions during static test to failure

Fig. 6. Longitudinal strain distribution at midspan during fatigue tests on DS3.

In the ultimate-load tests, DS1 to 3 failed in flexure like reinforced concrete beams (Fig. 8). Slip increased during the tests (e.g. Fig. 7), but no real distress of the shear connection was observed in any of these specimens.

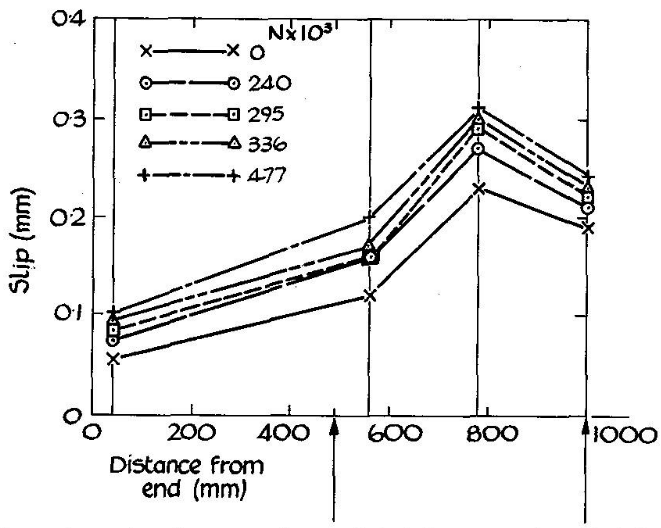


Fig. 7. Slip distributions at west end, specimen DS3.

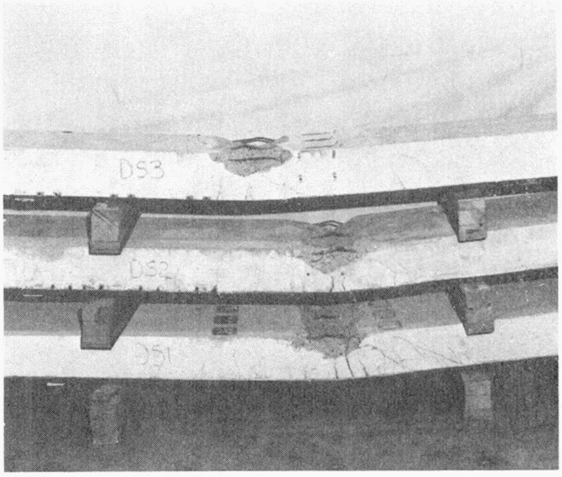


Fig. 8. Specimens DS1 to DS3 after test.

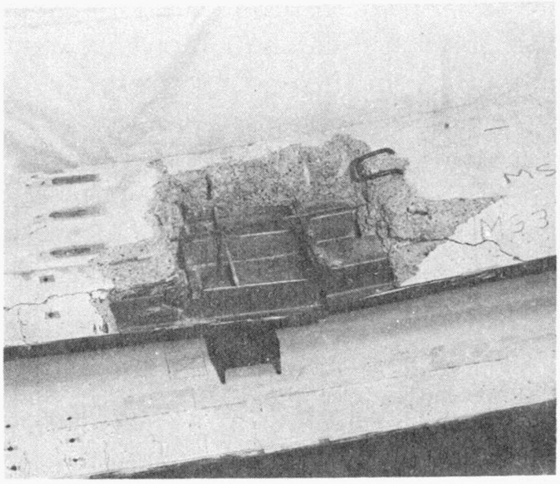


Fig. 9. Concrete removed at failure location in MS3.

Group 2. Load-deflexion curves and strain and slip distributions found in the static tests were similar to those of Group 1, and showed that repeated loading caused little deterioration of stiffness. All four specimens failed by fracture of the decking at a connector location (Fig. 9). Table 3 gives the number of cycles at failure, N_f, and at which cracks were first noticed, N_c. In MS2, 3, and 4, these occurred well in advance of failure, located at the corners of the tops of the corrugations in the heat-affected zone of the connector welds. In all three tests, they were observed at corresponding positions in both shear spans; but those in the spans that failed then increased in length more rapidly than the others.

Push-out tests. The load-slip curves for the specimens of Group 1 and Group 2 [2] were similar in shape to those for stud connectors. Failure occurred by crushing of the concrete against the dimples and distortion of the corrugations (Group 1) and by tearing of the metal decking (Group 2).

Discussion of Results

Group 1. Most of the irrecoverable deflexion of these specimens occurred during the first load cycle. The small increases in permanent set and the consistent distribution of longitudinal strains and slip show that repeated loading caused negligible deterioration at the dimple-concrete interfaces. The sets of P-δ curves show that additional cycles of load at low levels have very little effect, and suggest that the behaviour would have benn much the same had the entire loading spectrum been applied.

It had been assumed in planning the tests that full HB loading should correspond to a bearing stress of 0.6 u on the dimples, bond being neglected. In considering this as a design value, it is helpful to compare (Fig. 4) the loading imposed in the tests with the standard HB spectrum. The spectrum down to 0.61 HB is effectively covered by the tests. Lower loads should cause bearing stresses not exceeding about 0.3 u, which should not cause fatigue damage to the concrete.

There is evidence from test DS4 that a bearing stress of 0.7 u or 0.8 u could probably be used; but allowance must be made for the contribution to shear strength from the end overhangs (neglected in the calculations). For example, if the effective shear span was the actual span plus half the overhang length, the mean values of f_g/u in DS1 to 3 were 70% of those reported, and 80% in DS4. There should be no problem where forms are continuous over supports, but the strength of a shear span without overhang may be less than found in these tests. It is concluded that 0.6 u should be used in design for HB wheel loading until more evidence becomes available.

The static tests to failure showed that previous fatigue testing at bearing stresses up to 0.6 u causes little, if any, reduction in static strength. Both DS1 and DS2 had flexural strengths 1.21 times the value given by simple plastic theory, and the ratio for DS3 was 1.13.

Group 2. Comparisons of the results from Groups 1 and 2 show that their behaviour was similar, and that the slabs with bar connectors were slightly stiffer than those with dimples. The similarity of the distributions of slip was remarkable [2], bearing in mind that shear connection in the Group 2 specimens was provided only at two or three points in each shear span and (in MS2 to 4) not in end overhangs.

Examination of the connectors in MS3 after the test (Fig. 9) confirmed other evidence that little distress or distortion occurred at the shear connectors during fatigue testing.

It is evident from Table 3, Fig. 4, and the mode of failure that the nominal bearing stress f_g/u was not a critical parameter in these tests. In MS1 to MS3 increased bearing stress is accompanied by increased fatigue life. In MS4 no increase in the flexibility of the specimen was observed until after cracking of the decking, showing that the greatly increased bearing stress did not reduce the effectiveness of the shear connection.

The range of mean bearing stress explored was from 0.6 u to 1.1 u. CP 117: Part 2 [4] limits the load per connector for HB loading to 0.4 P_u, which corresponds to a mean bearing stress of 0.93 u for the bar connectors listed in Table 2 of CP 117. It is concluded that this same level of bearing stress could be used for bar connectors attached to corrugated sheeting.

It does not follow that the weld sizes should be as specified in CP 117, for in the present application each weld supports a greater length of bar, and attaches it to a plate that may be thinner than the flange of a typical girder. It can be shown [2] that the fatigue life was determined mainly by the stresses in the weld, rather than by the longitudinal stress in the decking, given as σ_{bd} in Table 3.

The Table also gives the mean stress on the weld throat, $\bar{\sigma}$, and the fatigue life N_u for a Class F detail having this maximum stress, taken from B.S. 153 [3]. For the present purpose, failure is assumed to occur in the test when the first crack is observed, given as N_c cycles in the Table. The ratios ($\log N_c/\log N_u$) give the 'safety factor' of the weld in relation to design to B.S. 153.

When account is taken of the assumptions made above, and the inevitable scatter of the results of fatigue tests, it can be seen that there is good agreement between the test data and the predictions of B.S. 153.

It is of interest that, as assumed in this analysis, all fractures occurred on the side of a connector closer to midspan, and at the connector where the measured slip was greater, not at the connector nearest to the load point.

The ratio of weld leg length to plate thickness was 1.9, far in excess of the limit of 0.5 given in CP 117: Part 2. These results suggest that a higher limit, perhaps 1.5, could be allowed for metal decking. It is concluded that with this limitation, the method of B.S. 153 may be used for designing fillet welds to bar connectors in corrugated composite plates.

Pushout tests. In the three pushout tests on Group 1 specimens the lowest mean bearing stress at failure was 1.65 u. This gives a generous margin above the proposed design stress of 0.6 u for repeated loading. These results and the static tests of Group 1 show that the design ultimate bearing stress for static loading could be increased to about 1.0 u.

The tests on bar connectors showed that the strength was governed by the sheeting, not by bearing stress on the bars, which exceeded 1.5 u in both tests.

Conclusions and Recommendations for Design

The plate specimens tested were in essence two-thirds scale versions of a prototype designed to the requirements of the Department of the Environment (Ministry of Transport) for use as permanent formwork in bridge decks subjected to HA and HB loading. The purpose of the work was to develop design rules for this formwork, using it if possible as the bottom reinforcement of the deck slab.

It was assumed that the design criterion was that the shear connection should be capable of resisting the local effects of the fatigue spectrum of HB loading without significant reduction in the flexural strength of the bridge deck under static loading. Two of the three types of decking studied are believed to be satisfactory for this purpose. Tentative design rules for these (Groups 1 and 2) are now given.

1. The mean bearing stress on the projected area of dimple shear connectors as used in specimens DS1 to 4 due to 1.0 times HB wheel loading should not exceed 0.6 u, where u is the design cube strength of the concrete. The stress should be calculated by the elastic theory, using a fully composite section and neglecting the tensile strength of concrete.
2. The corresponding stress for bar connectors as used in specimens MS1 to 4 is 0.9 u. The effective length of each bar is the lesser of the actual length and 1.3 times the length welded to the corrugated decking.
3. The fillet welds by which bar connectors are attached to decking may be designed for fatigue in accordance with B.S. 153: Part 3B, provided that the leg length of the weld does not exceed 1.5 times the thickness of the decking, and that the coexisting longitudinal tensile stress in the decking at the weld does not exceed 50 N/mm^2 (3.2 ton/in^2).
4. The limiting value of static bearing stress on dimples at the Collapse Limit State was not determined, but is not less than 1.0 u. When this condition is satisfied, the static flexural strength of the composite section may be taken as that given by the well-established rectangular-stress-block theory.

5. The strengthening effect of overhang of the composite plate beyong a simple support was not studied. Overhangs not less than 0.28 m were used in the tests.
6. Pushout tests for corrugated decking were devised. They are thought to give a reliable indication of the static strength of the connectors used.

Information about possible variations in the design of the sheeting used for Groups 1 and 2 may be useful. The ratio of dimple projection to sheet thickness was 2.5. That in the proprietary decking, 0.55, was found to be too small for complete interaction after repeated loading [2]. Obviously other factors, such as the shape of the dimple, are relevant: but it is thought that a successful design will have a ratio of at least 2.0. If a scaled-up version of the Group 2 sheets were made, it might be better to use relatively smaller bars and welds, for fatigue behaviour should be improved, and failure of connectors before shuttering would provide warning of deterioration.

The effects of corrosion and fire have not been studied.

The choice between dimpled and undimpled-with-bars formwork will depend mainly on relative cost. Both types are believed to be suitable for use in bridge decks.

Consequences of this work

The cost of corrugated steel decking depends on the demand for it, to a far greater extent than for many other materials, because of the large capital investment needed for its manufacture. It is evident that its use in bridge decks can reduce construction costs. The safety and economy of such structures can only be ensured through an understanding of the fatigue behaviour of possible types of shear connection between the steel decking and the concrete slab. The research reported above provides guidelines for the development of commercially viable profiles and methods of shear connection for use in structures subjected to repeated loading.

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Summary

Results are given of static and fatigue tests on eight composite plates, each consisting of a sheet of corrugated steel decking attached by dimple or bar shear-connectors to an in situ concrete slab. The local effects of repeated wheel loads up to 90 kN (20,200 lb) were simulated in the tests. Rules for designing two types of shear connection for composite plates in bridge decks are deduced from the results.

Résumé

On donne des résultats d'essais statiques et de fatigue opérés sur huit plaques composites dont chacune est composée d'une tôle d'acier ondulé attachée par encastrement ou par connecteurs de cisaillement sur une plaque en béton armé installée sur place. Les effets locaux dus aux charges de roue répétées jusqu'à 90 kN (20000 lb.) ont été simulés aux essais. Par les résultats obtenus on a dérivé des règles en vue de projeter deux types de connexions de cisaillement pour plaques composites de plates-formes de tabliers.

Zusammenfassung

Es werden die Ergebnisse von statischen und Ermüdungsversuchen an acht Verbundplatten mitgeteilt, wobei jede aus gewelltem Stahlblech besteht, die mittels einer Verbindung oder eines Schubverbinders an einer an Ort und Stelle vorhandenen Betondecke befestigt ist. Die örtlichen Einflüsse wiederholter Radlasten bis zu 90 kN (20000 lb) wurden bei den Versuchen simuliert. Aus den gewonnenen Resultaten wurden Regeln zum Entwurf zweier Typen von Schubverbindern für Verbundplatten an Brückenfahrbahnen abgeleitet.

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