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Autor: Bhasin, P.C. / Prakash Rao, D.S. / Tamhankar, M.G.

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Fatigue Behaviour of a Composite, Steel-Concrete Girder

Comportement à la fatigue d'une poutre mixte acier-béton

Ermüdungsverhalten eines Stahl-Beton-Verbundträgers

P.C. BHASIN
Ministry of Shipping and Transport
New Delhi, India

D.S. PRAKASH RAO
Structural Engineering
Research Centre
Roorkee, India

M.G. TAMHANKAR
Structural Engineering
Research Centre
Roorkee, India

S.P. SHARMA
Structural Engineering
Research Centre
Roorkee, India

SUMMARY
Tests were carried out to determine the behaviour of a steel-concrete composite girder, with basket type shear connectors, under pulsating loads. Shear connector capacity was calculated according to various codes of practice and compared with the test results. The girder was only loaded beyond the computed elastic limit after one million cycles had been applied below yield. Full composite action was exhibited up to failure.

RESUME
Des essais ont été effectués pour déterminer le comportement d'une poutre mixte, acier-béton, avec des connecteurs de type „basket", sous charges pulsatives. La résistance ultime des connecteurs a été calculée selon différentes normes et comparée aux résultats d'essais. La poutre a été sollicitée au delà de la limite élastique calculée après avoir subi un million de cycles de charges en dessous de la plastification. L'effet mixte complet a été observé jusqu'à la rupture.

ZUSAMMENFASSUNG
Der Beitrag behandelt dynamische Versuche an einem Verbundträger mit halbkreisförmigen Schubdübeln. Der Schubwiderstand der Dübel wurde unter Anwendung verschiedener Normen berechnet und mit den Versuchsresultaten verglichen. Nach einer Ermüdungsbelastung von einer Million Lastwechsel im elastischen Bereich wurde der Versuchsträger zyklischen Belastungen oberhalb des rechnerischen elastischen Bereichs unterworfen. Es wurde volles Verbundverhalten bis zum Bruch beobachtet.

1. INTRODUCTION

Steel concrete composite construction is extensively used in buildings as well as bridge superstructures. Of late, the composite construction is finding application in long span bridges too. The second Hooghly bridge in Calcutta, India when completed is expected to be the longest composite structure with its 457 m cable stayed main span. The interface connection between concrete and steel forms an important feature of the design in composite structures. Basically the connection has to ensure composite action through the longitudinal shear at the interface at all the stages of loading. This feature assumes added importance in bridge structures subject to varying loads in their life time. Several tests are reported on composite beams. However, most of them pertain to the stud connectors and under static loads. Tests are carried out on basket type connector shown in Fig.1 to determine its capacity in longitudinal shear. The test results are compared with the values predicted by various codes of practice.

These results are verified to check the integral action between steel beam and concrete slab in a composite girder. The girder was subjected to one million loading cycles, the amplitude varying upto the working load, then subjected to cyclic loading beyond elastic limit and loaded to failure.

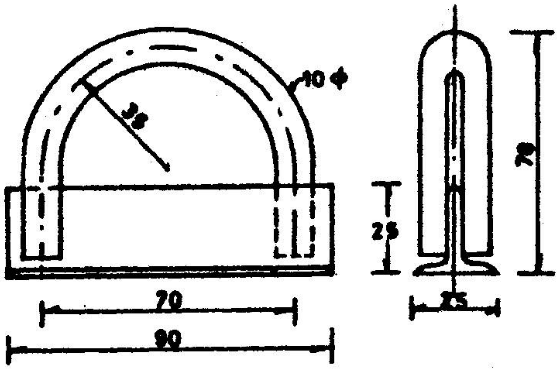

Fig.1 Basket type shear connector

2. STRENGTH OF SHEAR CONNECTORS

2.1 Specifications in various codes of practice

Emperical or semi-emperical formulae are available in several codes of practice to assess the capacity of the shear connectors subject to longitudinal shear. However, the formulae are not usually of general nature and have rather limited applications. These formulae have to be necessarily conservative to prevent the failure in longitudinal shear before the advent of the flexural failure of the structure. Specifications for standard tests are also not available in many codes of practice.

BS 5400, Part 5 [1] and CP 117 Part 1 [2] specify tests on standard specimen. The lowest value of the failure load of three standard specimen is to be taken for design purposes. The specified live load factor against connector failure is to be at least 2.5 as per CP 117 and the average force on shear connectors should not exceed 80% of the ultimate capacity as per CP 117 as well as BS 5400. The Canadian Standard [3] specifies a factor of safety as a function of the live load and dead load shears and moments. The AASHO specifications [4] incorporate a factor in the strength formula to account for the number of loading cycles. However, these are available only for channels and studs. The German code of practice [5] insists on making computations with the ultimate load factors even for constructional loads. The shear stress on the possible failure plane around the shear connector should not exceed the values specified in the code. The capacity of the shear connector is estimated by considering the enlarged area of the connector upto the next connector at an angle $\tan^{-1} 1/5$. These values are to be reduced by a factor 2/3 for structures subjected to dynamic loads like bridges. The concrete cover requirements according to the German standard are rather liberal. Whereas the British code requires a minimum cover of 25 mm at the top of the connector, the German standard allows the studs to be flush with the concrete surface if corrosion is not a problem. The Indian Roads Congress code of practice [6] provides a formula to estimate the strength of the shear connector based on the strength of concrete and as a function of the cube root of the ratio of area of failure plane and the enclosed area of the connector.

2.2 Tests on the shear connector

Tests on standard specimen based on BS 5400 were carried out to assess the ultimate capacity of the basket type shear connectors. One shear connector was welded to each side of the central steel I-section. The slip of the central steel girder relative to the concrete blocks during the push out tests on the specimen S1, S2 and S3 is shown in Fig.2. Mould oil was applied on the steel girder surface to prevent the bond at the interface. All the three specimen exhibited more or less the same load displacement characteristics in the initial stages. The specimen S2 and S3 were subjected to about 30,000 cycles of load varying from 20 to 80 kN before conducting the test to failure. The fourth specimen S4 did not show any change in load displacement characteristics after 50,000 cycles of load varying from 38 to 80 kN. The typical failure of the specimen is shown in Fig.3. The failure in S2 and S3 was through crushing of concrete whereas the shear connector gave way in S1. The changes in concrete block in the vicinity of the shear connectors was sought to be monitored by measuring ultrasonic pulse velocity through the blocks and the steel section. However, this was not successful as the ultrasonic tester was showing erratic readings and hence not pursued further.

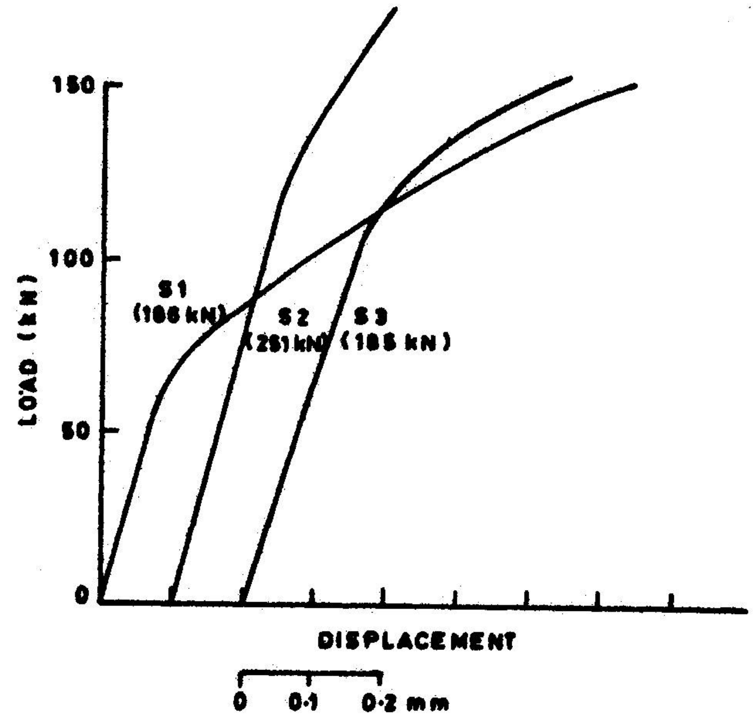

Fig.2 Load displacement curves of the standard specimen S1,S2 and S3. The figures in the brackets indicate the failure load

The capacity of the shear connectors computed according to various codes of practice is given in Table 1 for comparison. It can be seen that there is wide disparity in the values predicted by various codes of practice. The IRC code of practice appears to be the most conservative, whereas the values computed as per the German code appear to be on higher side than the test results.

Table 1 Strength of the shear connector based on various codes of practice

S.No.	Code of practice	Allowable load (kN) during working	at ultimate
1	BS 5400 [1]	37.0	74.0
2	CSA Standard [3]	41.8	-
3	DIN 1078 [5]	-	123.9*
4	IRC Code [6]	25.6	63.9
5	Laboratory tests (as per BS 5400)	-	92.5

* 82.6 kN for structures subject to dynamic loads, such as bridges

3. DESIGN OF THE TEST BEAM

The test beam was designed based on working stress method for a live load of 150 kN over two points as shown in Fig.4. The average strength of three concrete cubes of 150 mm cast along with the beam was 17 N/mm^2. The yield strength of the steel girder was assumed to be 260 N/mm^2 and the allowable strength as per IRC code of practice was 140 N/mm^2. The top slab had a nominal reinforcement of 6 mm bars at 160 mm in the transverse direction and 8 bars of 6 mm in the longitudinal direction, Fig.4.

The steel girder was greased well so that the bond between steel and concrete was effectively prevented. The integral action was possible only through the shear connectors provided at 160 mm spacing throughout the length of the girder. The ratio of Young's modulii of elasticity for steel and concrete was assumed to be 9 for calculation purposes. It may be mentioned, however, that variation in the value of the

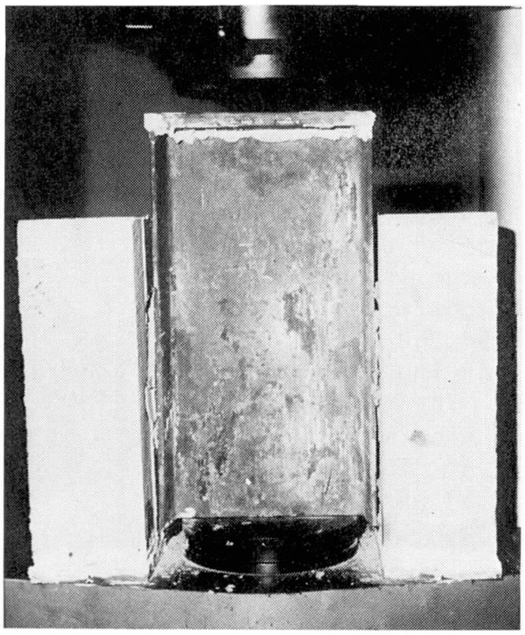

Fig.3 Typical failure at the interface of the standard specimen

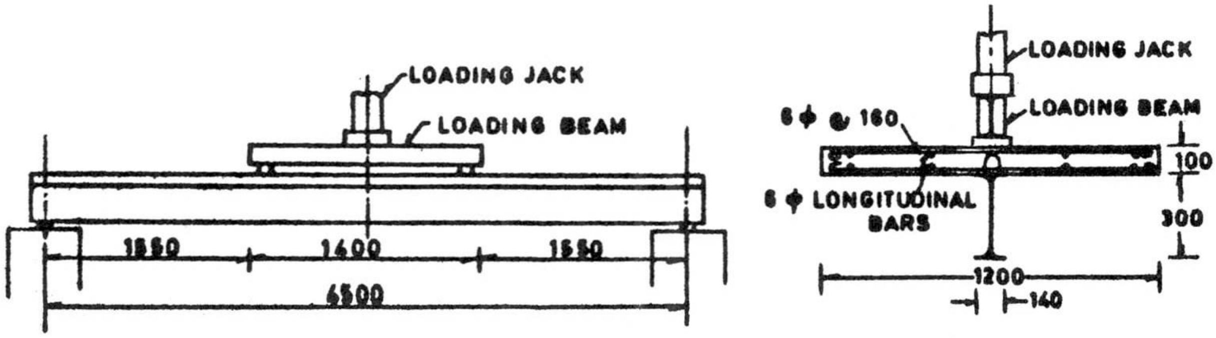

Fig.4 Longitudinal-and cross-sections of the composite girder

Young's modulus of concrete affects the computed stresses and deflections to a small degree.

4. TESTS ON THE COMPOSITE GIRDER

The composite girder was tested under static load at the start of the test as well as periodically during the fatigue loading. The deflections of the girder were measured by means of dial gages. These values compared well with the predicted deflection profile. The girder was then subjected to one million cycles of load varying from 40 to 150 kN at a frequency of 250 cycles per minute. This range of loading produces an average computed force varying from 19 kN to 36 kN on the shear connectors. The upper limit of the average force on the shear connectors is more than the value predicted by the IRC code but close to the value as per BS 5400. This range of loading was adopted to simulate the effects of force on shear connectors in a bridge structure which will be subjected to a constant force under dead load and a varying force due to live loads. The test set up is shown in Fig.5.

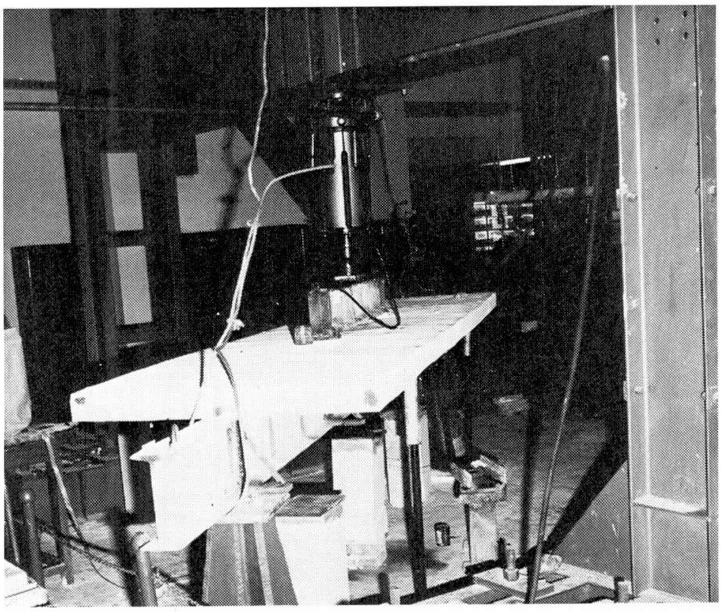

Fig.5 Test set up showing the girder and the pulsator

The deflections of the girder at mid span and one metre from the supports are shown in Fig.6 at the start of the test and after application of the cyclic load. The cyclic load does not seem to alter the behaviour of the beam appreciably. The concrete slab did not show any signs of distress at the interface.

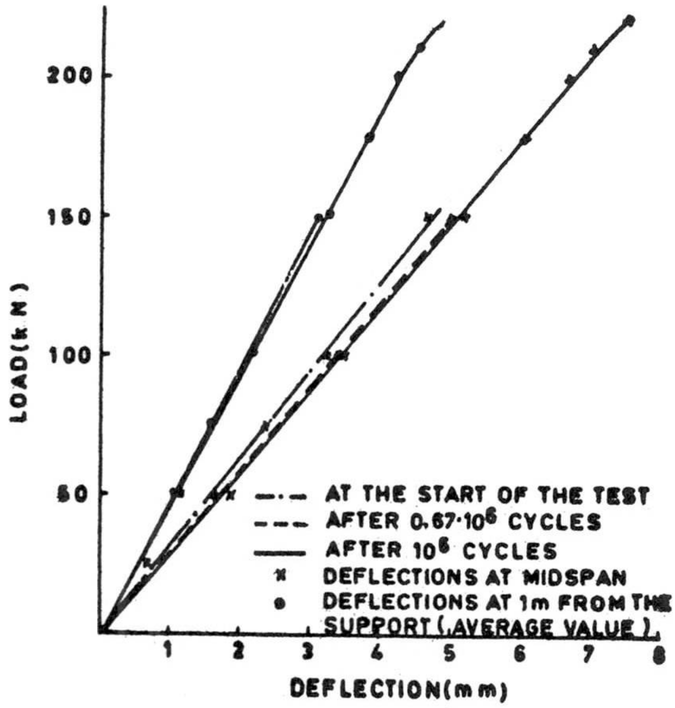

Fig.6 Load deflection curves of the girder

The girder was then repeatedly loaded beyond the computed elastic limit as shown in Fig.7. The first load cycle of 250 kN produced a residual mid span deflection of about 0.2 mm compared to the elastic deflection of 5.1 mm under 150 kN at the end of one million cycles

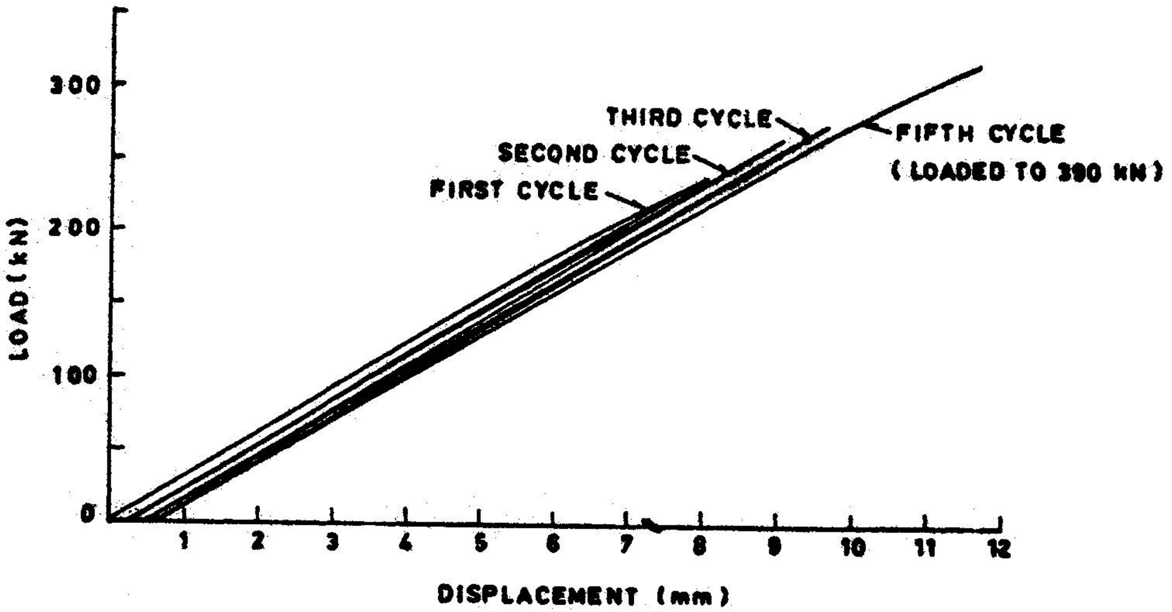

Fig.7 Mid span load deflection curves of the girder during cyclic loading beyond elastic limit

of loading. The residual mid span deflection was almost zero after another three cycles upto 270 kN.

The horizontal displacement of the concrete slab relative to the steel beam was monitored during the cyclic loading by means of dial gages set up at the ends of the girder, Fig.8. The relative displacements were very small in the beginning but grew at a large rate beyond 200 kN load. However, the recovery was almost full when the load was removed. During subsequent loadings the behaviour was similar, indicating that the average behaviour of the shear connectors was still elastic.

During the ultimate load test the left end of the slab started showing larger relative displacement than the other end. The right end of the slab showed no increase in the displacement between 200 kN and about 300 kN when the displacement started increasing at a larger rate than initial. The left end showed a steady increase upto about 310 kN when the relative displacement showed an increasing rate. The failure of the girder took place at 435 kN with the concrete in the vicinity of the shear connectors at the left end giving way. The concrete slab split in the longitudinal direction over the interface and the girder could not take any more load, Fig.9.

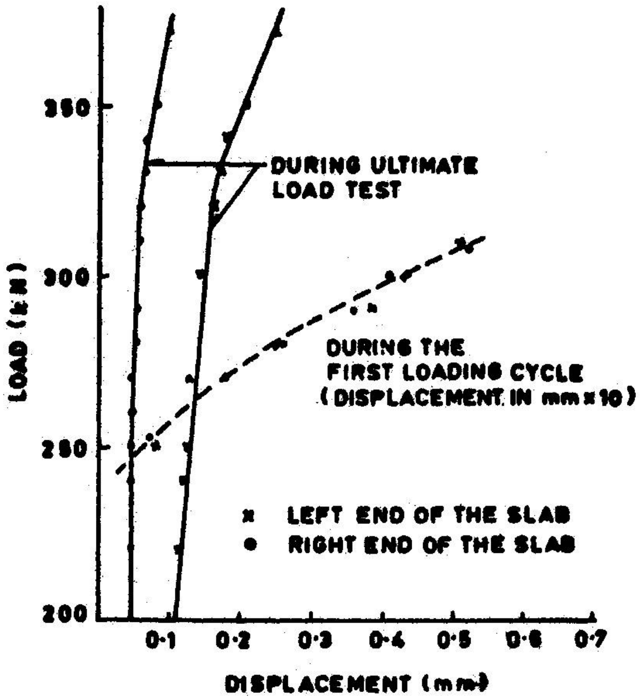

Fig.8 Horizontal displacement of the concrete slab relative to beam

5. DISCUSSION

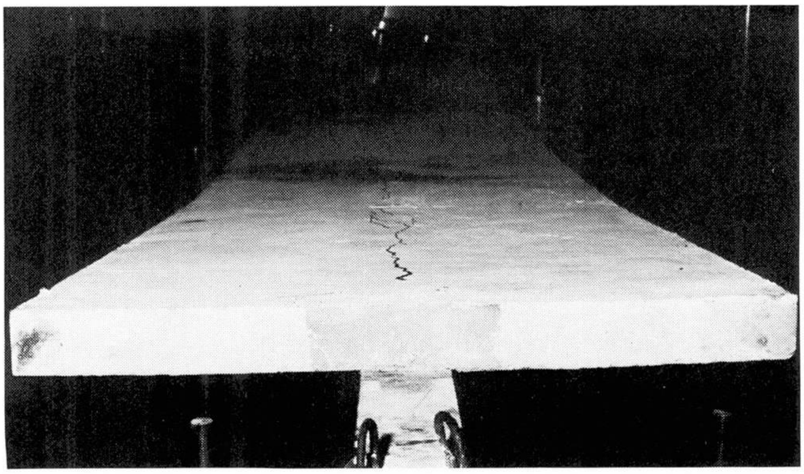

Fig.9 Longitudinal cracks on the concrete slab surface along the line of shear connectors at the ultimate load

The chief objective of the tests was to study the behaviour of basket type shear connectors under fatigue load by simulating the conditions in a composite bridge deck. The capacities of the shear connector at working and ultimate loads as per various codes of practice differ considerably from each other. The codes of practice usually refer to limited types of connectors and actual tests are required for other types. The capacity of the shear connector was adopted from the standard tests in this case. The composite girder exhibited usual behaviour even after one million cycles of loading and subsequent cyclic loading beyond computed elastic limit. This indicates that the shear connectors behaved well though the integral action was solely through shear connectors as the bond between steel beam and concrete slab had been destroyed by application of grease. The final failure was brought about by the splitting of the concrete slab along the line of shear connectors. The average load on the shear connectors at the failure stage works out to be about 170 kN, almost double that of the standard test value. It is likely that the capacity of the shear connector gets influenced by the presence of other connectors apart from the interaction between longitudinal shear and flexural compression. Influence of the connector spacing is taken into account by the German code of practice [5]. However, the average load per connector is still higher than the value computed as per the German code, Table 1. Further investigations into these aspects are necessary.

6. ACKNOWLEDGEMENT

The tests reported here were carried out at the Structural Engineering Research Centre, Roorkee, India.

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