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A New Test for Stud Connectors in Ribbed Slabs

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Summary

A new push test was developed to investigate the behaviour of headed studs in ribbed slabs. The main reason was that the standard push tests are not suitable for the validation of numerical models. A series of tests with the new test set-up was carried out and evaluated. The test set-up and some test results are briefly described.

1. Introduction

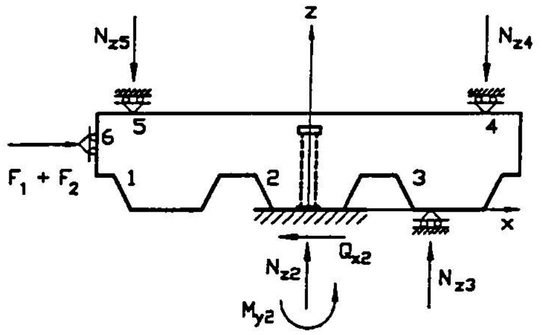
The behaviour of a composite steel-concrete beam is essentially influenced by the properties of the longitudinal shear connection. In present practice the resistance of shear connectors placed in the ribs of composite slabs are related to the resistance of a connector in a solid slab by means of one reduction factor. It is now generally agreed that this method is not satisfactory.

A research project was started to develop a numerical model for the simulation of the behaviour of a headed stud connector in the rib of a composite slab. It appeared that in the standard Eurocode 4 push tests as normally used, the boundary conditions were not unique. Therefore, a new push test suitable for the validation of the numerical model, was developed.

2. Tests: specimens, measurements and results

Contrary to the standard test procedure just one rib was tested (see Fig. 1). All loads and boundary conditions were carefully monitored. This included all support reactions (see Fig. 1), the internal forces in the stud (N_z, M_x and M_y) and the difference between the displacements of the concrete rib and the foot of the stud. This is essential for the validation of numerical models.

In total 22 tests were carried out. The parameters which were varied are: the steel sheeting (with and without sheet, with and without embossments, 'thin' and 'thick' sheet), the geometry of the rib, the place of the stud within the rib. the welding technique (through deck welded stud and stud placed in a precut hole in the sheet), the concrete strength and the hogging reinforcement.



The test results included: the shear force-slip relation, the load at first cracking of concrete, the place of the first crack (front side or rear side of the stud) and the failure mode. Material properties of the stud, the sheet and the concrete were determined.

Fig. 1 New test set-up

Most results confirmed the general behaviour known from standard push tests as described in Eurocode 4. Also, new ideas were gained which resulted in an improved insigth in the behaviour. It appeared that the steel sheeting had a significant influence on the behaviour. As well a larger thickness of the sheet as the presence of embossments increased the failure load. Both the failure modes 'tension shear failure of the stud' and 'concrete cone' were observed.

Analyzing all results some postulations about the behaviour could be made. One of them is that, for through deck welded studs, the behaviour for both failure modes is initially the same. At the moment that a crack originated at the rear side of the stud the behaviour became different, which finally resulted in a completely different failure mode.

Normally it is accepted that through deck welded studs in comparison with studs in precut holes, have higher failure loads. The new push test showed the contrary. This is probably caused by the fact that the concrete in front of the stud is completely restrained. For this reason it was found that the behaviour was quite different, although the failure mode is finally the same.

Although perhaps obvious, at small slip the headed stud connection transferred the load by stud bending and by a couple of normal forces: one in the stud and one in the rib at the rear side of the stud. The test results showed that, prior to the occurance of the maximum shear load, the full plastic moment of the stud was exceeded. Besides the plastic deformations of the stud, other non-linear phenomena were observed: buckling of the sheet, cracking of the concrete, crushing of the concrete, punching of the stud through the concrete, sliding of the concrete over the sheet and plastic deformations of the sheet caused by riding over. Most of them occurred at small slip already.

3. Conclusions

The results of the new push test are suitable for the validation of numerical models. The numerical model should be able to take complicated non-linear phenomena as described into account. Once the numerical model is able to predict the behaviour of the new push test, a powerful tool is available to determine design formulae for composite beams.

4. References

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Connection Characteristics for Joints between Hollow Core Slabs and Slim Floor Beams

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Summary

Hollow core slabs become part of a composite flooring when they are supported on beams. When slabs are integrated with slim floor beams, a system of multiple longitudinal shear interfaces will form in which the webs of the hollow core slabs also become a shear interface. The load-slip characteristics of the connection interfaces are described so as to give an impression of their role in finite element modelling, and their typical behaviour as observed in the calculation is explained.

1. Introduction

In slim floor structures, hollow core slabs (HC slabs for short) supported on beams inevitably become a part of a composite system when grouted joints are used. Although it is a conservative assumption to neglect this interaction in beam design, it should be allowed for when designing the slabs, as it has been shown by experimental and theoretical research that the vertical shear resistance of the slabs is considerably reduced as compared with the maximum resistance of the slabs on non-flexible supports. The real structural system includes various longitudinal shear interfaces with highly non-linear characteristics, and the stiffness of the joints will decrease considerably during loading of the system, causing then a reduction in the composite interaction rate.

1.1 Discretization into layered beam elements

In order to discretize a typical system, as in Fig. 1, into beam elements, four layers of elements are required, which should be connected appropriately by coupling elements that model the behaviour of the shear interfaces (i1) to (i4), as described below.

1.2 Description of interfaces

Shear interfaces develop mainly through balancing of the longitudinal normal forces due to the bending moment in the composite cross-section, the forces to be balanced being the compressive force at the top hulls of the HC slabs, which serve as flanges to the beam, and the tensile force at the beam section. The behaviour of the top hulls of the HC slabs is similar to that of the concrete slab in contemporary composite beams, but the method of transferring the compressive force to the beam is more complex. With reference to Fig. 1, the interfaces to be distinguished are: (i1) concrete bonding to the concrete or steel surface, (i2) connection of a reinforced or unreinforced top concrete layer to the beam through a cracked or uncracked vertical interface, (i3) connection between the top and bottom hulls of the slab units, and (i4) connection of the bottom hulls of the HC slabs to the beam. The problems arising due to the composite behaviour are related to the forces transferred through interface (i3), in which the web ribs of the slabs serve as shear connectors between the hulls.

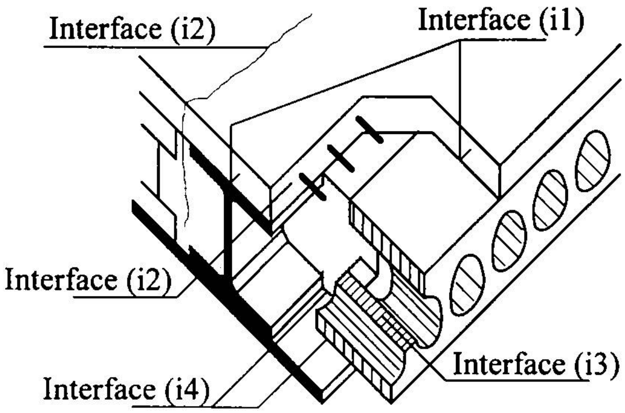

Fig. 1 General view of the various longitudinal shear interfaces activated in a slim floor in which HC slabs are integrated in the system

2. Connection characteristics

It is evident from Fig. 1 that if interface (i2) is inefficient, the majority of the force transfer from the top hull of the slab to the beam body must happen through (i3). This is always the case in structures with no reinforcement at interface (i2). The deformability of the system may be described in terms of the load-slip properties of the shear interfaces, of which (i1) and (i3) are characterized as non-ductile in the sense that the load drops considerably after the peak load is reached, and the slip required for reaching the peak load is quite small, normally much less than 1 mm. Independent of any transverse reinforcement at interface (i2), it should be characterized as ductile, as no sudden unloading will normally occur, and the same is also valid for (i4). The non-ductility of interface (i3) is critically reflected in the ability of the decking to bear loads, as a failure in the webs normally means collapse of the whole slab.

2.1 Methods to enhance behaviour of slabs

The considerable reduction in the vertical shear resistance of HC slabs is attributable to the transverse shear stresses in their webs, and any effective means of reinforcing the slabs must reduce the transverse shear stresses [1]. There are two methods for doing this, re-routing of the longitudinal shear forces and strengthening of the critical interface (i3). The practical importance of these measures is well demonstrated by the parametric studies carried out by finite element calculations, in that reinforcement of the top concrete across the beam normally means an enhancement of some 20 to 30 % and filling of the voids at the slab ends to a length equal to the depth of the voids approximately the same degree of improvement.

3. Reference

[1] Leskelä, M.V. and Pajari, M., "Reduction of the Vertical Shear Resistance in Hollow Core Slabs when Supported on Beams". Concrete 95, Conference Papers, Volume One, CIA and FIP (559-568), Brisbane, Australia 1995