Various tests for defining the behaviour of composite slabs

Various Tests for Defining the Behaviour of Composite Slabs

Arto TENHOVUORI
M.Sc. Civil Eng.
Helsinki Univ. of Tech.
Espoo, Finland

Arto Tenhovuori, born 1960, received his M.Sc. in 1991. From 1987 to 1993 he worked for a bridge consulting company in Oulu on practical bridge design and joined HUT in 1993. His research into compositestructures has concerned the behaviour of composite slabs.

Matti V. LESKELÄ
Ph.D. Civil Eng.
Univ. of Oulu
Oulu, Finland

Matti Leskelä, born 1945, received his Ph.D. in 1986 and has been carrying out research into composite structures from the early 1980's. His latest work has concerned problems of partial interaction and various shear connections in composite structures such as slim floors, composite slabs and concrete filled steel tubes.

Summary

There are several possibilities for experimentally defining the strength of the shear connection in composite slabs, in which a full plastic bending resistance cannot be reached. Harmonisation of the methods has not been done so far, and the paper discusses the characteristics of various tests such as the slab test to Eurocode 4, its modifications and some small scale tests. The applicability of the partial shear connection method to all slabs, independent of the ductility requirements, is also discussed.

1. Introduction

In EC4 (ENV 1994-1-1 [1]), two methods are given for defining the resistance of composite slabs in the case of shear connection failures: the m-k method and the method based on partial shear connection theory. It is allowed to apply the former method to all slabs, independent of the ductility characteristics of the shear connection, while the applicability of the latter is restricted to slabs having only ductile shear connections. In both methods the behaviour assumed for the design is generalized from six flexural tests, which should represent the essential characteristics of the structural system. The question may then be asked, are the tests required capable of this. It has been shown in various studies that the methods of EC4 for composite slabs should be improved, and there are some rules contradictory with real behaviour.

Some profiles do not meet the ductility definition to EC4, but numerical analyses by the finite element method would show that the behaviour of the slabs with these sheetings is nevertheless ductile enough that it would be appropriate to interprete it consevatively enough according to the partial shear connection theory. The bond strength for the theory, $\tau_{u.R}$, is determined from the results of slab tests with two concentrated loads, and is assumed to be constant, but a considerable variation will be observed when shear spans and slab depths are varied. It is justified to assume that the basic property of the shear connection, the longitudinal shear resistance of the connection per unit length, is constant, but it is the interpretation of the test results in $\tau_{u.R}$ that makes the variation. It is also possible to apply small scale tests, such as the Australian slip block test [4] and various pull-out tests, but it is not clear if they offer a better solution or not. The problem in real slabs is that different parts in their shear connection do not reach their ultimate resistance at the same instant which contradicts the idealized theories.

2. Forces at connection interface

Both vertical and longitudinal forces are introduced to the connection interface by vertical loads on a slab, but the longitudinal shear forces are mainly considered in the design, as the resistance to these forces controls the bending resistance of the member. Reasonable models are available for the longitudinal forces, but it is quite complicated to evolve the forces that cause vertical separation effects between the sheeting and the concrete accurately. However, their importance in the bonding behaviour has clearly been shown in tests, since the persistency of the interlocking systems depends on their resistance to vertical separation: in flexural tests for common embossments it was observed that the resistance to longitudinal shear in a re-entrant rib-profile was greatly enhanced when its resistance to vertical separation was improved by adding 'elbows' to the corners of the re-entrant ribs. It should also be noted that in all bond tests which are not based on flexure, the separation effects do not appear on the same level as in the bending tests, and in the shear block test [4] the vertical separation is deliberately prevented.

3. Numerical simulation of real slab behaviour

The method based on layered beam elements (LBE) [3] has been used succesfully for simulating various slab tests. It is possible to control both local and global properties of the shear connection so as to include all the essential parameters which affect the behaviour and results in the analyses. Shear connection is modelled as non-linear spring elements for which the load-slip characteristics were initially selected on the basis of tests made on Finnish profiles [2]. Characteristic for them is that they represent good resistance for both vertical separation and longitudinal shear. In order to make parametric studies, reference values for the nominal longitudinal shear strength, $\tau_{u.R}$ (EC4 definition), are related to slabs with total depth h equal to 120 mm and shear spans L_s equal to 350 mm (Fig. 1a). Notation $\tau(h, L_s)$ is used to identify various combinations of slab depth and shear spans, and these are considered with reference to strength $\tau(h, L_s) = \tau(120, 350)$ for which values 0.95, 0.9, 0.59, 0.52 and 0.45 MPa were given.

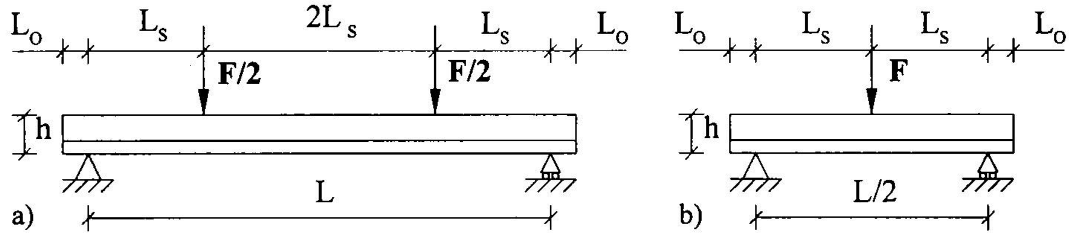

Fig. 1 Load configurations for slab tests: a) two loads according EC4 and b) single central load

3.1 Variability of $\tau_{u.R}$ in slabs with ductile shear connections

Slabs meeting the ductility requirement of EC4 were analyzed using the LBE method, and calculation results interpreted into τ-values are shown in Fig. 2 for various slab depths and shear spans. Uniform loading and loadings according to Fig. 1 above were applied, but for the uniform loading the shear span of L/3 was assumed, as this is in better agreement with the reality than L/4 [2]. According to the results, rate $\Delta M_{test}/\Delta L_s$ shows only minimal variation and is nearly constant. Extrapolation in Fig. 2 is based on this fact.

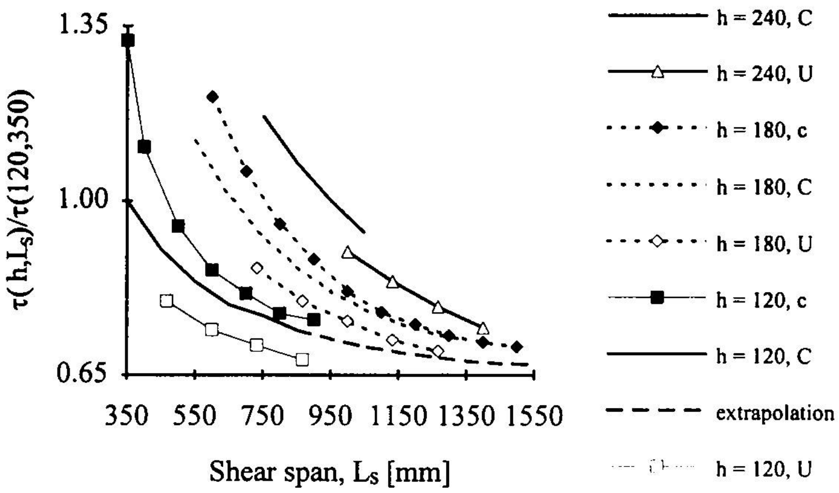

Fig. 2 Bond strength variation for strength level τ(120, 350) = 0.45 MPa in slabs with two concentrated loads according to EC4 (id. = C), a uniformly distributed load (id. = U) and a single concentrated load (id. = c) for different depths h and shear spans L_s of the slab.

3.2 Effect of non-ductility

Non-ductile and ductile behaviour of uniformly loaded slabs are introduced in Fig. 7.12 of ENV 1994-1-1 and the effect of non-ductility was studied by assuming for the connection a load-slip relationship in which the loads drops to 0 or 50 % of the maximum immediately after the peak is passed, and then the load remains constant. Although the cases considered should be classified as brittle, the results were interpreted into $\tau_{u.R}$-values represented as curves with the indentifier 'b' in Fig. 3. The difference in the load drop does not affect the ultimate load of the slabs, as the maximum shear forces are obtained simultaneously on the length of the shear spans. It should also be noted that the difference between ductile (d-curves) and brittle cases (b-curves) in Fig. 3 is only slight, and the trend in the curves is similar to that in Fig. 2.

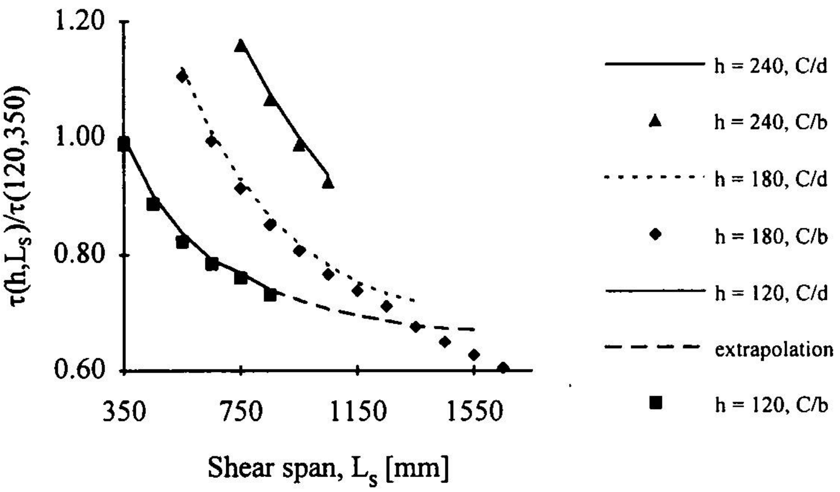

Fig. 3 Bond strength variation for strength level τ(120, 350) = 0.45 MPa for slabs with ductile (id. = d) and non-ductile (id. = b) shear connections and loaded according to EC4. Depth h and shear spans L_s in the slab are varied.

3.3 Effect of additional reinforcement

Additional reinforcements A_s are used in composite slabs for improving their fire resistance and the reinforcement is normally omitted in the normal temperature design, although a method is given in Annex E of ENV 1994-1-1 for the allowance of the bar reinforcement. A constant axis distance of 40 mm was assumed in LBE calculation and the resistances obtained, M_{test}, are compared with the resistance M_{Rd} given in E.5(1) of ENV 1994-1-1, calculated according to variable strength $\tau(h, L_s)$ in which A_s is omitted, i.e. the minimum of the bond strength was not applied, as the real effect of the reinforcement was studied here. In all strength levels considered, similar results were obtained and the curves for $\tau(120, 350) = 0.45$ MPa are represented in Fig. 4.

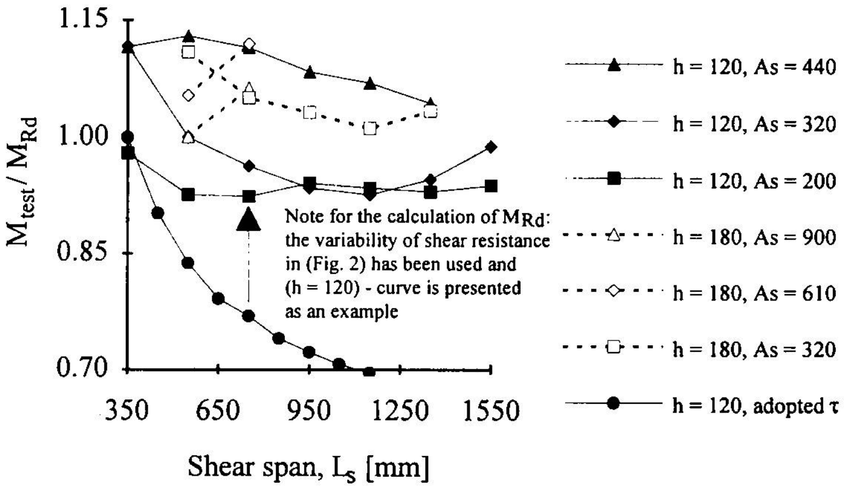

Fig. 4 Variation of bending resistance for slabs having additional reinforcement A_s when depth h and shear spans L_s are varied in slabs with $\tau(120, 350) = 0.45$ MPa, loaded according to EC4

3.4 Calculation results interpreted by the m-k method

The numerical analyses discussed above were also interpreted into coordinates for the m-k method, and the results for the strength level of the previous figure are shown in Fig. 5. A good correlation into linear regression lines is seen, when constant slab depths are considered, and all the lines cross the vertical axis practically at the same point (k), but there is a large scatter in the inclination (m), showing primarily the effect of the slab depth. The scatter is intensified with smaller strengths of the connection. In order to avoid the scatter in m, a change in the horizontal axis should be made, and it proved best to multiply the original axis by the effective depth of the slab, d_p, i.e. $A_p d_p/(bL_s)$ will be the new axis. Thereby a practically constant m is obtained in Fig. 6 for the data of Fig. 5 and there is now scatter in the values of k, the importance of which in the resistance evaluation is smaller, however.

3.5 Effect of friction at supports and number of failure mechanisms

The Australian shear block test [4] considers the friction in the shear interface, and due allowance is made for it in the design of slabs. In the LBE analyses friction at supports may be considered locally or globally as distributed to the length of the effective connection. The latter option is the one applied in the analyses above, and accordingly the strengths employed for the shorter shear spans are slightly higher than in reality. When the resistance of the slab decreases, the effect of friction will also become smaller, whereas in thicker slabs with short shear spans the share of friction in the resistance will be higher. In fact, the reason why the lines in Fig. 5 do not cross the vertical axis exactly at the same point can be explained by means of friction: for shallow slabs the true scatter would be larger than that presented here and for thick and short slabs it would be smaller in the case where the friction is taken into account in the most appropriate manner, i.e. as a summary the effect of friction will not invalidate the principles of the behaviour presented above.

In laboratory tests it is frequently observed that clearly higher ultimate loads are obtained in such slabs where both ends are failing simultaneously due to excessive slipping, as compared to the cases of single end failure which represent the determining values. This can be explained by means of energy considerations. A symmetrical failure presupposes that the bond properties are very uniformly distributed along the shear spans which is typical of slabs with high bond strength. Then it is also expected that the overall scatter would be smaller.

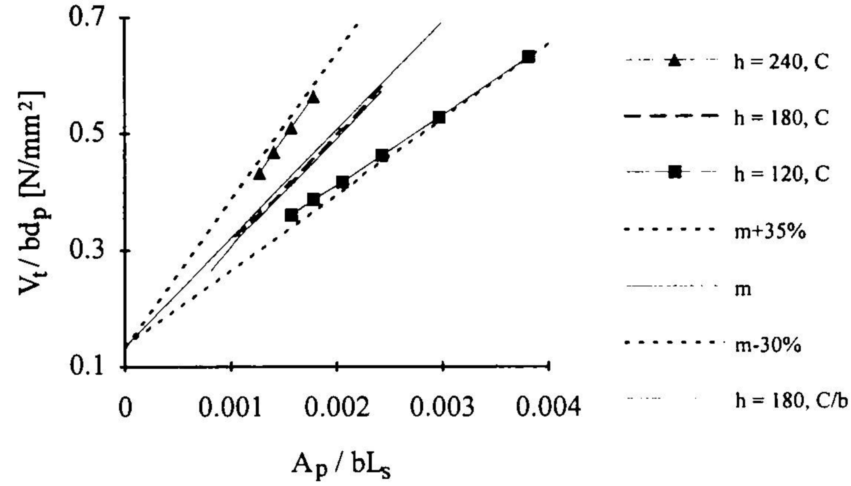

Fig. 5 Various analyses of slabs loaded according to EC4 and interpreted according to the m-k method, assuming that τ(120, 350) = 0.45 MPa, and h and L_s are varied. Coordinates are as given in EC4 [1].

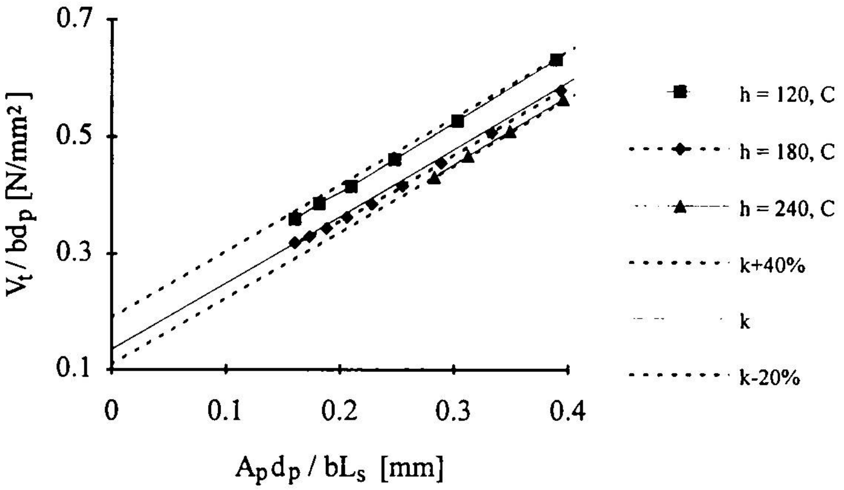

Fig. 6 Various analyses of slabs loaded according to EC4 and interpreted according to the modified m-k method, assuming that τ(120, 350) = 0.45 MPa, and h and L_s are varied. Coordinates given in EC4 are modified in the horizontal axis.

4. Conclusions

The bond strength will decrease with increasing shear span and it will increase with increasing slab depth, independent of load distribution, and the minimum value for this strength is obtained from slabs with a minimum depth (see Figs. 2 and 5). The smaller the average bond strength, the higher is the scatter in the results. The best strength, relatively, is obtained by employing a single concentrated load on the slab, and the strength will become gradually smaller when the load configuration is changed towards more uniform distribution (Fig. 2). For long shear spans the results approach an asymptote represented by the minimum value obtained from the double load test in EC4. Thus it would be cost-efficient to employ a single load configuration when long shear spans are required.

Ductile and non-ductile shear connections that have similar resistance per unit length behave basically in the same manner when shear spans not longer than one metre are concerned and the test results are interpreted according to the partial shear connection method, but for longer shear spans with non-ductile connections smaller strengths than for ductile connections of similar resistance will be obtained (Fig. 3). Therefore there should be no reasons for not allowing the use of this method for a wide range of so-called non-ductile connections, as it is always possible to apply a greater reduction when evaluating the characteristic value of $\tau_{u,R}$. In the m-k method the non-ductility has practically no influence on the regression line, due to the coordinate axes employed, and this is also seen in Fig. 5 in which one non-ductile set of results (identifier C/b) is presented together with ductile sets.

The principal effect of additional reinforcing bars is to improve the bond strength by reducing the crack widths, and the mode of failure can change to pure bending. All the results in Fig. 4 satisfy the requirement, $M_{test} \geq 0.9M_{Rd}$, and the additional tests required for reinforced slabs in EC4 are thus not necessary.

It was clearly shown that the m-k method is well suited for evaluating the resistance of slabs with respect to connection failures, and a reliable value for parameter k is obtained when constant depth is used for all specimens, varying only the length of the slabs (Fig. 5). For the reliability of the results, the minimum depth employed in practice should be selected for tests. Various depths of the slab represent clearly different values for parameter m, and the results will not be rational when varying depth are used in the regression analysis, i.e. the inclination of such line will be arbitrary. If the horizontal axis is modified so as to include the effect of the slab depth, the inclination of the regression lines will represent the real behaviour as far as constant depth is used in all tests, and it only remains to decide a reliable value for k. It should be noted that when working in the coordinates proposed, the slabs with the maximum depth are the determining ones, which is opposite to the τ_u method. It is finally pointed out that the research reported in this paper also includes principles for interfacing the m-k method and its modifications with the partial shear connection method, and a calculation formula was derived with which any single tested bond strength τ_u can easily be converted into other values of τ_u for other shear spans, while keeping the depth of the slab constant. This makes it possible to re-evaluate old tests for the harmonisation purposes.

References

[1] Eurocode 4 (1992). "Design of Composite Steel and Concrete Structures, Part 1-1: General rules and rules for buildings", ENV 1994-1-1:1992. CEN

[2] Tenhovuori, A., Kärkkäinen, K. and Kanerva, P. "Parameters and Definitions for Classifying the Behaviour of Composite Slabs". Composite Construction III, Proceedings of an Engineering Foundation Conference, Irsee, Germany, 1996.

[3] Leskelä, M.V. (1992) "A Finite Beam Element for Layered Structures and Its Use when Analysing Steel-Concrete Composite Flexural Members". Constructional Steel Design: World Developments, 354-358. Elsevier Applied Science

[4] Patrick, M., (1990) "A New Partial Shear Connection Strength Model for Composite Slabs". Journal of the Australian Institute of Steel Construction, V.24, No3, 2-17