STRENGTH OF COMPOSITE SLABS: COMPARISON OF BASIC PARAMETERS AND THEIR BACKGROUND

Matti V. Leskelä

Rakenteiden Mekaniikka, Vol. 25
No 2 1992, ss. 20 - 38

ABSTRACT: Composite slabs are a special type of concrete slab which have their tensile reinforcement in the form of profiled steel sheeting. For economical use of the steel sheeting, adequate anchorage must be provided in order to allow full plastic flexural resistance in the slab. The design problems involved in composite slabs are closely connected with predicting the anchorage resistance required for verification of slab performance.

There are two types of anchorage parameter which can be used to determine the ultimate load resistance of a composite slab for design purposes, (1) a nominal bond or anchorage strength, calculated for a horizontal project area of the sheet surface along the shear span of the slab, and (2) parameters m and k for determining the ultimate shear resistance of the slab independent of the failure type.

Both parameters are determined experimentally but questions sometimes arise to whether these parameters have something in common or not, whether there is any correlation between the two, or whether the nominal bond strength performs better for describing the true flexural behaviour up to failure. In brief, this paper is intended to study, whether the coefficients m and k can be considered to give a reliable prediction of the ultimate resistance if the slab has the nominal bond strength determined for it beforehand. For this purpose, a formulation is provided for predicting the ultimate shear resistance of the slab in the case of a typical test setup, i.e. the slab tests are simulated on the assumption that the true bond strength of the steel sheeting is known. Some numerical examples are given to study the correlation. Eurocode-type notation (EC4, Part 1, section 1.6) is used for the symbols where this proves to be reasonable. The compressive strength of the concrete is defined according to the Finnish tradition, with respect to the cube strength.

INTRODUCTION

Profiled light-gauge steel sheetings are a useful form of concrete reinforcement because their cross-sectional area is considerable, in the normal case of 0,9 mm sheet thickness far more than 1000 mm^2 of cross-sectional area is obtained. If no special means are provided for bonding the sheeting to the concrete, the profile cannot hardly be effectively utilised as a reinforcement, because the tensile stress resultant must be maintained in order to develop full plastic flexural resistance in the slab.

There are two types of anchorage parameters available to determine the ultimate load resistance of a composite slab for design purposes,

(a) τ_{ud}, a nominal design bond strength calculated for a horizontal area $b \cdot l_b$, b being the design width and l_b the anchorage length along the span of the slab. τ_{ud} is determined experimentally.

(b) parameters m and k for determining the shear resistance of the slab in the case of an anchorage failure occurring. The coefficients m and k are based on experiments carried out with full-scale test slabs. The results are analysed using a standard procedure given in EC4 and its background documents /1, 3/.

NOMINAL BOND STRENGTH τ_u

The concept of horizontal shear resistance τ_u in relation to partial shear connection theory is only suitable for composite slabs with a ductile behaviour as defined in EC4, Part 1, 10.3.1.4 /1/:

> The behaviour of a test slab is classified as brittle if the failure load does not exceed the load causing first end slip by more than 10 %. If the maximum load is reached at a midspan deflection exceeding L/50, the failure load shall be taken as the load at the midspan deflection of L/50. All other cases are classified as ductile.

The definition given above is not quite exact. No indication is given in this section for detecting the first end slip of the sheeting, and in many cases it would be a matter of a good guess to judge the initiation of slipping unanimously from the test recordings. Luckily enough, a very careful scrolling through paragraphs other than 10.3 of EC4/Part 1 reveals that the initial slip load is defined in passing when considering deflections in composite slabs in the paragraph 7.6.2.2, clause (9): the load causing an end slip of 0.5 mm shall be called the initial slip load.

Anyway, the idea behind the definition is clear: in order to develop a plastic distribution for the connection shear flow, a certain amount of end slip is required to be sure that the tensile stress resultant of the sheeting, F_t is anchored by an evenly distributed horizontal shear stress τ_u, see Fig. 1.

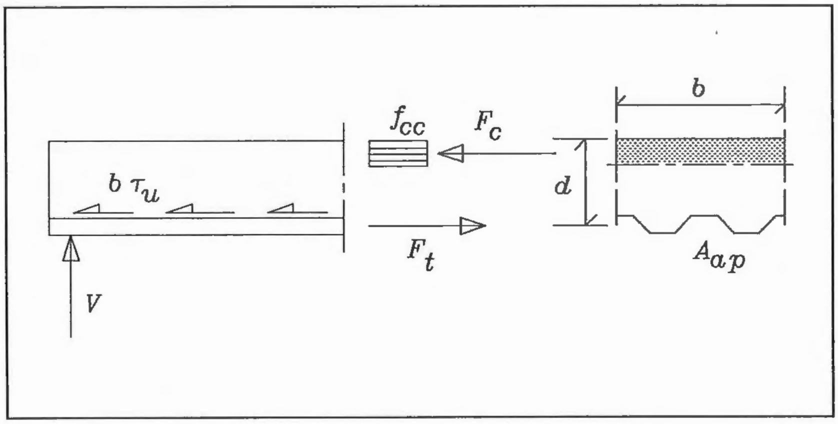

Fig. 1. If the connection is ductile, the bond stresses in the connection interface can develop a plastic uniform distribution τ_u

PARAMETERS m and k

There is no mechanical background for the parameters m and k, although such ideas have sometimes erraneously been put forward. The coefficients are solely based on the evaluation of test results in certain coordinates: it has been found that there is a close correlation between the maximum vertical shear force obtained in the tested slabs and their principal properties (see Fig 1.):

- depth of the slab, d,
- shear span of the slab, L_s,
- cross-sectional area of the sheeting, A_{ap},
- tensile strength, f_{ct}, or some other similar strength parameter of the concrete.

If these properties are varied, the test results will show nearly linear behaviour in the coordinates given in Fig. 2., hence a linear regression analysis can be applied to evaluate the tests and to give a design relationship for the maximum vertical shear force predicting the ultimate resistance and the principal properties of the slab /1/.

$$V_{l.Rd} = (m\rho d/L_s + kf_{ctd})bd/\gamma_V\,, \quad \gamma_V = 1.25 \tag{1}$$

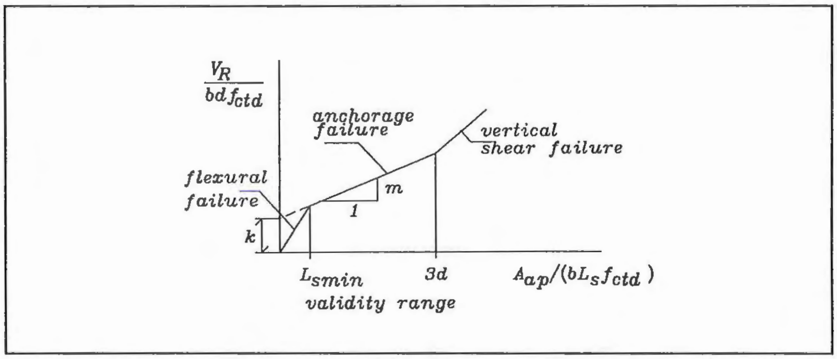

Fig. 2. Evaluation of the test results to give the design parameters m and k

SLAB TESTS AND FAILURE MODES

The parametric tests are carried out using simply supported slabs with two equal concentrated loads placed symmetrically at L/4 and 3L/4 on the span, Fig. 3. Thus the shear span L_s = L/4, depth of the slab, d, and the longitudinal shear strength τ_u are the quantities that have to be estimated in order to evaluate failure. Although not exactly constant, τ_u can be given approximately as a fixed value depending only on the properties of the steel sheeting (embossments, profile shape, sheet thickness, indentations, etc.).

Let τ_u be a known property of the slab and the sheeting considered hereafter. Let it also be assumed that the whole of the sheet profile is stressed in tension. If the shear span L_s is varied, one of the failure modes explained below will take place during loading:

(1) *Flexural failure*

L_s must be greater than a certain minimum value L_{smin} to ensure the prevention of premature anchorage failure. Here the term 'premature' is used to emphasize that a full plastic ultimate moment can develop before the final failure.

(2) *Anchorage failure in connection with flexural cracking on shear spans*

Normally this means that bending moments on the shear spans are high enough to induce flexural cracks, which tend towards the concentrated loads due to interaction with the vertical shear force.

(3) *Anchorage failure in connection with a high longitudinal shear flow on shear spans which still remain uncracked, no end slip produced*

Normally this means that the shear spans must be short. When the shear flow reaches the critical value $b\tau_u$, anchorage failure will take place simultaneously with the diagonal or vertical cracking of the concrete and a total collapse will ensue. This type of failure is not usual and belongs to non-ductile rigid connections.

(4) *Diagonal tensile failure of concrete*

For very short shear spans diagonal tension failure can occur in the concrete. Normally this type of failure requires a load which is very much higher than in the other modes, because part of the load is transferred to the support by the arching effect formed on the short shear spans.

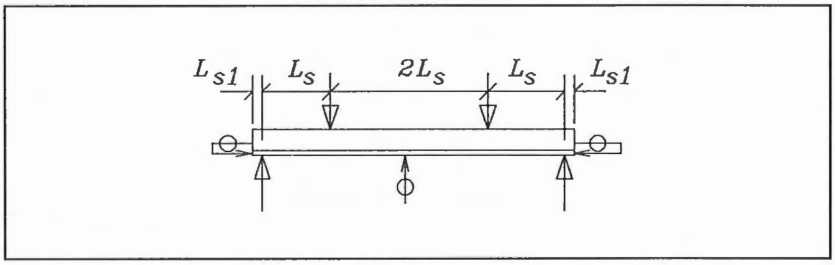

Fig. 3. Test arrangement for the determination of parameters m and k. Central deflection is measured for definition of the ultimate load in the test. End slip between the concrete and the steel profile is measured in order to define the initial slip load and the mode of failure: the mode is defined as flexural if initiation of slipping is lacking before the ultimate load.

FAILURE PREDICTION

Equations for predicting the failure loads for different span and depth configurations are derived on the basis of the material and mechanical definitions listed below.

(1) The flexural cracking resistance of the slab, $M_{cr.R}$, is predicted according to the ultimate tensile strain ε_{ctu} at the level of the centroid of the sheet profile. Although ε_{ctu} may have a somewhat random character, measurements suggest that it will most likely be $\varepsilon_{ctu} = 0.2 \ldots 0.4$ o/oo. Thus

$$M_{cr.R} = \varepsilon_{ctu} (EI)_i / e_a, \tag{2}$$

$(EI)_i =$ composite flexural stiffness of the uncracked slab cross-section,

$e_a =$ distance of the sheet profile centroid from the neutral axis.

$(EI)_i$ is calculated on the basis of the complete interaction using the formula

$$\begin{aligned}
(EI)_i &= (1 + \alpha_i)(EI)_c, \\
(EI)_c &= E_c b h_c^3 / 12, \\
(EA)_c &= E_c b h_c, \\
e_i &= d_a + h_c / 2, \\
\alpha_i &= e_i^2 (EA)_{ap} / (EI)_c, \\
e_{c0} &= (0.5 h_c (EA)_c + d (EA)_{ap}) / ((EA)_c + (EA)_{ap}), \\
e_a &= d - e_{c0}.
\end{aligned} \tag{3}$$

(2) The longitudinal shear flow along the shear span is calculated with respect to the vertical shear force V,

$$v_l = (EA)_{ap} e_a V / (EI)_i. \tag{4}$$

If the maximum value for v_l is $b\tau_u$, the failure load for the mode (3) is

$$V_{vl.R} = b\tau_u (EI)_i / ((EA)_{ap} e_a). \tag{5}$$

(3) The shear stresses of the concrete on the level of the neutral axis are calculated accordingly as

$$\begin{aligned}
\tau_c &= (EA)_{c0} e_c V / (EI)_i, \\
(EA)_{c0} &= E_{cd} b h_c / 2, \quad e_c = e_{c0} / 2.
\end{aligned} \tag{6}$$

If the maximum shear stress to be reached is f_{ct}, the failure load for the mode (4) is

$$V_{dt.R} = f_{ct} (EI)_i /((EA)_{c0}\, e_c). \tag{7}$$

(4) The effective design secant modulus of the concrete, E_c , is calculated according to EC2/Part 1 /2/

$$E_{cd} = 9525(0.8K + 8)^{1/3}, \quad (MPa), \tag{8}$$
$$K = \text{cube strength of the concrete (MPa)}.$$

In order to allow for the incomplete interaction due to the bond slip, a reduced value $E_{cd} = E_c /1.5$ could be used for the deflection calculations.

(5) The flexural resistance of the slab in the case of full shear connection ($L_s \geq L_{smin}$) is calculated according to the reinforced concrete formula

$$M_{pl.R} = \mu bd^2 f_{cc}, \quad f_{cc} = 0.7K,$$
$$\mu = \omega(1 - \omega/2), \quad \omega = A_{ap} f_y /(bdf_{cc}). \tag{9}$$

For the cases $L_s \geq L_{smin}$ (L_{smin} is defined in the next section), the failure load is

$$V_{mpl.R} = M_{pl.R} /L_s . \tag{10}$$

FLEXURAL FAILURE VS. ANCHORAGE FAILURE

For certain shear spans $L_s > L_{smin}$ the bond resistance is capable of preventing premature anchorage failure along the sheared connection interface. Let the uncracked section of the shear span be denoted with l_b = length in which the longitudinal shear flow can develop to its plastic value $b\tau_u$. In the cracked section of the shear span the bond will have deteriorated because the stiffness of the concrete between the adjacent cracks is not sufficient to transfer the full shear flow of the connection into the compressed concrete. Thus a linear variation from $b\tau_u$ to zero is assumed for this section (see Fig. 4).

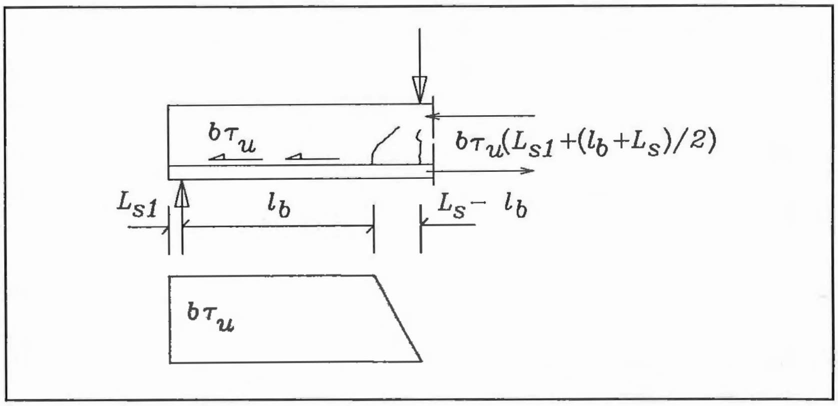

Fig. 4. Partial plastic connection shear flow distribution for the failure load estimation $V_{I.R}$

If the plastic yielding of the steel profile is to be reached before the initial slip load, the following condition must be satisfied

$$A_{ap} f_y \leq b\tau_{bis} [L_{s1} + (L_s + l_b)/2], \quad \text{or} \tag{11}$$
$$l_b \geq 2A_{ap} f_y/(b\tau_{bis}) - L_s - 2L_{s1} .$$

τ_{bis} denotes the nominal bond stress τ_b at the moment of the initial slip. l_b is estimated according to the anchorage properties. Let τ_b be distributed uniformly along the total L_s provided that there are no other cracks than that under the point load. When $\tau_b < \tau_{bis}$, more cracks can form on the shear spans on condition that the tensile resultant F_{cr} inducing cracking is exceeded in the connection interface (Fig. 5). F_{cr} is calculated from the ultimate tensile strain of the concrete,

$$F_{cr} = \varepsilon_{ctu} /(\frac{1}{(EA)_c} + \frac{d^2}{4(EI)_c}) \tag{12}$$

$$l_b = \frac{1}{2} (\frac{F_{cr}}{\tau_{bis} b} + L_s - L_{s1}) \tag{13}$$

l_b can now be inserted into equation (11) in order to solve it for L_s,

$$L_s \geq \frac{4}{3} \frac{A_{ap} f_y}{b\tau_{bis}} - L_{s1} - \frac{F_{cr}}{3\tau_{bis} b} = L_{smin} \tag{14}$$

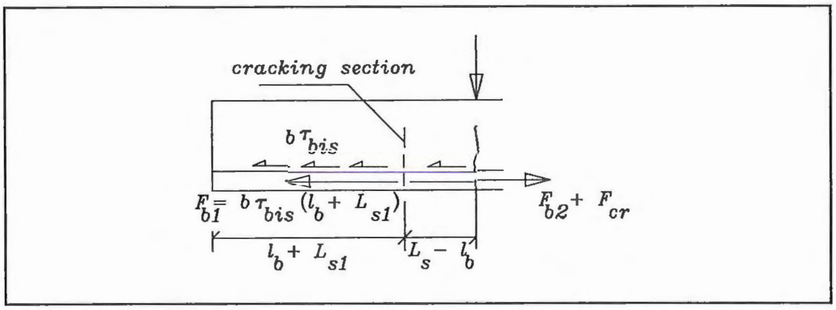

Fig. 5. Anchorage length l_b is evaluated according to the possibility of an additional crack forming, if the resultant F_{b1} - F_{b2} > F_{cr}

Let us now assume that $L_s < L_{smin}$, i.e. full flexural resistance $M_{pl.R}$ cannot be reached. According to the flexural equilibrium and the ultimate limit state of the rotating section (see Fig. 4), we can write

$$V_{I.R}\, L_s = (d - x/2)b\tau_u\, [L_{s1} + (L_s + l_b)/2], \tag{15}$$

$$x = \tau_u\, [L_{s1} + (L_s + l_b)/2]/f_{cc},$$

thus

$$V_{I.R} = \frac{b\tau_u}{L_s}\{d - \frac{\tau_u}{2f_{cc}}[L_{s1} + (L_s + l_b)/2]\}[L_{s1} + (L_s + l_b)/2] \tag{16}$$

Equation (16) is based solely on the force equilibrium and does not consider the failure condition of the concrete at the edge of the principal crack. Thus it shows only the upper bound for the failure load, which still has to be evaluated from the failure condition of the concrete.

Let the stresses of the compression zone be σ_c and τ_c. Failure will ensue when the principal tensile stress due to the components σ_c and τ_c becomes equal to the tensile strength, i.e.

$$f_{ct} = -\sigma_c/2 + \sqrt{(\sigma_c/2)^2 + \tau_c^2}\,. \tag{17}$$

If a parabolic distribution is assumed for the shear stresses τ_c and normal stresses σ_c, this equation can be further manipulated to give

$$V_{I.R} \leq (L_{s1} + l_b)b\tau_u\sqrt{(f_{ct}/\sigma_c)^2 + (f_{ct}/\sigma_c)} + V_{ap.R}, \tag{18}$$

in which $V_{ap.R}$ denotes the shear resistance of the steel sheeting alone. In order to manage with equation (18), the normal stress σ_c needs to estimated. A natural guess for it will be $\sigma_c = f_{cc}$. Furthermore, $V_{ap.R}$ should be estimated according to the profile form of the sheeting.

One further boundary case has to be considered: the yielding of the steel profile for situations $L_s < L_{smin}$. Although the connection is capable of letting the steel profile develop a yield, failure must be considered to be of the anchorage type if initial slip has occurred. This most propably means that the slab cannot develop full plastic resistance $M_{pl.R}$ due to slipping. This time equation (16) predicts the failure load directly according to $M_{pl.R}/L_s$ or higher, if no boundary conditions are set.

To simulate the real situation, we assume that $0.8M_{pl.R}$ can be reached if yielding is entered at the same time as τ_u develops. If yielding is entered at the moment of reaching τ_{bis}, it is assumed that full plastic moment can develop. The linear variation is set between $(0.8 \ldots 1.0)M_{pl.R}$, and the ultimate load is calculated according to the maximum moment evaluated for the case. It must noted that reduction 0.8 may be true for some sheetings but not unconditionally for all.

NUMERICAL CALCULATIONS

The calculations were performed by means of a PC program designed to use the formulation given. The procedure for the determination of the most probable failure mode to take place may be listed briefly as follows:

(1) Calculate L_{smin}, $V_{dt.R}$, $V_{vl.R}$, $V_{l.R}$ and $V_{mpl.R}$. For $V_{l.R}$ apply equations (16) and (18) to get the minimum value. Check that $V_{l.R}$ does not exceed the boundary value defined by yielding,

(2) If $L_s > L_{smin}$, take $V_{mpl.R}$ for the resistance,

(3) If $L_s \leq L_{smin}$, take $V_{l.R}$ for the resistance,
Furthermore, check that upper bounds $V_{dt.R}$ or $V_{vl.R}$ are not exceeded by $V_{l.R}$ (rigid, non-ductile connection).

Three slab depths, having values d = 105, 145 and 185 mm were employed for the calculation, and a representative cross-section area $A_{ap} = 1200\ mm^2$ was assumed for fictitious sheeting having a yield strength $f_y = 360$ MPa. The bond strength levels τ_u = 0.35, 0.50, 0.65 and 0.80 MPa were used to study whether the formulation will yield any reasonable results or not. The 'randomness' of the cracking of the concrete was modelled by letting ε_{ctu} have values in the range 0.2 .. 0.4 o/oo. The variation in ε_{ctu} directly affects the value of l_b. The influence of different slip performances was studied by using two ductility settings. A more ductile connection was considered by setting $\tau_{bis} = 0.5\tau_u$, and a less ductile one was considered by setting $\tau_{bis} = 0{,}9\tau_u$, which can still be classified as ductile according to EC4.

The most representative of the calculation results are gathered together in Tables 1. and 2. Different shear spans were employed in the range of $L_{smin} > L_s > 3d$. A linear regression analysis was used to see the correlation of the results for a line in the coordinates given in Fig 2. The correlation coefficient r = ± 1 indicates a perfect fit while r = 0 would imply no fit at all. Figs. 6. to 9. present the calculation results when the less ductile connection was employed.

Table 1. Coefficients m and k and correlation coefficient r from a linear regression analysis employing different sets of parameters τ_u when eighteen values for $V_{l.R}$ were calculated for each τ_u. Three depths d were used, d = 105, 145 and 185 mm, and two shear spans for each slab depth. Three values for ε_{ctu} were given to simulate the 'randomness of cracking', ε_{ctu} = 0.2, 0.3 and 0.4 o/oo. One concrete strength class was employed, K35 (= cube strength 35 MPa), τ_{bis}/τ_u = 0.5 was assumed for all cases.

τ_u	m	k	r
0.80 MPa	281.62	0.039	0.994
0.65 MPa	284.94	0.030	0.996
0.50 MPa	262.16	0.036	0.993
0.35 MPa	111.82	0.116	0.995

Table 2. Coefficients m and k and correlation coefficient r from a linear regression analysis employing different sets of parameters τ_u when eighteen values for $V_{l.R}$ were calculated for each τ_u. The same three values for ε_{ctu} were used as in Table 1. Concrete strength class K35, $\tau_{bis}/\tau_u = 0.9$ was supposed for all cases.

Slab depths d = 105, 145, 185 mm			
τ_u	m	k	r
0.35 MPa	47.89	0.180	0.876
0.50 MPa	96.63	0.198	0.855
Slab depths d = 75, 95, 115 mm			
τ_u	m	k	r
0.65 MPa	72.20	0.251	0.925
0.80 MPa	89.14	0.296	0.909

Considering now 'test results' of Table 1., we can see that there is no systematic change in the correlation coefficient between the different effectivenesses of the bond. Only one concrete class was employed (the same is also true of real testing), and this may influence the minimum scatter of the values. It seems that there is no clear difference between classes $\tau_u = 0.80$ and $\tau_u = 0.65$ MPa for the coefficients m and k. Only the class having $\tau_u = 0.35$ MPa differs markedly from the others, and this class may be considered to have only a moderate bond effectiveness. One factor should be noted between the groups having 'moderate' and 'fair' bonds: it seems as though the dependence of the resistance on the shear span is greater for a fair bond than for a moderate one. This is seen from the values for the coefficients k, which clearly regulate the dependence on the strength of the concrete.

The contents of Table 1 may look 'too good', and it must be asked whether the perfect fit has something to do with the selections made. Most of all these include the ratio τ_{bis}/τ_u, which characterizes the ductility of the connection. More calculations were made to see what would happen if the connection were more brittle. Still remaining in the area of a ductile connection, the ratio $\tau_{bis}/\tau_u = 0.9$ was chosen according to the limit at which the connection would still be classified as ductile. The results of this calculation are shown in Table 2. A clear trend was noted for this computation: if the slab depths were kept the same for all bond classes, there was a clear reduction in the correlation coefficient as the bond strength increased. This is due to the decrease in L_{smin} relative to the increasing bond, i.e. the span (L_{smin} - 3d) grows smaller and it is not possible to maintain it within reasonable limits unless the slab depths employed are reduced for higher bonds. The correlations could be maintained satisfactorily for the depths chosen.

It can be also expected that all the classes in Table 2. would have yielded correlation coefficients $r \geq 0.9$ if the depths had been chosen appropriately. The sets employed are shown to emphasize that the selections made for testing the slab are of great importance for the interpretation of its behaviour and parameters. These questions are discussed further in the next section.

TESTING OF COMPOSITE FLOOR SLABS IN EC4

The rules given in EC4/Part 1, section 10.3, aim only to explain the test for determining the behaviour in the anchorage failure mode, i.e. the main reason for testing the slabs is to obtain values for m and k to use in design (see equation (1)), or to find τ_{ud} for the partial connection theory explained in EC4/Part 1, annex F.

A minimum of six tests is required to determine the appropriate m and k. The specimens should be divided into groups of two or three so that the shear span in one group should be as long as possible whilst still providing an anchorage failure and that in the other group as short as possible whilst still providing the same mode of failure. As it is a well-known fact that if the shear span is shortened unlimitedly part of the shear force is transmitted directly to the support, the shear span in tests must not be less than 3d.

The reliability of the system is clearly dependent of the grouping of the specimens, i.e. arranging the test plot groups to be as far from each other as possible. Another point of importance is evaluation of the test results, i.e. how to judge the real failure mode. The spirit of EC4 is that all test results producing a failure load less than $V_{mpl.R}$ should be classified as anchorage failures. This avoids use of the reduction coefficients otherwise required to evaluate flexural resistance.

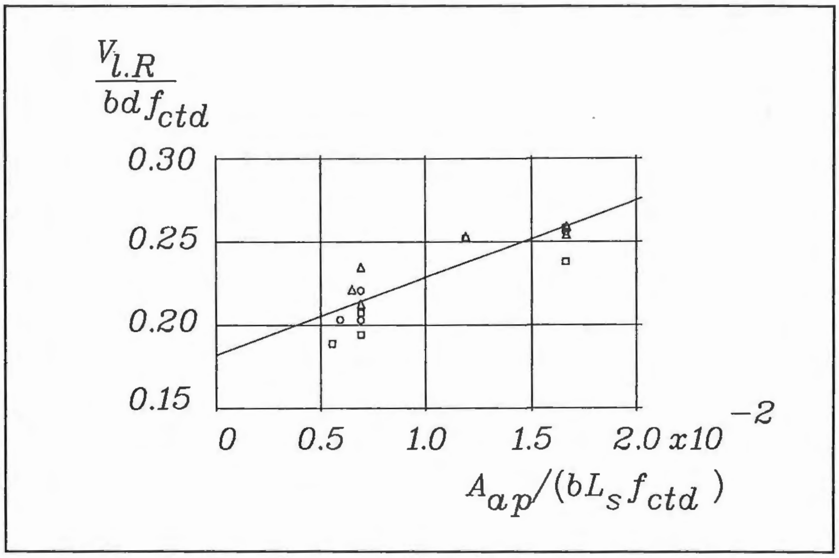

Fig. 6. Plot of calculation data for τ_u = 0.35 MPa, τ_{bis}/τ_u = 0.9. Eighteen data points for three slab depths d = 105, 145, 185 mm. Correlation with the line of best fit, r = 0.876.

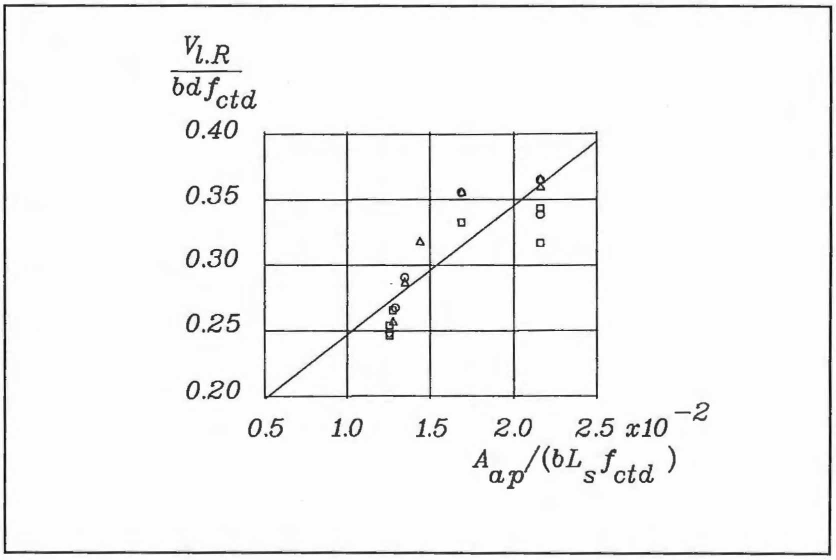

Fig. 7. Plot of calculation data for τ_u = 0.50 MPa, τ_{bis}/τ_u = 0.9. Eighteen data points for three slab depths d = 105, 145, 185 mm. Correlation with the line of best fit, r = 0.855.

The correlation coefficient in Table 1. is practically unity, i.e. there is a perfect correlation for the regression line in all cases considered. But the question arises of what to do if the bond is near 'perfect', or the connection shows a very rigid performance, i.e. there is only a small range of shear spans having a possibility to produce an ancorage failure. The expectations can be postulated as follows:

- for sheetings having only a moderate bond strength it is possible to arrange test groups easily as one wishes, because the range of L_s producing anchorage failures is not limited due to the high value of (L_{smin} - 3d),

- for the sheeting having an effective bond, (L_{smin} - 3d) becomes quite small and it may not be possible to find easily an appropriate range of test setups to characterize the idea of two groups, i.e. if only tests with an anchorage failure are approved, the plots will be relatively close together, unless there is some pre-knowledge of the characteristics of the connection available to choose the slab depths appropriately. Wrong selections will produce a poor correlation factor.

In section 10.3.1.4 of EC4/Part 1, clause (5), a simple determination rule for the design relationship m and k is given on condition that no deviation in any individual test result from the mean for its group exceeds 10 %:

- semi-characteristic values are obtained from the two groups by taking the minimum value for the group reduced by 10 %. The straight line explaining the dependence of the resistance $V_{l.R}$ on the parameters m and k is formed through these semi-characteristic values.

In other cases a very elaborate procedure, not fully explained in EC4/Part 1, must be undertaken. Details for this procedure are found in a background document to EC3 /3/.

Nothing is said in EC4/Part 1 about the correlation required for satisfactory evaluation of the test results. As seen before, there may be a reducing correlation in the regression analysis as the bond strength increases. In the background document for EC3 /3/ the correlation for the calculation model chosen is said to be sufficient if $r \geq 0.9$, but nothing further is said about the procedure in the case of insufficient correlation. Perhaps another model for the calculation should then be chosen.

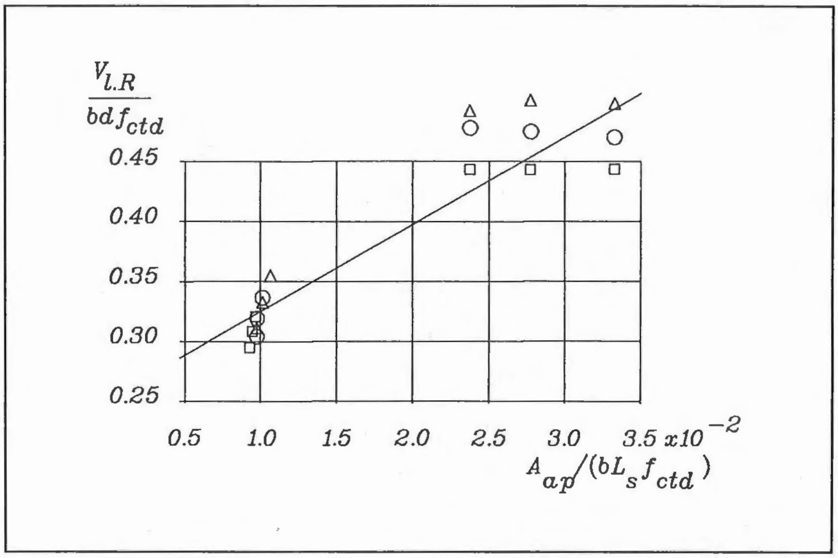

Fig. 8. Plot of calculation data for τ_u = 0.65 MPa, τ_{bis}/τ_u = 0.9. Eighteen data points for three slab depths d = 75, 95, 115 mm. Correlation with the line of best fit, r = 0.925.

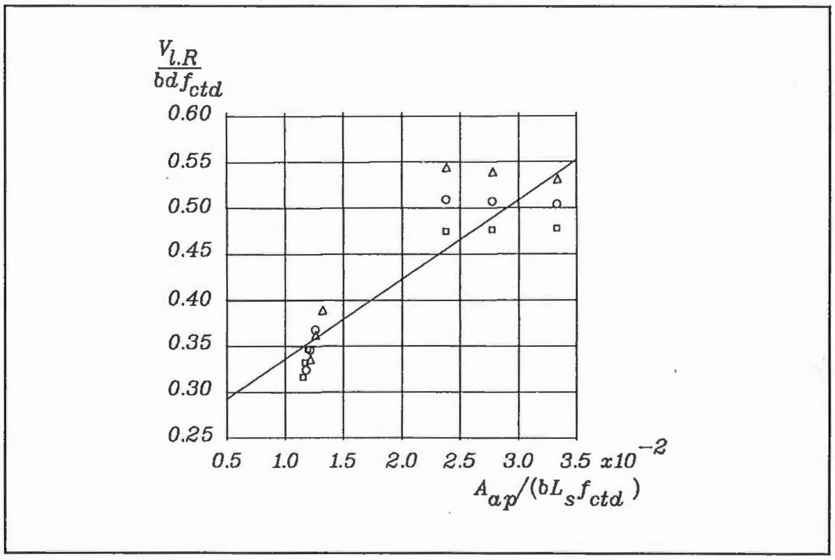

Fig. 9. Plot of calculation data for τ_u = 0.80 MPa, τ_{bis}/τ_u = 0.9. Eighteen data points for three slab depths d = 105, 145, 185 mm. Correlation with the line of best fit, r = 0.909.

DISCUSSION AND CONCLUSIONS

In the formulation derived for the anchorage type of failure concerning a loading setup with two point loads on a simply supported span, it was shown that the minimum shear span required to produce flexural failure is dependent on the bond characteristics and the ductility of the connection. This is a totally different idea from the current approach of considering L_{smin} on the basis of $A_{ap} f_y/(b\tau_u)$.

The calculations performed here show that certain pre-knowledge is preferable in order to be able to make the right decisions for the specimen dimensions (d, L_s). Interpretation of the test results is also of importance: to identify an anchorage failure, one must only check for the amount of end slip at the moment of failure (0.5 mm required to indicate the initiation of bond slipping and entry into the anchorage failure mode), independent of the plastic yielding in the zone of maximum moment.

The partial shear connection theory was not applied in the manner expressed in EC4/Part 1, due to the fact that this theory finds its best applications in cases having deep sheet profiles, i.e. part of the steel section is compressed. For the moment there is no such sheeting being used in practice in Finland, although increasing interest can be seen towards them. This paper does not claim to go deeply into the problems of where part of the steel section is compressed. It is enough to state that the theory explained here could also be used for such a problem, providing that appropriate alterations are made to the formulation concerning stress resultants and equilibrium conditions. Another paper may be prepared for this case.

ACKNOWLEDGEMENTS

Special thanks are expressed to RAUTARUUKKI OY/Paavo Rannila Oy for letting the author check the validity of the theory explained in this paper with some values from the world of real sheetings. Further thanks are expressed to HOMECON OY for their manifestations of suspicion concerning the validity of the m-k method, and for giving an impulse to the formulation of the problem. In closing, LOHJA OY must not be forgotten, in view of their enthusiasm for developing products based on composite steel sheeting.

NOTATIONS

A Eurocode type of symbolism was used in this paper where it proved to be appropriate. Due to the fact that the notations employed are not familiar, it may be useful to list those not fully explained in text.

ap	index for the steel sheeting
d	index for the design values, effective depth of the cross-section
R	index for the resistance
h_c	depth of slab over the steel sheeting
d_a	distance of the sheeting centroid from the top of the sheeting
A_{ap}	cross-sectional area of the steel sheeting
$M_{pl.R}$	flexural resistance of the cross-section, calculated according to the plastic theory
$V_{l.R}$	maximum shear force obtained for the structure before anchorage-type failure
(EA)	block notation for axial stiffness
(EI)	block notation for flexural stiffness
m	tangent modulus of the regression line, calculated according to the criterion of ordinary least squares, see /4/
k	distance of vertical axis intersection point from origo of the regression line, see /4/
r	correlation coefficient calculated from the linear regression analysis, see /4/

REFERENCES

/1/ Eurocode No. 4: Design of Composite Steel and Concrete Structures
Part 1 - General Rules and Rules for Buildings. Revised draft. Issue 1, Oct. 1990. Eurocode 4 Editorial Group

/2/ Eurocode No. 2: Design of Concrete Structures
Part 1 - General Rules and Rules for Buildings. Revised Final Draft, December 1989. Eurocode 2 Editorial Group

/3/ Report No.: BI-87-112
Procedure for the determination of design resistance from tests
November 1987, update June 1988, update November 1988
TNO Institute for Building and Materials and Structures
RWTH Aachen Institul für Stahlbau
University of Technology, Eindhoven

/4/ Wonnacot, T.H. and Wonnacot, R.J.:
Introductory Statistics, Fifth Edition
John Wiley & Sons, Singapore 1990

Matti V. Leskelä, PhD(Eng.), Academy of Finland