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Profiled Steel Sheeting and Composite Action

Helmut Bode

Roland Kunzel

Johannes Schanzenbach

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Profiled steel sheeting and composite action

by Helmut Bode[1], Roland Künzel[2] and Johannes Schanzenbach[2]

1. Summary

The use of profiled steel sheeting for buildings leads to some advantages on site and speeds up construction. In Germany - as probably worldwide - composite slab and composite beam action are taken into account for economic design purposes.

This paper deals with certain influences of the use of profiled steel sheeting on composite slab and beam action. Two main aspects are under consideration:

- strength evaluation and design calculations based on partial interaction theory;

- simplified engineering models for a better understanding, including main parameters, which govern behaviour, failure-mode and strength.

2. Composite slab action and partial shear connection

Recently very interesting experimental works on composite slabs have been carried out at TNO in Holland /1/ and in Lausanne in Switzerland /2/. Our theoretical consideration and comparison calculations are based on these experimental research works with well documented test reports.

[1] Prof. of Civ. Engrg. Universität Kaiserslautern, FRG
[2] Research Engr. Universität Kaiserslautern, FRG

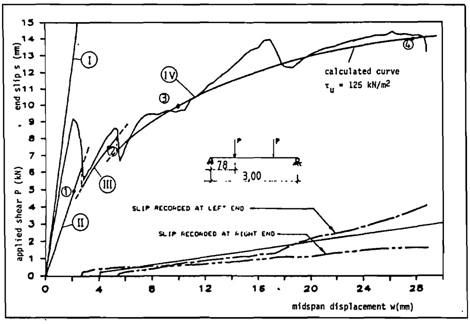


Fig. 1

Fig. 1 and 5 represent typical load deflection curves from different profiles. In both cases, the full bending strength was not reached, because of horizontal shear failure with large end slip at the steel concrete interface. These large relative displacements encouraged us to apply the partial interaction theory to evaluate ultimate loads. Stark /3/ has proposed a very similar method in case of Prins-Floors, Bode /4/ has adopted this procedure to determine the bending capacity of plain Holorib sheetings without embossments, but with additional end anchorage measures. But what is the amount of horizontal shear or bonding strength? Are pull-out or push-out tests necessary to determine these strength values?

Pull-out tests have been carried out at Bochum University /5/, and now Daniels is running such tests at Lausanne /6/. But in addition to the problem of transferring results from pull-out tests to slab tests, there is another possibility of getting relevant values: one can use an accurate computer-program for the non-linear elastic-plastic calculation of slabs (or beams), which includes non-linear horizontal shear-slip relationships at the steel-concrete interface. We are using the program "NG-Verbund" established by Schanzenbach. By means of such a program, it is possible, to calculate behaviour (for example

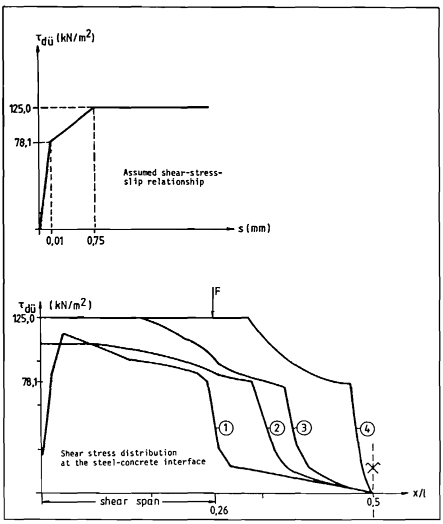


Fig. 2

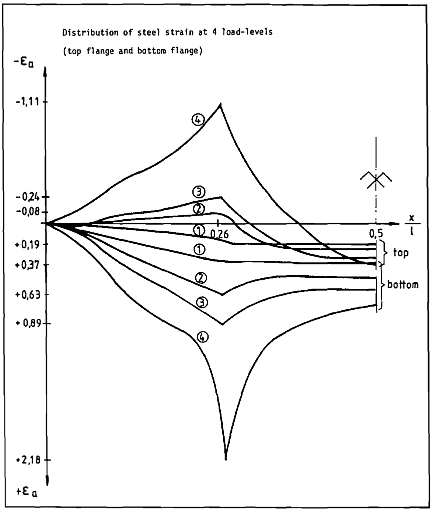


Fig. 3

deflections) and load carrying capacities of test specimens under the same loading as in the tests. By this computer simulation, the shear-slip characteristic can be calibrated such,that experimental and theoretical results are in good agreement (see fig. 1 und 5). The computer program leads among other things to the results shown in fig. 1 to 3 (in case of HI-Bond) and fig. 5 to 6 (in case of Super-Holorib). Besides distributions of strains and redistribution of the horizontal shear even into the region between the two loads where the total vertical shear force is zero, one gets characteristic shear-slip relation ships for each profiled steel sheeting. Wölfel /7/ has proposed a linear elastic procedure for the same purpose.

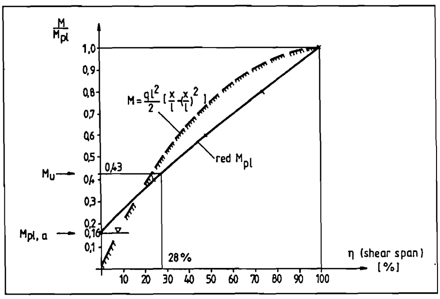


Fig. 4

Considering end slip (fig. 1 and 5) and the strain distribution in the steel part (fig 3), it is obvious, that strains are growing larger under the loading points, while the total load is approaching the failure load: the total bonding strength in the adjacent shear span ist not able to anchor the maximum tensile force of the steel sheeting. But the profile itself is able to sustain an additional bending moment, which enlarges slip, strains and curvature up to its full plastification.

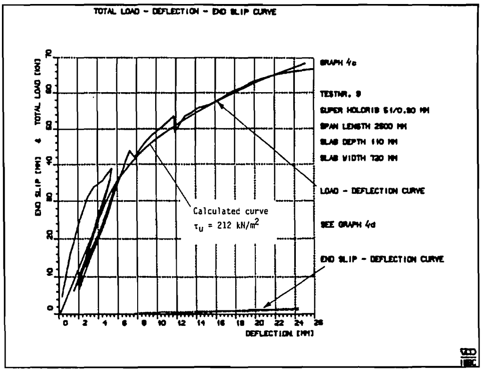
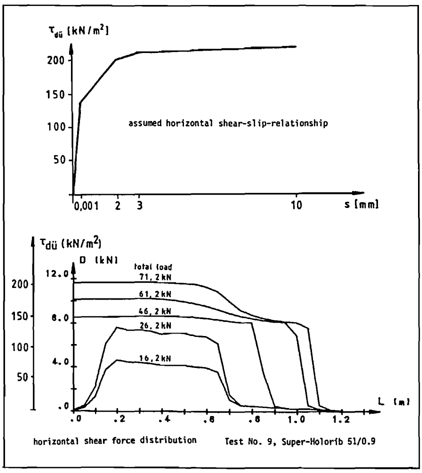


Fig. 5

Figures 4 und 7 demonstrate how to apply partial interaction theory in order to evaluate bending capacity and ultimate loading. With the special degree of shear connection in the considered shear span, i. e. the sum of embossment strengths divided by the plastic tensile force of the profile, one can enter the diagram on the horizontal axis and read the ultimate bending moment from the vertical axis.



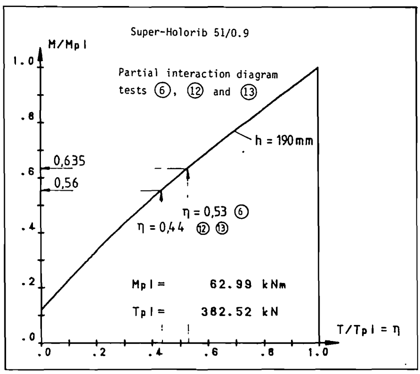


Fig. 6

Fig. 7

Super-Holorib 51/0.9, TNO-tests (Brekelmans), τ_u = 212 kN/m^2 from EDV-calculation (Schanzenbach)

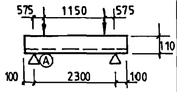
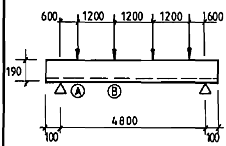
No.		f_y	f_{cu}	$P_{zyl.}$	n	$\frac{M_{cal}}{M_{pl}}$	M_{test}, M_{pl}	$\frac{M_{test}}{M_{cal}}$	failure mode
		N/mm^2	N/mm^2	kN	-	-	kNm	-	
1 2 9		340 347 333 ~ 340	41,1 40,6 37,8 ~ 40	68 70 66 ~ 68	162/333 = 0,49	0,71 (Diagramm)	20,95, 28,43	20,95/20,19 = 1,04	horizontal shear failure, point A
6		316	35,5	70	A 0,53	0,635	26,07, 51,84	0,79	
					B > 1,0	1,0	52,85, 51,84	1,02	flexure
12 13		386	35,5	92,5 90 ~ 91,25	A 0,44	0,56	32,45, 62,99	0,92	
					B > 1,0	1,0	65,60, 62,99	1,04	flexure

Table 1

Table 1 contains results of Super-Holorib composite slab tests and show very good agreement between the tests and this theoretical solution based on partial interaction. Doing this we assume an average value for the horizontal shear strength, without natural or chemical bond at the beginning of loading. From pull-out tests we know, that there is a first peak, which diminishes after a small amount of slip. There could be other uncertainties. But also have in mind from fig. 4 and 7, that the shape of the interaction curve reduces possible deviations from the real shear-strength in evaluating the bending capacities.

We think this partial interaction method is a simple, but usefull alternative to the well-known m and k-method, associated with the names of Ekberg, Porter, Schuster and others. We feel that this method leads to a better understanding of forces and their distribution over the whole length and total cross section with slip at the steel concrete interface.

3. Reduced shear strength of headed studs and composite beam action

Composite beam action in solid slabs as well as in case of profiled steel sheeting is provided generally by means of welded headed studs up to 7/8" shank diameter. It is well known that in North-America and in the UK as in other countries these studs are welded semi-automatically through the steel deck onto the upper flanges of rolled beam sections. Our companies however are afraid of bad weather conditions and too thick paintings and zink coating, which have negative effects on the throughwelding procedure. Therefore the steel profiles have punched out holes, or they are ending besides the stud rows. One advantage is, however, that studs up to 22 mm (7/8") can be used.

In the following we shall concentrate our considerations on Holorib-type steel sheetings. An important advantage of this composite slab type is the fire resistance of at least 90 minutes without considerable additional measures.

The experimental work on headed stud connectors, on which the following considerations are based, has been carried out in the laboratory of structural engineering at Kaiserslautern University /8/. The financial support of the German research foundation DFG is gratefully acknowledged.

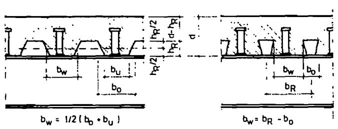
Nearly 60 additional push-out tests have been carried out with two different profiled steel sheetings:

- Holorib 51/150/0.88
- Fischer (Trapezoidal) Fi 60/200/0.88

in order to get better values for stiffness at service conditions, for ultimate strength and deformation capacities.

Up to now, the reduction in strength due to the profiled steel geometry is based on the reduction coefficient originally proposed by Driscoll, Slutter, Fisher /9/ and others, which is used in Germany as well as in other European countries.

Applied to the Holorib-geometry, the reduction factor equals 1, but these additional and other



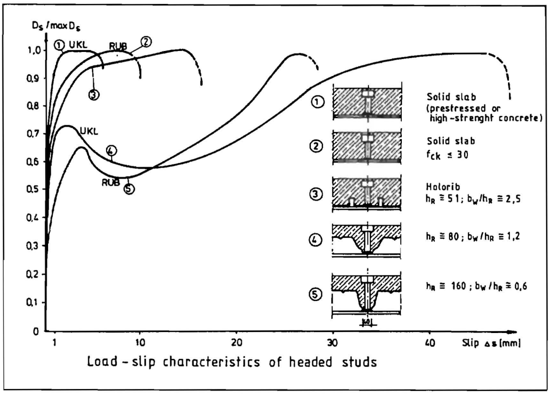
$$\max D_{dü}^{*} = 0{,}6\ \frac{b_w}{h_R}\ \frac{h - h_R}{h} \qquad \max D_{dü} \leq \max D_{dü}$$

Reductionfactor for the nominal strength per stud connector (ribs oriented perpendicular to the beam)

Fig. 8

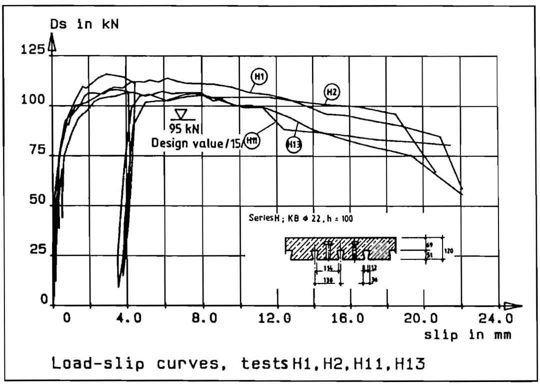
push out tests show, that a smaller reduction is required. We think, the reduction formula has not been thought for Holorib!

The various geometries of available steel sheetings lead to different types of load-slip relationships, as shown in fig. 9. This is independent of wether composite slab action is taken into account or not. But it should be mentioned, that relative displacements of about 15 mm and more are absolutely unimportant in composite beam design. This is likewise valid for the second maximum of certain curves in fig. 9, after the structural mechanisms around the studs have changed.



Load - slip characteristics of headed studs

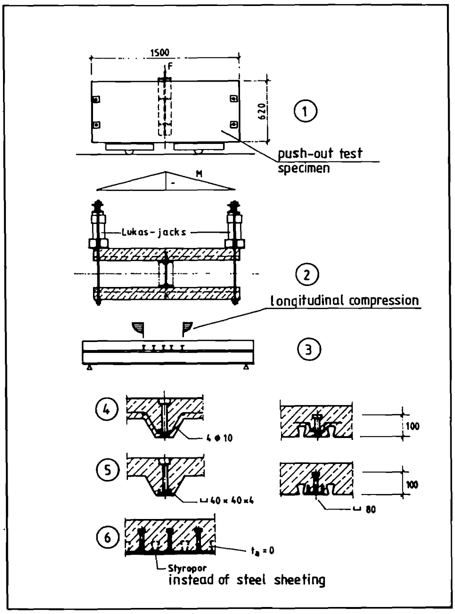
Fig. 9



Load-slip curves, tests H1, H2, H11, H13

Fig. 10

Fig. 10 contains 4 representative results of basic push-out tests. Only the transverse shear reinforcement is slightly different. The mean value for Ds is 110 kN. Compared with a value of 160 kN /10/, in case of solid concrete slabs without ribs, it means really a reduction to about 70 %.



Additional parameters affect the shear strength, see fig. 11:

- transverse (negative) bending: D_s is enlarged to about 110 %, but not reduced, if an adequate transverse reinforcement, which sustains tensile forces due to bending, is provided;

- longitudinal compressive stresses: they magnify the shear strength D_s to 110 % (but at beam ends no compression is available in general);

Fig. 11

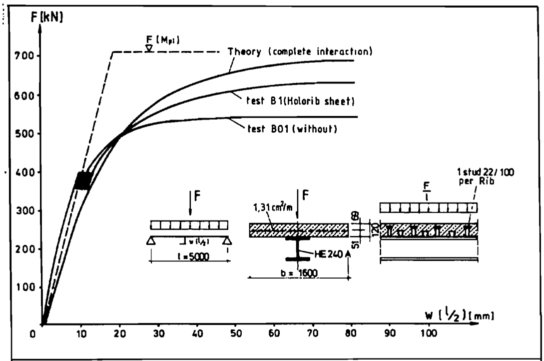
- special locally arranged reinforcement around the stud shaft leads to about 120 % of D_s, if the bottom part of concrete slab would have failed too early;

- more effective are special devices like small channel stubs (130 %), but these both measures have not yet been applied on site.

In addition, if the slab has the same profiled geomtry, but the steel sheeting is missing, this leads to a reduction coefficient of about 2/3.

The deformation capacity is more favourable than in case of solid concrete decks, and this even for high concrete grades; that means, deformation capacities of such connections are always sufficient for equally distributed headed studs and partial interaction design methods.

At Bochum University, 6 composite beams of 5 m length and 7/8"-headed studs with 15 cm equal spacings have been tested /11/, and the strength can be evaluated by means of partial interaction, see fig. 12 and 13 as well as table 2.



Load - deflection curves, beam-tests B1 / B01

Fig. 12

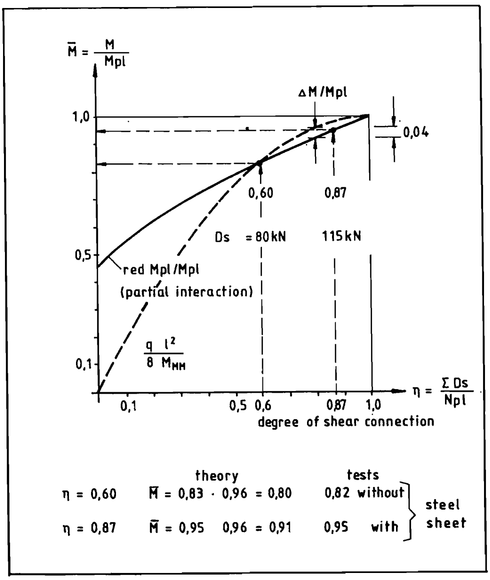


Fig. 13

test No.	transverse reinforcement cm^2/m	$M_{u,test}$ kNm	Mpl kNm	$\frac{M_u}{Mpl}$	average values	steel sheet
BO1	1,3	369	472	0,78		
BO2	5,7	374	457	0,82	0,82	with-
BO3	10,1	396	463	0,86		out
B1	1,3	429	455	0,94		
B2	5,7	459	488	0,90	0,95	with
B3	10,1	478	478	1,00		

Table 2

Test specimen B1, B2 and B3 have Holorib-type decks, specimen BO1, BO2 and BO3 have the same deck geometry, but no steel sheet at all. The favourable influence of the steel profile is obvious. The three tests in each group vary with regard to different transverse bending and accompanying reinforcement.

The average test moment capacities are 0,82 and 0,95 times the fully plastic bending moments, the failure is due to reduced degrees of shear connection, i. e. 60 % and 87 %, respectively. The interaction diagram (fig. 13) provides nearly the same reduced bending capacity, if we enter this diagramm with shear strengths of

- 115 kN in case of Holorib steel sheeting
- 80 kN without any steel sheeting.

Both values take due regard of the beneficial affects of transverse bending.

The problem is, however, how to get design values of the connectors from test curves (for example fig. 10) for design purposes.

Fig. 14 shows a proposal to examine such kind of connector yielding load D_{pl}. This value does not exceed 90 % of the maximum test value and should be reliable even at 10 mm slip at the steel beam-concrete interface.

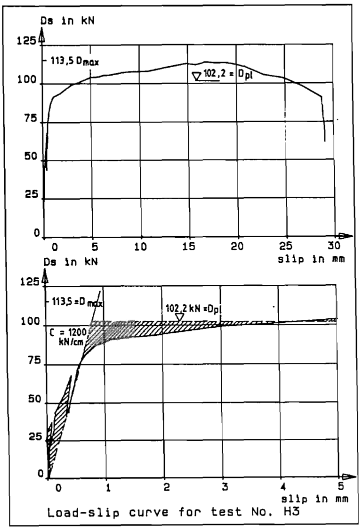


Fig. 14

In order to deduce a new type of reduction formula from tests, the structural engineering model of fig. 15 has been used. It is thought to provide at least the "connector yield load", and we assume the compressed concrete in front of the stud shank to be partly spalled off. The remaining connection region is able to sustain the compression field as well as tensile forces, which appear in a representative truss model (see fig. 15). The steel shank then is stressed in tension and shear (near the stud welding), and the shear-tension interaction in fig. 15 /12/ provides the possibility to calculate the ultimate strength. In case of Holorib, we have an additional shear load capacity as indicated in the figure. If the Holorib steel sheeting is missing, the total capacity is reduced by the above mentioned 30 %.

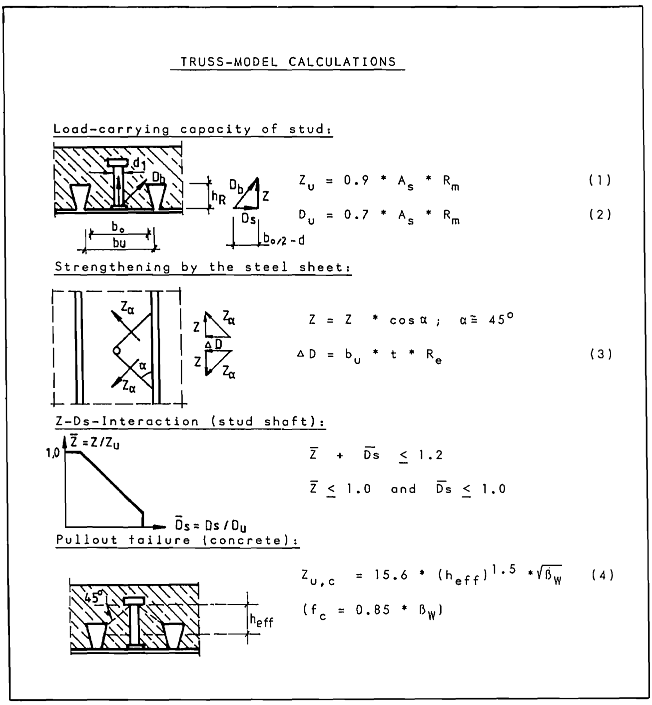
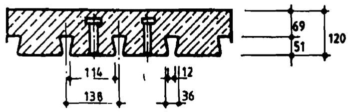


Fig. 15

Example: (solution for 1 stud)



$t = 0.88$ mm
$R_m = 507$ N/mm^2
$\beta_W = 44$ N/mm^2

1) Load-carrying capacity of one stud:

$$\frac{Z}{Ds} = \frac{h_R}{(bo/2 - d_1)} = \frac{51}{(57 - 22)} \Rightarrow Z = 1.46\ Ds$$

Eqs. (1 a. 2): $Z_u = 176.6$ kN

$D_u = 137.4$ kN

2) Z-Ds-Interaction (first step):

$$\frac{1.46\ Ds_1}{176.6} + \frac{1.0\ Ds_1}{137.6} \leq 1.2 \qquad \Rightarrow \underline{Ds_1 = 77.2 \text{ kN}}$$

3) Strengthening by the steel sheet in case of Holorib:

Eq. (3): $\Delta D_a = b_u * t * R_e = 36.4$ kN

4) Iteration (2. and 3. step):

a) $\Delta D + Ds_1 = 1.47\ Ds_1 \Rightarrow \dfrac{1.46\ Ds_2}{176.6} + \dfrac{1.47\ Ds_2}{137.6} \leq 1.2$

$\Rightarrow \underline{Ds_2 = 63.3 \text{ kN}}$

b) $\Delta D_a + Ds_2 = 1.57\ Ds_2 \Rightarrow \ldots$

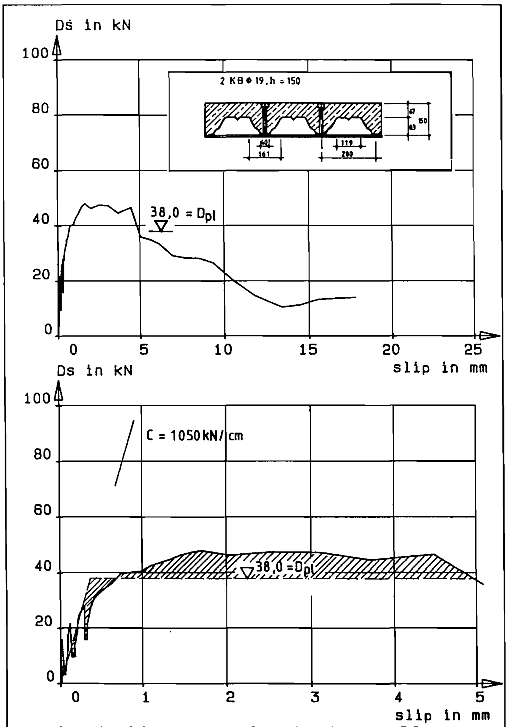
$\Rightarrow \underline{Ds_3 = 60.8 \text{ kN}}$

$$Ds = 36.4 + 60.8 = \boxed{97.2 \text{ kN} = Ds}$$

$$Z_s = 1.46 * 60.8 = \boxed{88.8 \text{ kN} = Z_s}$$

5) Pullout-failure:

Eq. (4): $Z_{u,c} = 15.6 * (h_{eff})^{1,5} * \sqrt{\beta_W} = \underline{88.4 \text{ kN} \leq Z_s}$



Load-slip curve for test No. S8

Fig. 16

This simplified engineering model can even be used to obtain the ultimate shear load in case of more than one stud per rib, see fig. 16. The stud heads are anchored in the concrete part above the ribs to resist tensile forces. But this tensile strength is limited, and the so called 45°-shear cone concept can be applied. If the distances between 2 or even more studs are too small, then the cones are overlapping, thus reducing the pullout strength. Let us assume a linear relationship: if two or more studs are placed on the same point, the pullout strength equals that of one stud. If the distance is equal to or larger than 2 times the effective height, each stud shows its full pullout strength depending on the effective embedment height.

If the pullout strength governs the strength of the connection loaded in shear, the behaviour up to failure is not as ductile as in case of only one headed stud, compare for example fig. 16 and 10.

We think, this method shown in fig. 15 is a nice and useful engineering model even to take into account more than one stud per rib and to establish the required stud lengths and distances. This is an alternative to the proposals recently made by Hosain and others /13/ and /14/.

4. Conclusions

This paper deals with some aspects of the use of profiled steel sheeting for building constructions. Considered are composite beam and slab actions. Regarding strength calculations, simplified engineering models can be established for a better understanding, to reduce the required number of tests and to quantify the influence of certain parameters.

The following conclusions can be drawn:

1. The use of profiled steel sheeting weakens composite beam action, but enhances the deformation capacity significantly:

2. This allows for partial interaction design in case of composite beam action, which is based on a simplified plasticity theory with unlimited large slip at the steel concrete interface;

3. Profiled steel sheeting with embossments or other interlocking measures allow for composite slab action. The strength of such slabs with a ductile failure mode can be evaluated in the same way by means of partial interaction theory. This is a nice and useful alternative to the American m + k-method.

4. The strength of mechanical connectors in case of composite beams is determined by means of push-out tests. Concrete decks on profiled steel sheets reduce the strength of headed studs compared with such connectors in solid slabs.

5. To establish an alternative reduction factor, this paper concerns a simplified or modified truss modell, which leads to design formulae even in the case of more than one stud per rib.

6. In case of composite slabs, we feel that a horizontal shear or bonding strength can be established without pull- or push-out tests. The paper demonstrates, how we are using an accurate computer program including slip at the steel-concrete interface.

7. This contribution is related to ultimate strength considerations and doesn't take due regard of serviceability and other design problems.

Appendix I. Notation

w	Deflection
s	slip (relative displacement at steel-concrete-interface)
M	bending-moment
Mpl	plastic moment
T	horizontal shear-force at steel-concrete-interface
Tpl	horizontal shear-force corresponding to Mpl
$\eta = T/Tpl$	degree of shear-connection
$\tau_{dü}$	shear-stress
τ_u	ultimate shear-stress
ε_a	steel-strain
A_s	cross sectional area of stud
b_o, b_u, b_w, b_R	upper, lower, average and nominal width of concrete rib (see Fig. 8)
h	overal height of stud after welding
h_R	depth of rib
h_{eff}	effective height of a stud in tension
$D_{dü}{}^*$	nominal shear capacity of one stud in a ribbed slab /15/
D_u, Dpl	ultimate and plastic shear capacity of one stud
D_s	shear capacity f(s)
d	total thickness of concrete slab
β_W	concrete strenght ($f_c = 0{,}85 \cdot \beta_w$)
Z_u	tensile capacity of one stud (shank failure)
$Z_{u,c}$	tensile capacity of one stud (concrete failure)
R_m, R_e	mechanical properties of shear connectors and steel sheet

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