Behaviour of steel deck reinforced composite floors

Behaviour of Steel Deck Reinforced Composite Floors

Comportement des planchers mixtes acier-béton

Tragverhalten von Stahl-Beton-Verbunddecken

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SUMMARY

This paper presents a brief overview of previous studies on composite floors, employing profiled sheeting and concrete for use in buildings. Revisions to present design recommendations are proposed based upon results of a recent experimental programme conducted at EPFL. When carrying out performance testing, it is recommended that the post slip behaviour, brittle or ductile, be established. Factors of safety for these two types of behaviour and recommendations for further study are proposed.

RÉSUMÉ

Cette communication présente un aperçu des recherches effectuées à ce jour sur les planchers mixtes composés de tôles profilées recouvertes d'une dalle de béton. On y propose une révision des recommendations de calcul actuellement en vigueur, sur la base des résultats d'un programme expérimental récemment effectué à l'EPFL. On propose en particulier, lors de la réalisation d'essais de charge, de déterminer le caractère du comportement après glissement, ductile ou fragile. Des facteurs de sécurité pour ces deux types de comportement, ainsi que des recommendations pour des études à venir sont finalement proposés.

ZUSAMMENFASSUNG

Der vorliegende Bericht enthält eine Übersicht bisher durchgeführter Forschungsarbeiten auf dem Gebiete von Verbunddecken aus Profilblech mit darüber gegossener Betondecke. Aufgrund der Resultate aus einem an der EPFL durchgeführten experimentellen Forschungsprogramm wird eine Überarbeitung der bestehenden Berechnungsgrundlagen vorgeschlagen. Es wird insbesondere angeregt, bei der Durchführung von Belastungsversuchen zu untersuchen, ob das Tragverhalten nach erfolgter Relativverschiebung zwischen Profilblech und Beton duktil oder spröd ist. Für duktiles und sprödes Tragverhalten werden Sicherheitsfaktoren eingeführt. Schliesslich werden Anregungen für zukünftige Forschungsarbeiten gemacht.

1. REVIEW OF LITERATURE

For many years, engineers and designers have realised the advantages of combining profiled steel sheeting with a concrete slab to form a composite floor system. The first "modern" cold-formed composite floors were produced in the early 1950's and by the mid 1960's several manufacturers were producing a number of successful products. However, each manufacturer had to determine allowable loads through independent research.

The failure mode, most commonly occuring for typical cross sections and span lengths, is the loss of shear bonding between the profiled sheeting and the concrete slab. As a result, several research programmes were conducted to establish a design method for this mode of failure.

The first study was conducted in 1964 by Bryl [1] who proposed a design method based on allowable bending stress in an uncracked composite section, permissible bond stress and the ultimate strength of end anchorages. This method was subsequently recommended by the Swiss Institute of Steel Construction in 1973 [2].

Since 1967, several empirical equations have been proposed which have encompassed a variety of variables. The first extensive testing programme was conducted by Schuster, Porter and Ekberg which led to the development of the first of these equations [3] [4] [5]. All specimens in this test programme consisted of open trapezoidal ribs with embossments. Embossments are small indentations pressed into the webs of the profiled sheeting during the forming process. Of the 209 one way specimens tested only four failed by the flexure mode [6]. The majority failed due to shear-flexure cracking near a concentrated load. This was accompanied by a loss of shear bonding indicated by end slip in the shear span nearest the crack. End slip is defined as the relative movement between the concrete slab and the profiled sheeting at the end of the specimen.

The ultimate shear strength of a given profile is predicted by :

$$\frac{V_u\ s}{b\ d} = \frac{m\ \rho\ d}{\ell_v} + k\sqrt{f_{c,cyl}} \tag{1}$$

V_u : maximum shear at failure obtained from tests [N],

s : centre to centre spacing of shear transfer devices (a constant value over the length of the shear span, equal to unity for embossed sheeting),

b : width of test specimen [mm],

d : effective slab depth (distance from extreme concrete compression fiber to the neutral axis of the full cross section of the profiled sheeting) [mm],

m : slope of the shear bond regression line,

ρ : reinforcement ratio ($\rho = A_p/b\ d$, A_p : cross-sectional area of the profiled sheeting [mm^2]),

ℓ_v : length of shear span, the distance between an applied load and the nearest support or alternatively, the ratio of maximum bending moment to maximum shear force (for a uniformly distributed load use one fourth of the total span length) [mm],

k : ordinate intercept of shear bond regression line,

$f_{c,cyl}$: concrete cylinder strength at time of testing [N/mm^2].

In equation (1), both m and k are constants derived from a regression analysis of the test data. This method of predicting ultimate shear bond capacity was accepted by the European Recommendations for Composite Structures in 1981 [7] and by the British Standards Institution in 1982 [8]. Also, it was adopted by the American Society of Civil Engineers in 1985 [9] and was proposed by the Commission of the European Communities (draft Eurocode 4) in 1984 [10].

Further experimental investigation was conducted by Schuster and Ling who employed both trapezoidal and rectangular profiled sheeting with embossments [11]. They found that some specimens could carry significant load after the initiation of end slip. It was assumed that "mechanical interlock" transfers shear between the slab and the steel sheeting when end slip is present. This lead to a simplification of equation (1) which is valid for both types of behaviour :

$$\frac{V_u}{b\ d} = \frac{m}{\ell_v} + k \tag{2}$$

Equation (2) requires the determination of two constants, m and k, from a linear regression analysis of test data as does equation (1). Concrete strength and the reinforcement ratio have little effect on shear bond capacity. Thus, there is no significant difference between the results of equations (1) and (2) [11] [12]. In the description of the failure mechanisms, Schuster and Ling identified two components of shear bonding : mechanical and frictional. An assumed distribution of each component is proposed along the span both before and after end slip is observed [11].

In addition to the variables identified above, test programmes have been completed to study the effects of : the surface condition of the profiled sheeting [13], repeated loading [6], shoring conditions while the slab is poured [6] [13] and the thickness of the profiled sheeting [11]. Testing is required to determine ultimate shear strength for these and other variables not explicitly included in equations (1) and (2). For a manufacturer producing a range of products, the cost of such testing can be prohibitive. Seleim and Schuster proposed a semi-empirical design equation which included the thickness of the steel decking, t, [11] :

$$\frac{V_u}{b\ d} = \frac{k_1\ t}{\ell_v} + \frac{k_2}{\ell_v} + k_3\ t + k_4 \tag{3}$$

This is beneficial for manufacturers producing profiled sheeting with several different material thicknesses. As for equation (2), equation (3) was derived using specimens which exhibited end slip prior to failure. To use this equation, the values of k_1, k_2, k_3 and k_4 must be established by performing a multi-linear regression analysis of the test data.

Parsannan and Luttrell examined the ultimate load capacity of several different composite cold-formed floor specimens and also proposed a method which reduced the number of tests required [14]. This empirical equation may eliminate the need for testing if the section is similar to the data base from which it was derived. Further, Parsannan and Luttrell observed two types of behaviour after end slip was recorded. These were specimens unable to carry additional load (type 1) and specimens which had the ability to carry further load (type 2). It was observed that type 2 specimens could carry load after end slip was recorded on the second shear span. Finally, three types of shear bonding, chemical, mechanical and frictional, were identified and the relative importance of all three was proposed for both types of behaviour [14].

A comparison of design procedures, including equations (1), (2) and (3), was performed by Evans, Wright and Harding for a number of profiled sheetings produced in Great Britain [15]. Limitations and advantages of different design procedures were examined. The factors, m and k, in equations (1) and (2) were noted to have no physical meaning and thus, they cannot be related to shear bonding. At present, the need for performance testing is unavoidable due to the number of different types and the complexity of typical profiled sheetings. Based upon this review of the literature, the following objectives were established :

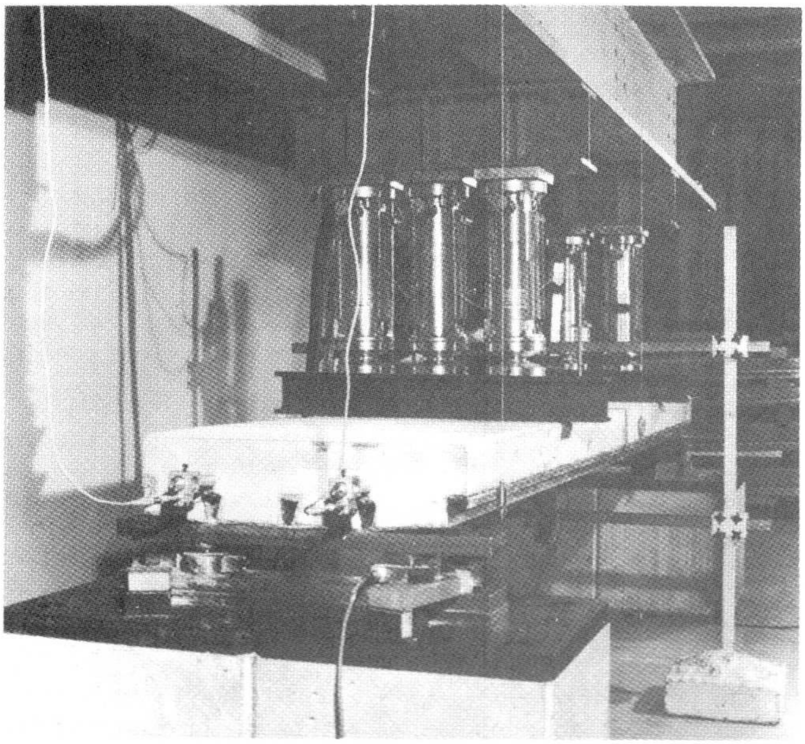
- confirm the load-deflection behaviour of composite floors commonly used in Switzerland,
- examine the present design procedures and propose recommendations,
- identify areas of research.

2 EXPERIMENTAL INVESTIGATION

2.1 Test specimens

Six specimens were constructed and tested to failure. Each test specimen consisted of a single profiled sheeting 3200 mm long and a concrete slab approximately 600 mm wide. These specimens were simply supported during testing; the span length was 3000 mm. Two line loads of equal magnitude, placed symmetrically about the midspan, were applied to each specimen.

Three different types of sheeting were selected according to their availability in Switzerland and for their different methods of providing shear transfer. Although



a) Test frame and instrumentation (specimen HR 51:160).

b) Specimen dimensions.

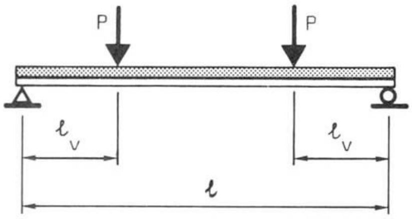


Figure 1 : Test set-up.

Table 1 : Dimensions and material properties.

CROSS-SECTION	SPECIMEN	NOMINAL VALUES [mm]					MEASURED VALUES [N/mm^2]	
		ℓ *)	ℓ_v *)	d	t	b	f_y	$f_{c,cyl}$
d, b, t	MR 58:110	3000	1020	110	1.00	635	313	35
	MR 58:160	3000	780	160	1.00	635	313	35
d, b, t	HR 51:110	3000	1020	110	0.91	624	305	35
	HR 51:160	3000	780	160	0.91	624	305	35
d, b, t	HI 55:110	3000	1020	110	0.88	680	307	35
	HI 55:160	3000	780	160	0.88	680	307	35

*) See Figure 1 b).

several modes of shear transfer have been identified, two are commonly used [4]. These two types were included in this investigation and are :

- embossments pressed into the webs,
- a re-entrant rib geometry.

Thus, the profiled sheetings consisted of open trapezoidal ribs, dovetailed ribs and alternating open trapezoidal and dovetailed ribs. The open trapezoidal sheeting had inclined embossments on the webs of the ribs and all other profiled sheetings were not embossed. The nominal rib depths ranged from 51 mm to 58 mm and nominal material thicknesses ranged from 0.88 mm to 1.0 mm.

A minimum slab thickness of 110 mm was selected to guarantee a minimum concrete cover of 50 mm [7]. A maximum slab thickness of 160 mm was selected since it is typically used in normal construction. Concrete mix details were consistent with current concreting procedures. Thus, a pumping concrete mix was specified. The maximum aggregate size was 30 mm and 325 kg of ordinary portland cement was used per m^3 of mix. A 6 mm diameter 150 x 150 mm wire mesh was placed on metal supports such that a cover depth of 25 mm below the surface of the concrete was maintained during pouring of the slab.

Shoring was placed to support the profiled sheeting at midspan until the concrete had cured. Tensile specimens, cut from the profiled sheeting, and concrete cylinders were tested according to current standards to determine material properties.

A typical test specimen is shown in Figure 1 a) and a sketch illustrating the relative positions of the supports and applied loads is provided in Figure 1 b). A summary of nominal specimen dimensions, load locations and actual material properties is given in Table 1.

2.2 Test procedure

Loads were applied using hydraulic jacks and transmitted to the specimen using 100 mm wide spreader beams. Six electric transducers were used during each test. Two transducers measured midspan deflection. At each end of the specimen, two transducers measured end slip between the concrete slab and the profiled sheeting.

All tests were displacement controlled. A midspan deflection rate of 0.2 mm/min was applied until end slip was first recorded. Thereafter a rate of 0.5 mm/min was used. At a midspan deflection of 30 mm (for the specimens tested $\ell/100$) testing was discontinued. The test set-up, hydraulic jacks and transducers are shown in Figure 1 a). A more detailed review of the test specimens and test procedure is given in [16].

2.3 Test results

The following behaviour was the same for all six specimens tested : upon initial loading, a linear load-midspan deflection curve was recorded. Subsequently, both non-linearity and reduced stiffness was observed. This reduction first occurred when small flexural cracks formed in the lower concrete fibers near midspan in the region of constant bending moment. Cracking was heard sporadically but could not be observed visually as the cracks were concealed by the profiled sheeting. End slip was not recorded at this load level. Such cracking continued to occur until a critical value of horizontal shear stress between the profiled sheeting and the concrete slab was reached. This was indicated by :

- a loud noise,
- an abrupt reduction of applied load,
- the initiation of end slip in the shear span closest to the crack.

The magnitude of initial slip was about 0.3 mm for the embossed specimens and 0.5 mm for the specimens without embossments. Initial end slip is defined as the magnitude of end slip when it is first recorded.

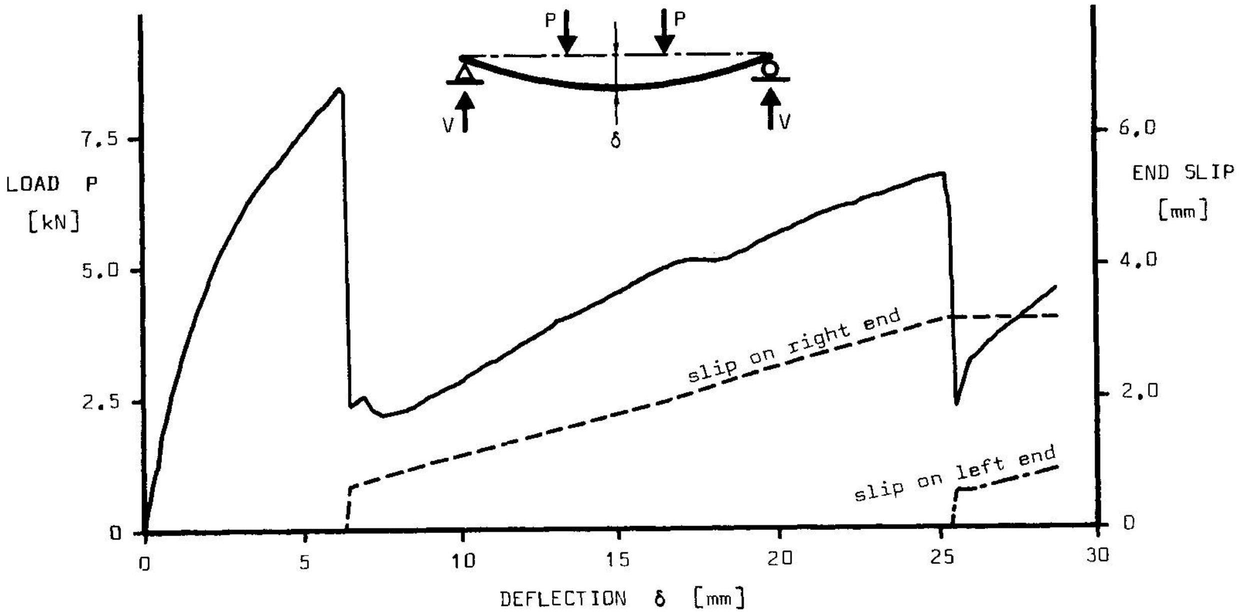


Figure 2 : Brittle behaviour (specimen MR 58:110).

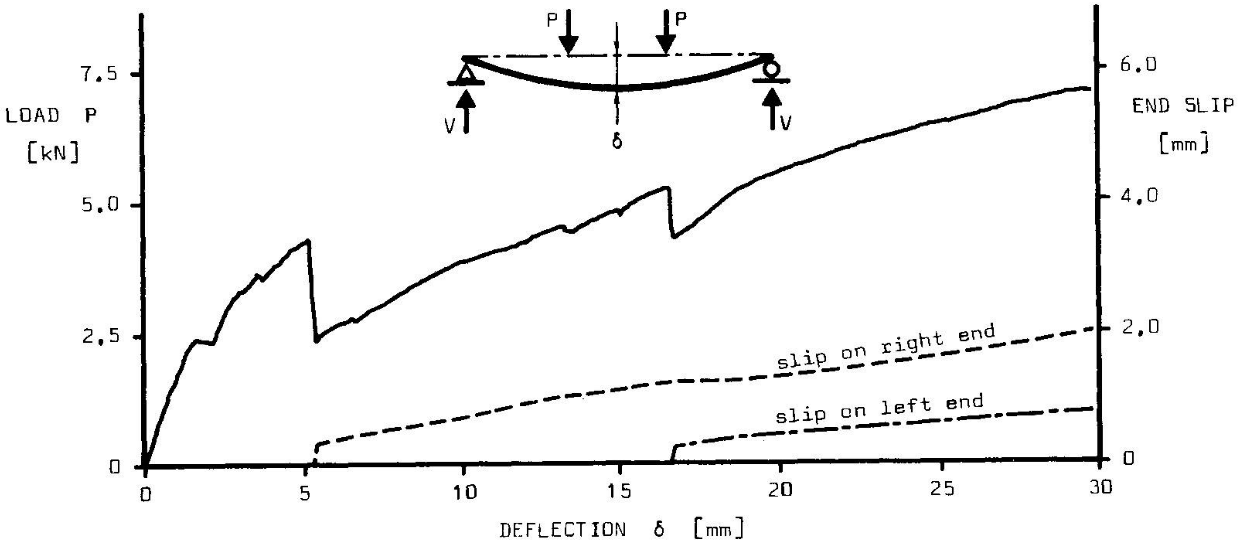


Figure 3 : Ductile behaviour (specimen HI 55:110).

After the initiation of end slip the load-deflection behaviour of the specimens varied widely. Some specimens were able to carry applied loads greater than that which initiated end slip, other specimens did not regain their previous load carrying capacity. Two of the six load-midspan deflection curves from this experimental programme are shown in Figures 2 and 3. These two specimens were chosen as they best illustrate the range of post slip behaviour observed.

3. ANALYSIS OF TEST RESULTS

3.1 Calculation of theoretical limits

Upper and lower bounds of load-deflection behaviour for each specimen was calculated. Firstly, each specimen was analysed assuming the concrete slab was

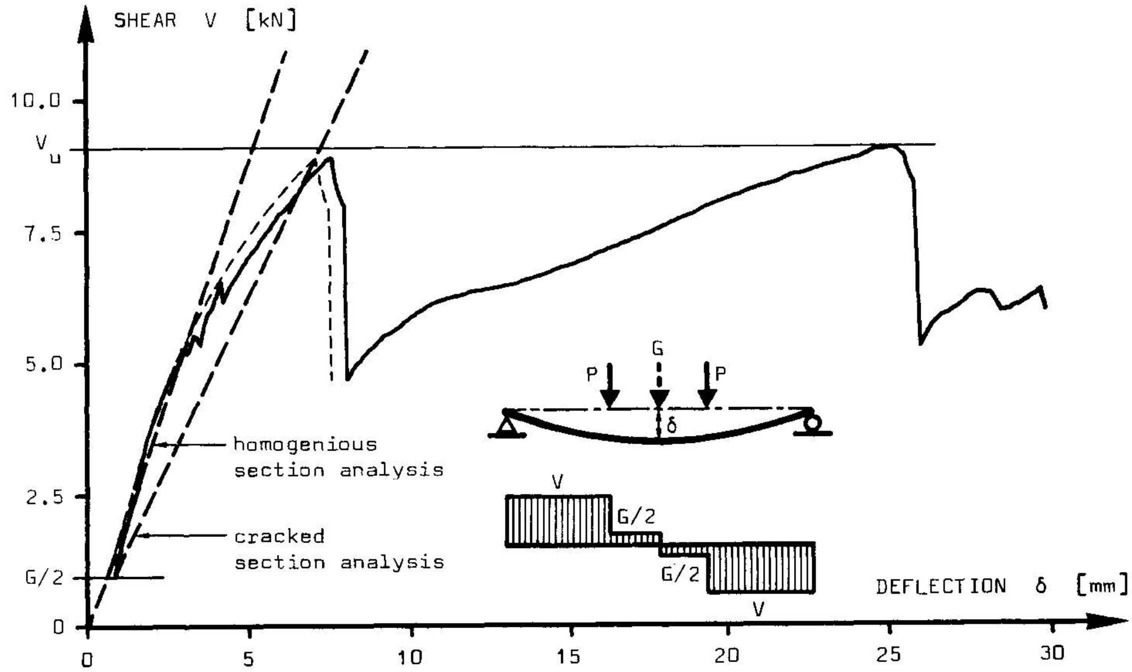


Figure 4 : Homogeneous and cracked section analysis (specimen HR 51:110).

uncracked and secondly, an analysis was performed assuming full composite interaction of the cracked section.

To compare theoretical values and test results, the initial shear and deflection due to self weight must be included. As noted earlier, shoring was inserted at midspan in order to limit the deflection of the profiled sheeting when the concrete slab was poured. After removal of the shoring, the dead weight of the concrete slab can be modelled by a concentrated load at midspan. For the static system used in this test series, the total shear in each shear span is defined as :

$$V = P + \frac{G}{2} \tag{4}$$

V : total shear in each shear span,

P : the load applied by each hydraulic jack (total applied load to the specimen : 2 P),

G : support reaction of the shoring due to the dead weight of the concrete slab.

A typical comparison between test results and these bounds is shown in Figure 4. The homogeneous section analysis closely describes the initial behaviour of the specimen whereas the cracked section analysis best describes behaviour just prior to the initiation of end slip. For all specimens tested, the shear and deflection where end slip initiated is between these two bounds.

3.2 Previous results

Design procedures which employ equation (1) are the most commonly used for classifying the behaviour of composite cold-formed profiled specimens. These procedures all require the experimental determination of ultimate load. A minimum of six test results is standard practice. Most researchers perform two groups of three tests having different span lengths and slab thicknesses. A regression analysis of these results determines the unknown constants, m and k, in equation (1) [3] [6]. These constants enable a prediction which is a straight line when plotted using the axes in Figure 5. Six lines from five studies are presented in Figure 5 [5] [6]

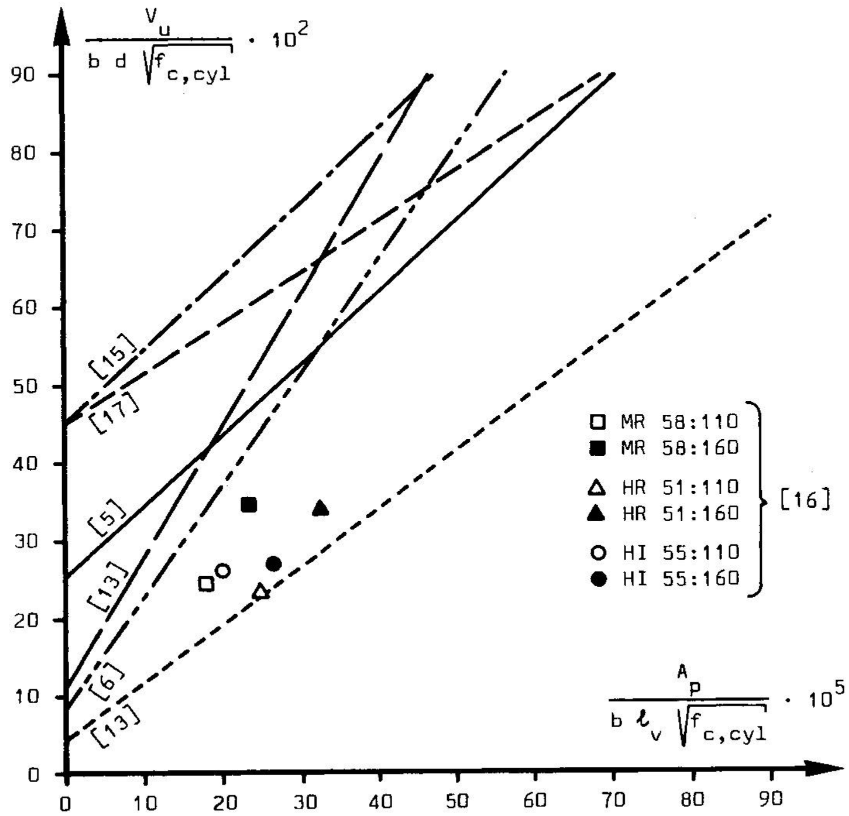


Figure 5 :
Comparison of experimental predictions using eq. (1).

V_u in [N]

$f_{c,cyl}$ in [N/mm^2]

[13] [15] [17]. No systematic agreement between these predictions may be seen, except for the positive slope. Therefore, extrapolation of the lines beyond the supporting test data and application to other profiles cannot be justified.

Test results from the present study are compared with the predictions of Figure 5 using the same parameters. These results are enclosed by the upper and lower predictions.

3.3 Classification of behaviour

The ultimate shear strength predicted using equation (1) is based upon the average values of ultimate load observed during testing. This method does not take into account other parameters. For example, load capacity after end slip is recorded may vary, see Figures 2 and 3. These two types of behaviour, named type 1 and type 2 by Luttrell [14], are designated "brittle" and "ductile" in this report. This terminology was also used by Aribert and Moum [18].

Brittle behaviour (type 1) is accompanied by the initiation of end slip when the ultimate load is recorded. Ductile behaviour (type 2) occurs when the ultimate load subsequently exceeds the load which initiated end slip, and when midspan deflection is several times greater than observed at first slip.

It is useful to introduce two ratios, P_{max}/P_{slip} and $\delta_{max}/\delta_{slip}$ to quantify the "reserve ductility". Limiting ratios can be used to define ductile and brittle behaviour.

P_{max} : the ultimate load observed before a midspan deflection of 30 mm is recorded,

δ_{max} : the midspan deflection corresponding to P_{max},

P_{slip} : the load applied to the test specimen when initial end slip is observed,

δ_{slip} : the midspan deflection corresponding to P_{slip}.

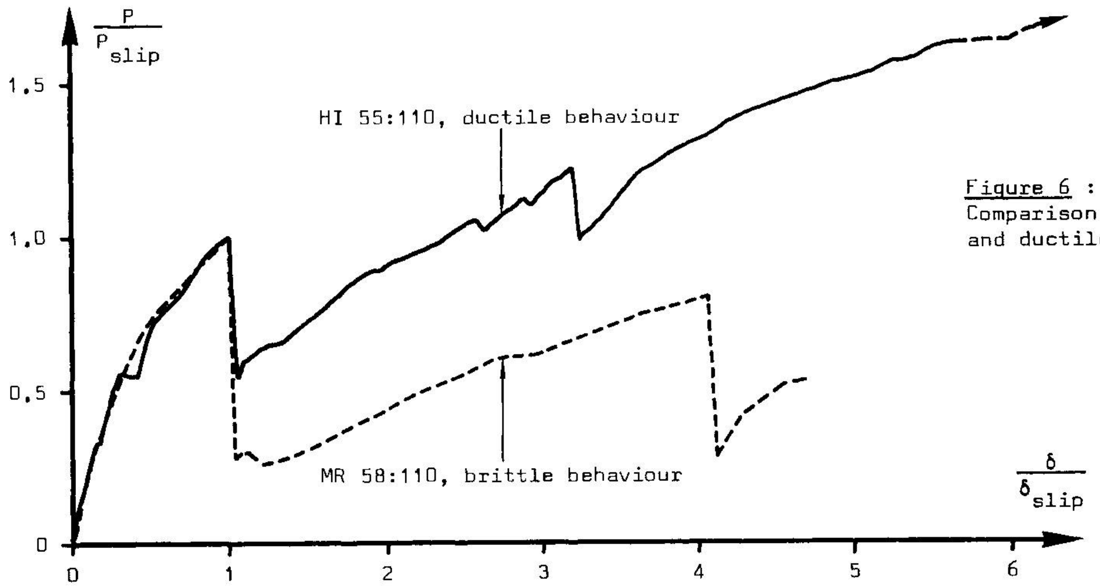


Figure 6 : Comparison of brittle and ductile behaviour.

Normalized load-deflection curves for brittle and ductile specimens are presented in Figure 6. They illustrate the differences in post slip behaviour between brittle and ductile behaviour. The ultimate load in the ductile specimen is 1.63 times greater than the applied load at first slip. The midspan deflection at ultimate load is 5.7 times larger than the deflection at first slip. This agrees with the results of Schuster and Ling who observed that the applied load at first slip may be between 50 and 60 % of the ultimate load [11]. Thus, the following ratios are proposed to define ductile behaviour :

$$\frac{P_{max}}{P_{slip}} > 1.5 \qquad \frac{\delta_{max}}{\delta_{slip}} > 4.0$$

If either ratio is less than the limits given above, the behaviour is termed brittle.

4. SAFETY CONSIDERATIONS

Safety factors consist of several parts. These may be divided into two partial safety factors. One accounts for uncertainties in the strength of a component and the other accounts for variations in loading. The strength partial safety factor consists of three parts for composite profiled floors. These are :

- mode of failure,
- scatter of test results,
- warning prior to, and consequence of, failure.

Horizontal shear debonding is the dominant mode of failure. For this type of failure a partial safety factor of 1.25 is commonly accepted [10]. This partial safety factor remains the same for both brittle and ductile behaviour. Scatter between test results is treated differently by code writing organizations. A factor of 1.15 is used in this study assuming that a minimum of six tests are performed and that the test values do not vary from the regression line by more than 10 % [10]. The third part of the strength safety factor cannot be determined using statistics alone. In accordance with other structural elements, such as reinforced concrete beams, this factor was established as 1.0 for ductile behaviour and 1.25 for brittle behaviour. This corresponds to the factors of safety used in under-reinforced and over-reinforced concrete beams.

Usually, the partial safety factor for loading is taken to be 1.4 for floor systems [19]. The following overall factors of safety are then obtained :

Brittle composite floors : $\gamma_m = 1.25 \cdot 1.15 \cdot 1.25 = 1.8$

$\gamma_f = 1.4$

$\gamma = \gamma_m \gamma_f = 1.80 \cdot 1.4 = 2.5$

Ductile composite floors : $\gamma_m = 1.25 \cdot 1.15 \cdot 1.0 = 1.44$

$\gamma_f = 1.4$

$\gamma = \gamma_m \gamma_f = 1.44 \cdot 1.4 = 2.0$

γ : overall safety factor,
γ_m : partial safety factor for material,
γ_f : partial safety factor for loading.

The overall safety factor proposed by the draft Eurocode 4 is 2.0 regardless of behaviour [7]. This is the same as the factor of safety proposed for ductile specimens.

5. CONCLUSIONS

The following conclusions are limited to cold-formed composite floors which fail by loss of horizontal shear bonding through the occurrence of end slip. In addition, they apply to profiled sheetings using either embossments pressed into the webs or re-entrant web profiles.

1.- Two types of load-deflection behaviour exist, brittle and ductile.

2.- Brittle and ductile behaviour is distinguished using the two ratios P_{max}/P_{slip} and $\delta_{max}/\delta_{slip}$. The ratios defining ductile behaviour are :

$$\frac{P_{max}}{P_{slip}} > 1.5 \qquad \frac{\delta_{max}}{\delta_{slip}} > 4.0$$

If either one of these ratios is smaller than the values given above, the behaviour is termed brittle.

3.- Factors of safety which are dependent upon load-deflection behaviour can be adopted by present design methods. These factors are proposed to be :

$\gamma = 2.5$ for brittle behaviour

$\gamma = 2.0$ for ductile behaviour

4.- The ultimate shear strength of profiles which are not yet included in the draft Eurocode 4 design procedure (those relying upon profile shape alone to transfer shear bonding) can be included if these safety factors are employed.

5.- When equations (1) and (2) are used to predict the shear strength of a specimen, the results must be used with caution. Variables not expressly included in the equation can cause large variations in behaviour.

6. RECOMMENDATIONS

Presently, the behaviour of cold-formed profiled floors is not adequately described by design formulations. The following recommendations are made :

1.- Effect of first end slip on ultimate shear strength. Present design methods define ultimate load but do not take into account differences in behaviour. A more rational design basis which recognizes the differences between brittle and ductile behaviour should be adopted. This change will enable current design methods, for example Eurocode 4, to safety predict the ultimate shear strength of a wider variety of specimens.

2.- End anchorage. Composite cold-formed floors constructed with end anchorages, tied to the supporting steel frame, may restrict end slip. These end anchorages can take several forms :

- shear studs welded through the profiled sheeting to the supporting beams,
- cold-formed shear connectors with fasteners driven through the profiled sheeting,
- hot-rolled end angles or studs welded directly to the supporting beams.

Further research is needed in order to examine the degree of horizontal shear transfer accomplished by end restraints. Specimens which exhibit brittle behaviour may become ductile if sufficient shear capacity is added using end anchorages.

3.- Other variables affecting strength. The following variables influence the strength of cold-formed composite floors and consequently they should be considered in design formulations :

- continuous span composite floors,
- the presence of steel reinforcement,
- propping the deck prior to pouring the slab,
- surface condition of the profiled sheeting,
- repeated loadings,
- end anchorages,
- etc.

Steel deck reinforced composite construction may have applications in structures such as car parks and more recently also in bridges. This presents additional challenges since the effects of weathering and repeated loadings require additional examination.

ACKNOWLEDGMENTS

The first author wishes to express his appreciation to the staff at ICOM who contributed to the preparation of this report, in particular M.J. Fidler who conducted the tests and B.J. Daniels for his assistance in preparing the text. Partial funding for this project was supplied by grants from several manufacturers of cold-formed sheeting.

REFERENCES

[1] BRYL, S. The composite Effect of Profiled Steel Plate and Concrete in Deck Slabs. Acier-Stahl-Steel, Brussels, vol. 32, no 10, 1967, pp. 453-459.

[2] BADOUX, J.-C., CRISINEL, M. Recommandations pour l'utilisation de tôles profilées dans les planchers mixtes du bâtiment. Publication B5, Zurich, Centre suisse de la construction métallique, 1973.

[3] SCHUSTER, R.M. Composite Steel-Deck-Reinforced Concrete Systems Failing in Shear-Bond. 9th Congress, Preliminary Report, Zurich, IABSE, 1972, pp. 185-191.

[4] PORTER, M.L., ECKBERG, C.E. Design Recommendations for Steel Deck Floor Slabs. University of Missouri-Rolla, Third International Specialty Conference on Cold-Formed Steel Structures, vol. II, 1975, pp. 761-791.

[5] PORTER, M. L., EKBERG, C. E., GREIMANN, L. F., ELLEBY, H.A. Shear-Bond Analysis of Steel-Deck-Reinforced Slabs. Journal of the Structural Division, New York, vol. 102, no 12, 1976, pp. 2255-2268.

[6] PORTER, M. L., EKBERG, C. E. Compendium of ISU Research on Cold-Formed Steel-Deck-Reinforced Slab Systems. Bulletin 200-78263, Iowa State University, Engineering Research Institute, 1978.

[7] European Convention For Constructional Steelwork (ECCS). Composite Structures. London, The Construction Press, 1981.

[8] Structural Use of Steelwork in Building, Part 4 : Code of practice for design of floors with profiled sheeting. British Standard BS 5950, London, British Standard Institution, 1982.

[9] Specifications for the Design and Construction of Composite Slabs and Commentary on Specifications for the Design and Construction of Composite Slabs. Technical Council of Codes and Standards Division, New York, American Society of Civil Engineers, 1985.

[10] Composite Steel and Concrete Structures, First Draft. Eurocode 4, Brussels, The Commission of the Europeans Communities, 1984.

[11] SCHUSTER, R. M., LING, W. C. Mechanical Interlocking Capacity of Composite Slabs. University of Missouri-Rolla, Fifth International Specialty Conference on Cold-Formed Steel Structures, 1980, pp. 387-405.

[12] SELEIM, S. S., SCHUSTER, R. M. Shear-Bond Capacity of Composite Slabs. University of Missouri-Rolla, Sixth International Specialty Conference on Cold-Formed Steel Structures, 1982, pp. 511-531.

[13] PORTER, M. L., EKBERG, C. E. Coating Effects of Cold-Formed Steel Deck Slabs. University of Missouri-Rolla, Fifth International Specialty Conference on Cold-Formed Steel Structures, 1980, pp. 369-386.

[14] PRASANNAN, S., LUTTRELL, L. D. Flexural Strength Formulations for Steel-Deck Composite Slabs. Morgantown, West Virginia University, 1984.

[15] EVANS, H. R., WRIGHT, H. D., HARDING, P. W. Composite Floors : Comparisons of Performance Testing and Methods of Analysis. IABSE Reports, vol. 48, Zurich, 1985, pp. 219-225.

[16] CRISINEL, M., FIDLER, M. J., DANIELS, B. J. Flexure Tests on Composite Floors with Steel Sheeting. Publication ICOM 158, Ecole Polytechnique Fédérale de Lausanne, 1986.

[17] LONG HUNG, H., FULOP, A., MOUM, C. Planchers à bacs collaborants, recherche expérimentale. Annales de l'Institut Technique du Bâtiment et des Travaux Publics, Paris, no 363, 1978, pp. 86-105.

[18] ARIBERT, J.-M., MOUM, C. Efficacité de la connexion dans les planchers mixtes de bâtiment. IABSE Reports, vol. 48, Zurich, 1985, pp. 227-236.

[19] Norme SIA 161 : Constructions métalliques. Zurich, Société suisse des ingénieurs et des architectes, 1979.