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Tests on composite slabs and evaluation of relevant Eurocode 4 provisions

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Abstract. The paper addresses some key issues related to the design of composite slabs with cold-formed profiled steel sheets. An experimental programme is first presented, involving six composite slab specimens tested with a view to evaluating Eurocode 4 (EC4) provisions on testing of composite slabs. In four specimens, the EC4-prescribed 5000 load cycles were applied using different load ranges resulting from alternative interpretations of the reference load W_t. Although the rationale of the application of cyclic loading is to induce loss of chemical bond between the concrete plate and the steel sheet, no such loss was noted in the tests for either interpretation of the range of load cycles. Using the recorded response of the specimens the values of factors *m* and *k* (related to interface shear transfer in the composite slab) were determined for the specific steel sheet used in the tests, on the basis of three alternative interpretations of the related EC4 provisions. The test results confirmed the need for a more unambiguous description of the *m-k* test and its interpretation in a future edition of the Code, as well as for an increase in the load amplitude range to be used in the cyclic loading tests, to make sure that the intended loss of bond between the concrete slab and the steel sheet is actually reached. The study also included the development of a special-purpose software that facilitates design of composite slabs; a parametric investigation of the importance of *m-k* values in slab design is presented in the last part of the paper.

Keywords: composite slabs; profiled steel sheets; *m-k* factors; longitudinal shear; load cycles; Eurocode 4; chemical bond loss; design software.

1. Introduction

The use of cold-formed profiled steel sheets for the construction of composite slabs is an increasingly popular technique in the construction sector in Europe. A key aspect of the design of composite slabs according to the pertinent European code, Eurocode 4 (CEN 2004) is the determination of factors "*m*" and "*k*" used to estimate the longitudinal shear strength of the composite slab. These factors are unique for each cold-formed profiled steel sheet and have to be determined experimentally according to the procedure described in Annex B of EC4. The work reported herein includes the determination of these factors for a specific type of steel sheet and focuses on the evaluation of EC4 provisions in the light of the measured response of the test specimens.

Due to its relevance to design, this longitudinal shear test has been carried out in the past by several

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researchers. Marimuthu *et al.* (2007) tested 18 composite slab specimens, the main test parameter being the shear span. It was found that if the shear ratio is high enough, but not higher than about 12, the behaviour of the slab is governed by flexure, while for shear ratios lower than about 4, strength of the slab is governed by shear bond failure. They also found that by applying the loading cycles prescribed by EC4 the strength of the slab is not reduced with respect to the strength of the monotonically loaded specimens. The longitudinal shear resistance of a steel-concrete composite bridge deck was examined by Jeong *et al.* (2007), who found that to define the longitudinal shear resistance of such composite decks subjected to surface loads, it is desirable to use test data with sufficient shear span length, so that the effect of the normal force at the interface becomes negligible. A simplified method for estimating the shear strength of composite slabs was proposed by Lopes and Simoes (2008) on the basis of their experimental and analytical results; they state that the proposed method yields more reliable results than the EC4 method, but note that further experimental verification is necessary. Calixto *et al.* (2009) compared the *m-k* method and the Partial Connection method (allowed as an alternative by EC4) in the light of experimental results. Good correlation between the two methods was found, which is not in line with the finding of Marimuthu *et al.* (2007) that the two methods provide results differing (on the average) by 26%. Tzaros *et al.* (2010) tested four composite slab specimens and determined the *m* and *k* factors from the line resulting from linear interpolation between the points defined by the measured specimen strength, taken as the one corresponding to longitudinal shear failure (brittle loss of chemical bond between concrete and the steel sheet); they found that the *m*-factor is governed by the tensile stress of the steel sheet along the length between the load point and the middle point of the specimen. The experimental results were well reproduced by the rather sophisticated analytical model developed by these authors, which is not intended for design purposes. A similar test programme involving 9 composite slab specimens was carried out by Tsalkatidis and Avdelas (2010), who used a 3D numerical model set up in a commercial program (ANSYS), to predict the test results, and found good agreement in terms of load vs. deflection diagrams.

In the framework of the study presented herein six composite slab specimens were tested. The main parameters studied were the span (hence the shear ratio), the upper and lower load level applied during the cyclic loading, and the estimation of *m* and *k* factors using alternative interpretations of the pertinent EC4 provisions. The need for a more rational definition of some of the parameters prescribed by the Code was made clear by the findings of this study, and is discussed in Section 5. The paper concludes with a brief presentation of a special-purpose software developed to facilitate design of composite slabs, which is used for a parametric investigation of the importance of *m-k* values in slab design.

2. Research significance

The tests required for the determination of design values for *m* and *k* factors are described in detail in Annex B of EC4. However, some of the code provisions are open to different interpretations. In paragraph B.3.4(3) the load W_t is defined as the measured failure load of the preliminary static test. In paragraph B.3.4(5) the same term W_t is defined as the “failure load”, to be calculated as the maximum load imposed on the slab at failure plus the weight of the composite slab and spreader beams. Intuitively, the second definition is more rational since self-weight of the specimen and the load of the spreader beams result in additional longitudinal shear. For the specimens tested herein, the weight of the spreader beams when added to the self-weight of the slab gives a force higher than $0.2W_t$ which is the lower level applied during the 5000 loading cycles. For this reason two different amplitude levels

$0.2W_t$ and $0.6W_t$ were used in the tests, as described in Section 3.

Another point of interest is whether the upper level of cyclic loading ($0.6W_t$) is sufficient for inducing loss of bond between concrete and the steel sheet. Since the Eurocodes do not include a commentary and no relevant background documents were found by the authors, it is not known whether the intention of the EC4 provisions is to verify that concrete and the steel sheet can sustain 5000 load cycles without loss of bond, or, on the contrary, the applied cyclic loading is meant to induce loss of bond and the purpose of the subsequent monotonic loading test is to estimate the capacity of the composite slab after loss of chemical bond.

A third point open to interpretation is the definition of 'failure load' from the load - deflection diagram determined from the tests (discussed in Section 5 of the paper). Hence, three alternative interpretations of the loading to be applied were made in the present experimental study:

- Use of the load corresponding to maximum strength, developed just prior to loss of chemical bond.
- Use of 80% of the aforementioned value whenever the failure load does not exceed by at least 10% the load that corresponds to slip equal to 0.1 mm between the steel sheet and concrete; in this case the response is not considered by EC4 as ductile. In the light of the relevant literature and the present tests, the authors believe that this case deserves further consideration.
- Use of the ultimate (post-peak) strength, related to friction mechanisms developing after loss of chemical bond, and manifested by local buckling of the steel sheet.

For comparison purposes, for the tested specimens all three cases were considered, and *m* and *k* values were calculated separately for each case.

3. Test description

The specimens (Fig. 1) were designed and manufactured (Figs. 2 and 3) mainly with a view to

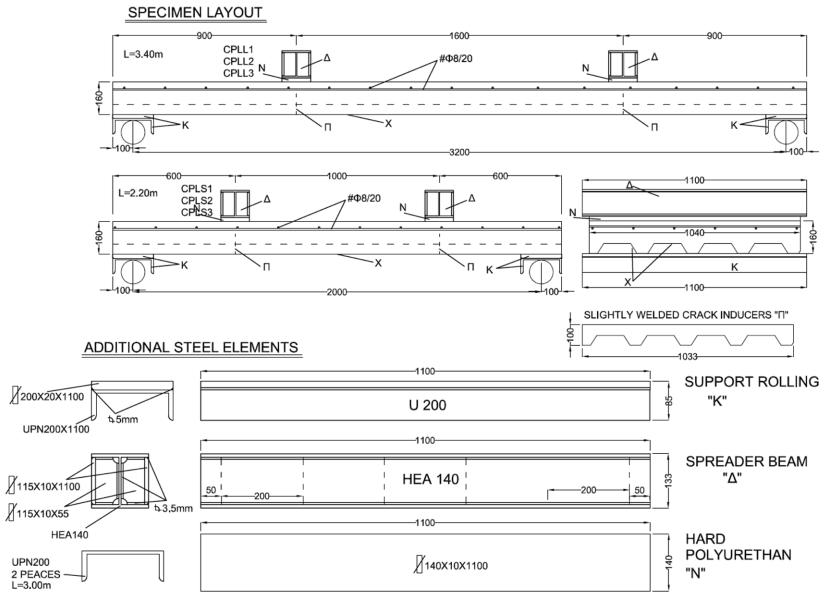


Fig. 1 Geometry and reinforcement of the specimens

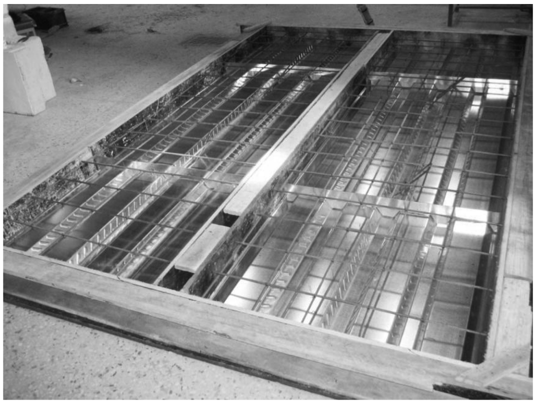


Fig. 2 Specimens CPLL ready for concreting

Fig. 3 Concreting and curing of the specimens

determining the m and k factors, which have to be defined through testing for every newly produced steel sheet, meant to be used in composite slabs, for the design against longitudinal shear. This check is typically the most critical for the design of composite slabs; hence EC4 describes in detail the entire procedure (in section 9.7.3 and in Annex B, paragraph B.3). The experimental setup with the specimen ready for testing is shown in Figs. 4 and 5.

The concrete used had a mean compressive (28-day cylinder) strength of 24.7 MPa, with a standard deviation of 1.8 MPa, resulting in a characteristic (5% fractile) value of 21.7 MPa, which for all practical purposes can be assigned to EC2 class C20/25. The reinforcement placed on the upper side of the slab (mainly for shrinkage cracking control) was a single grid of 8 mm bars spaced at 200 mm. The nominal yield stress of this reinforcement was 500 MPa and the maximum available strain (ε_{su}) was greater than 7.5%. The proprietary steel sheet used in the composite slab was 0.75 mm thick; its cross section is shown in Fig. 1 (bottom-right). The yield stress of the sheet was 325 MPa with maximum stress 379 MPa at 20.0% strain Fig. 6. Six specimens, in two groups of three, were manufactured; each group was characterised by the length (hence the shear span) of the specimen. The span was 3.20 m for group CPLL (Fig. 4) and 2.00 m for group CPLS (Fig. 5); the corresponding shear spans are 10.6 and 4.2. Crack inducers were placed at the loading point of all specimens, as prescribed in section B3.3 of EN1994-1-1 (CEN 2004). One specimen in each group was loaded (with displacement control) monotonically to failure. The other two specimens were subjected to 5000 load-controlled cycles

Fig. 4 Experimental set-up with specimen CPLL in place

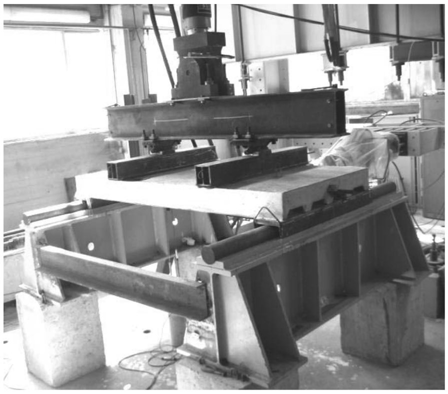


Fig. 5 Specimen CPLS ready for testing

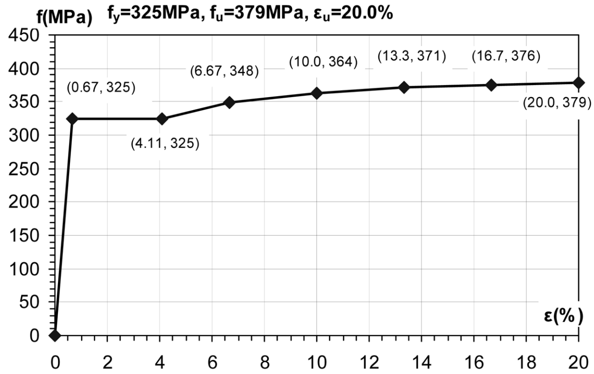


Fig. 6 Stress-strain diagram of steel sheets

within predefined limits $0.2W_t$ and $0.6W_t$, where W_t is the failure load measured during the test of the first specimen. These specimens were then subjected to displacement-controlled loading to failure. It is pointed out that if W_t is defined as the total ultimate load of the slab (i.e., including the weight of the composite slab and spreader beams, see B.3.4 of EC4), then the $0.6W_t$ limit is clearly defined. However, in this case there is a issue with the definition of the lower limit $0.2W_t$, since this load was found to be less than the weight of the slab and the spreader beams used, hence a tensile force would be needed to materialise this lower limit, which is unrealistic and most probably is not the actual intention of the Code. Since the authors did not find any justification notes regarding this particular clause of EC4, the following loading programme was finally implemented: One specimen was subjected to 5000 load cycles with an upper limit of $0.6W_t$ as defined previously (total strength of the specimen plus the self weight of the spreader beams and the slab), and a lower limit of 1/3 the net load (i.e., the load applied through the hydraulic jack) of the upper limit, hence maintaining the max/min ratio envisaged by the Code for the net forces only. Using this interpretation the 5000 cycles applied to specimen CPLL2 corresponded to an upper limit of 9 kN and a lower limit of 3 kN; taking into account the self-weight and the weight of spreader beams (14.95 kN) the corresponding total loads W_t were 23.95 kN and 17.95 kN, respectively. During this test no loss of chemical bond between the steel sheet and the concrete slab was observed. For this reason it was decided to explore in subsequent tests another, more

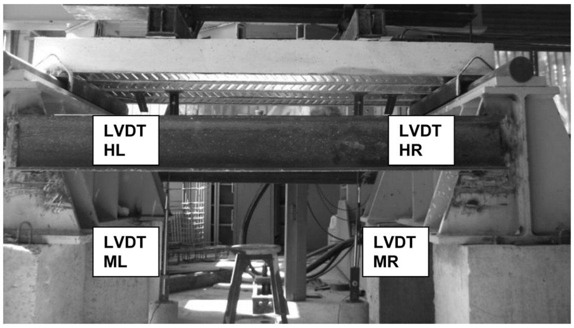


Fig. 7 Layout of LVDTs

conservative, interpretation, increasing the load limits for specimens CPLL3, CPLS2 and CPLS3. The applied load cycles now had limits $0.2P_{ult}$ and $0.6P_{ult}$ where P_{ult} was the measured ('net') failure load recorded during the test of specimens CPLL1 and CPLS1. This way, the applied loads varied between 4 kN and 12 kN for specimen CPLL3, with corresponding total loads 18.95 kN to 26.95 kN when self-weights are taken into account. The value 20kN is the failure load that was recorded during the test of specimen CPLL1. During the test of specimen CPLS1 the recorded failure load was 42.33 kN, with self weight 10.63 kN. The applied loads varied from 8.46 kN to 25.40 kN for specimens CPLS2 and CPLS3 (total values from 19.09 kN to 36.03 kN). It was found that although the applied loads (5000 cycles) were increased, no loss of chemical bond at the interface between the steel sheets and the concrete topping was detected in the specimens. Chemical bond was lost when the specimens were loaded monotonically, subsequent to the cyclic loading sequence.

The composite slabs were instrumented with four displacement transducers (LVDTs), whose layout is shown in Fig. 7. These were used to measure the deflection in the span of the specimen at the loading points, Middle Left (ML) - Middle Right (MR), and another two transducers, Horizontal Left (HL), Horizontal Right (HR), were used to measure the sliding of the steel sheet relative to the concrete topping at the lower face of the specimen. The diagrams given in the next section plot the displacement measured by these LVDTs and are named as δ_{ML}, δ_{MR}, δ_{HL}, δ_{RL} respectively. The data "P_{tot}" plotted on the vertical axis is the *total* load recorded during the tests, that includes the load of spreader beams and the self weight of the specimens.

4. Specimen response and test results

Fig. 8 illustrates the specimen CPLL1 during the test. The crack at the left loading point is also shown enlarged in the same figure. Fig. 9 depicts the slip between the steel sheet and the lower concrete surface at the left end of this specimen. Also in this figure a part of LVDT "HL" is shown. This specimen was loaded by monotonically increasing imposed displacements. It is noted that while there is significant capacity of the concrete topping in compression and of the steel sheet in tension, the parameter defining the resistance of the composite slab is the transfer mechanism between these two materials. Prior to debonding the interface shear stresses are resisted primarily by chemical bond. During the test of specimen CPLL1, for a deflection between 2 and 3 mm, the chemical bond between the steel sheet and the concrete topping was lost and a flexural crack formed under the left loading point (Fig. 8). At this point the strength of the specimen was reduced by about 50% and the deflection at the

Fig. 8 Specimen CPLL1 during testing

Fig. 9 End slip of specimen CPLL1

left loading point was increased to 5 mm. As the imposed displacement was increased the strength of the specimen started to pick up again but with significantly lower stiffness. While prior to debonding the steel sheet acted as tension reinforcement, the load resisting mechanism activated after loss of bond included the flexural strength of the (bare) steel sheet and friction between concrete and the steel sheet, more specifically mechanical interlock at the embossments and friction at the remaining surface of the sheet. It must be noted that the flexural strength of the steel sheet is increased, due to the presence of concrete at its top, which offers restraint against local buckling phenomena in the steel sheet; such buckling appeared (Fig. 10) for a higher imposed displacement than that which would appear if the sheet was loaded as an (unrestraint) simply-supported beam. The post-debonding mechanism proved to be quite effective and the strength of the specimen picked up, reaching maximum values at imposed displacements between 20 and 25 mm (Fig. 11), which are about 10 times those at debonding.

Despite the nominal symmetry of the test set-up, only in two specimens the main flexural cracks formed at both loading points; in all others only one major crack formed, either on the left or on the right side, and most of the inelastic deformation concentrated therein. For some specimens, additional flexural cracks appeared close to the loading point where the main crack formed. For high inelastic deformations local buckling occurred at the compression side of the steel sheet, under the points of flexural cracks in concrete (Fig. 10).

In Figs. 11 and 12, load - deflection curves are shown for the high (10.6) and low (4.2) shear ratio specimens, respectively; each diagram is plotted for the side of the specimen where failure occurred. It is clear from the diagrams that the resistance due to friction mechanisms (reached at deflections of 20~

Fig. 10 Buckling at the compression side of the steel sheet for tested specimen

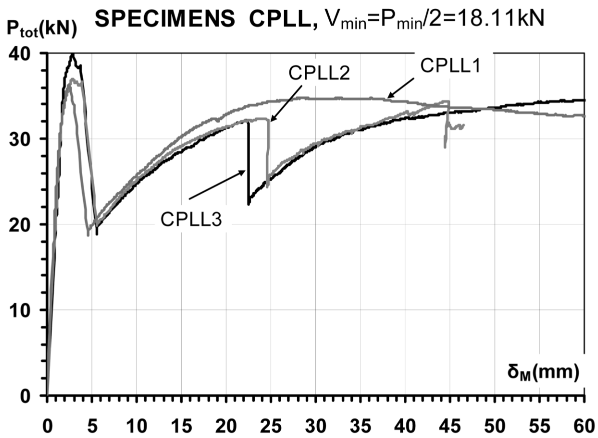


Fig. 11 Comparative load - deflection diagrams for specimens CPLL at the side of the specimens where failure occurred

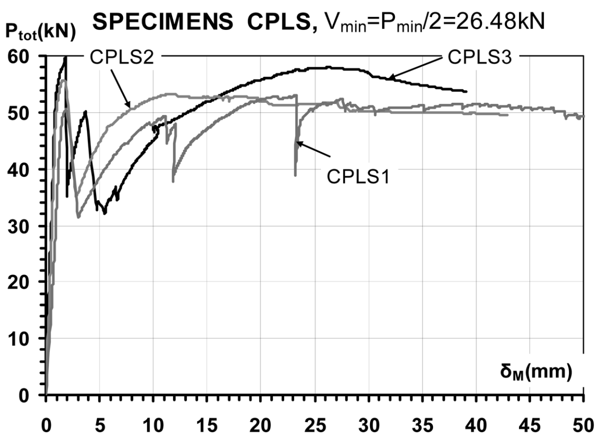


Fig. 12 Comparative load - deflection diagrams for specimens CPLS at the side of the specimens where failure occurred

25 mm) is only about 10% lower than that prior to debonding in the CPLL group, and almost the same in the CPLS group. It is noted again that the maximum resistance is reached at only 2 to 3 mm of deflection, which clearly shows the importance of the post-debonding mechanism to the deformability of the composite slab. The post-peak branch of the load-deflection curve is favourably affected by the ductile stress-strain diagram of the steel sheets (Fig. 6).

Slip between the steel sheet and concrete was measured at both ends of the specimens and, as expected was higher at the end where the main crack formed. The measured slip displacements are about 15 mm at the end of the test, as shown in the load vs. longitudinal slip diagrams of Figs. 13 and 14. It is pointed out that debonding occurred in both the low and high shear ratio specimens, the only difference being the deflection at which debonding took place, which was higher in the CPLL group (high shear ratio).

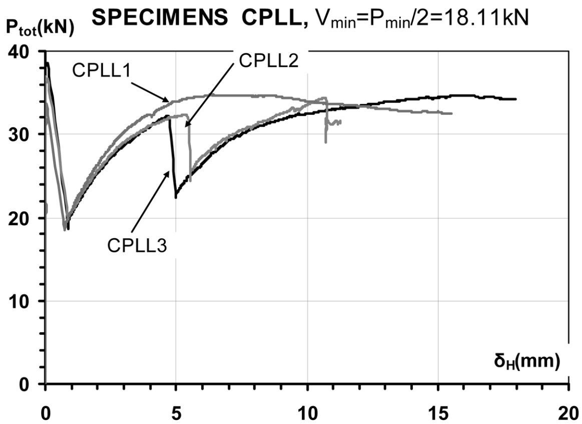


Fig. 13 Load vs. horizontal sliding of the steel sheet relative to concrete for specimens CPLL

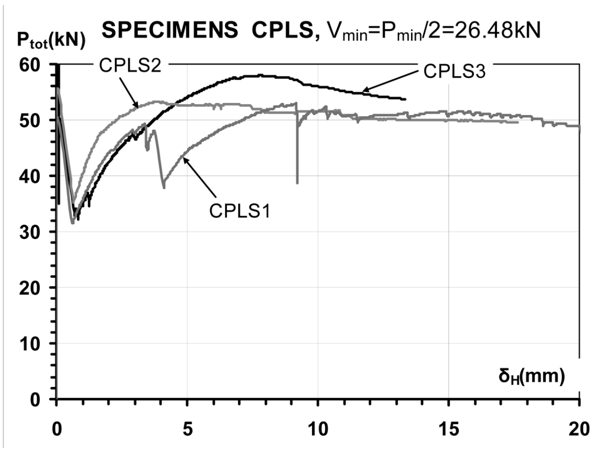


Fig. 14 Load vs. horizontal sliding of the steel sheet relative to concrete for specimens CPLS

5. Estimation of *m* and *k* factors using alternative approaches

The shear forces measured in the tests were used for the determination of m and k factors, needed for the verification of the composite slab against longitudinal shear; these shear forces (V_t) are equal to half the values of the developed strengths and are shown in Figs. 11 to 14. From the tests of two groups of composite slabs, each of which included three specimens, the variation in measured strength did not exceed 10% in each group. Therefore, according to §B.3.5(3) of EC4 the characteristic strength is taken as the lower recorded strength reduced by 10%. The line plotting the normalised V_t vs. A_p values was then drawn, from which $m = 82.23$, $k = 0.028$ was found.

A critical point mentioned in paragraph B.3.5(1) of EC4 is discussed in the following. This paragraph states that if the behaviour is ductile, the representative experimental shear force V_t should be taken as 0.5 times the value of the failure load W_t as defined in B.3.4. If the behaviour is brittle this value shall be reduced by a factor 0.8. According to paragraph 9.7.3(3) the longitudinal shear behaviour may be consider as ductile if the failure load exceeds the load causing a recorded end slip of 0.1 mm by more than 10%. As can be seen in the load vs. horizontal slip diagrams, given in Fig.s 13 and 14, the measured slip is 0.1 mm at the beginning of the first descending branch of the diagram. It has to be stressed that this rule is defined on the basis of values that are very sensitive to estimate from the test results and have no clear physical meaning. In contrast, there is physical meaning in the comparison between the minimum strength (strength reduction after loss of chemical bond between the steel sheet and concrete topping) and the strength that is developed between steel and concrete through the friction mechanism. This indicates that subsequent to loss of chemical bond, a stable system results with significant strength, very close to the strength at the loss of chemical bond, as noted in the previous section. According to this, the response that is shown in diagrams 11 and 12 is considered ductile and the values that were calculated above, $m = 82.2$ and $k = 0.028$ are valid. If the factors m and k are calculated according to the 'unfavourable' interpretation of EC4 (reduction of $0.9W_t$ by 20%), the resulting values are $m = 66.08$, $k = 0.022$.

Finally, another approach for estimating m, k was applied. According to this, the factors m and k are calculated on the basis of the shear force resisted by the friction mechanism (i.e., not by the shear force at the point of loss of chemical bond, prior to the first strength drop). As mentioned previously, the strength due to chemical bond and the strength due to friction between the steel sheet and concrete do not differ substantially. Following these calculations, the resulting line has a steeper slope than in the other two methods, $m = 91.04$, but smaller initial ordinate, $k = 0.0117$. The diagrams that show the lines that define the m and k values for each case are given in Fig. 15, both separately and in a comparative diagram.

6. Software for the design of composite slabs

As part of the present study, special-purpose software for the design of composite slabs with profiled steel sheets was developed, based on Eurocode provisions and engineering judgement where appropriate. VB.NET programming language was used in a windows-based, user-friendly, stand alone application.

This section focuses on the software's capability to perform parametric evaluation of shear strength for a set of structural configurations (i.e., number of spans, span lengths, loading, and boundary conditions). The key requirements were to: (a) facilitate the investigation of the impact of different approaches for the estimation of m-k factors on the overall design of composite slabs, (b) fully comply

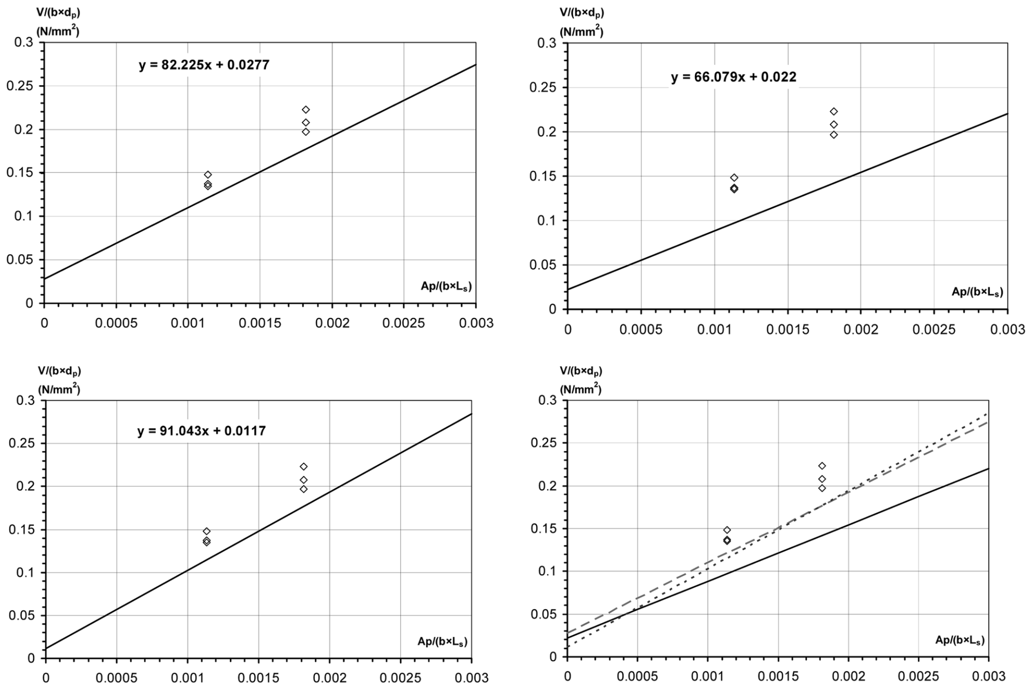


Fig. 15 Lines defined by *m* and *k* factors, and comparative diagram

with the current code framework (i.e., Eurocodes 2, 3 and 4), (c) assist the designer to use typical sheet profiles in practical applications, while acquiring a deeper understanding regarding the mechanics of the behaviour of the profiled sheet through a transparent computational scheme where all assumptions and calculations are made available at every step, and (d) evaluate the effectives of design separately, for the construction stage and the operation stage.

The program flow is straightforward and no prerequisites are needed. At start-up, the designer can first input the general information related to the project under study and decide the basic parameters of the problem. This is actually the only action initially available, in the sense that other options are not yet accessible but "unlock" gradually to establish a clear, "one-way flow". Next, the (geometrical and material) section properties of the profiled sheet are given by the user, followed by the definition of the desired composite slab section and the detailed visualization of the computed resistance of the slab against positive bending moments, negative bending moments, vertical shear and longitudinal shear forces (Fig. 16). At the same time, all calculations made are recorded on a separate logfile to provide a transparent overview of the analysis steps and facilitate an in-depth understanding of the design process. Having defined the desirable profiled sheet and composite slab section, the designer can specify the structural system of the slab. A continuous beam of up to 10 spans can be described (Fig. 17). The beam is solved with the use of the three moments (Clapeyron) method. There is no limit as to the length (L) of each span or to the live load (Q) set, while the self weight (G) of the steel sheets (required for the construction phase) and of the composite slab (for the operation phase) are calculated automatically. The structure is solved for the load combination $G + Q$ for the construction phase and for $1.35G + 1.50Q$ for the operation phase; however, the user can choose independently the critical distribution of the live load among the various spans. The software also permits consideration of different support conditions,

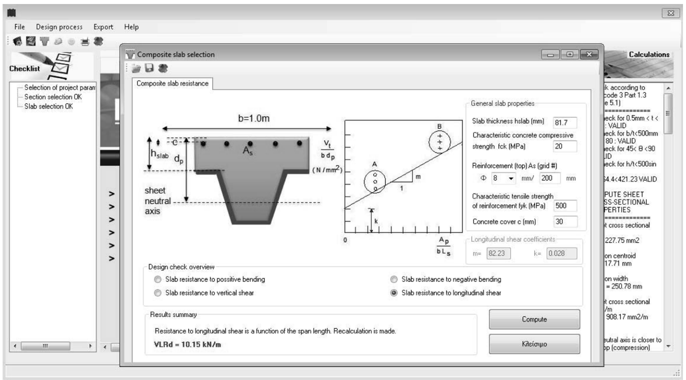


Fig. 16 Visualization of the composite slab resistance against longitudinal shear forces

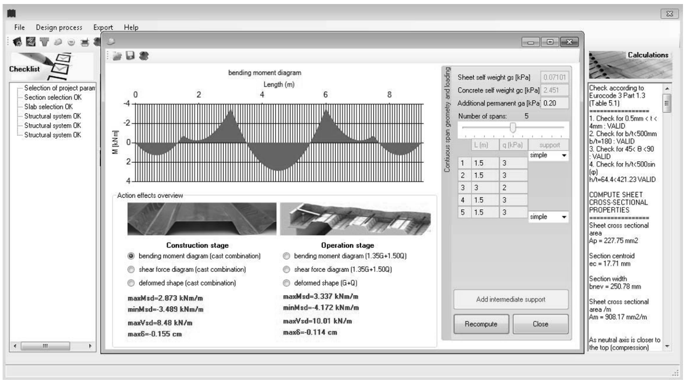


Fig. 17 Description and modification of the structural system by introducing an intermediate support to split the critical span

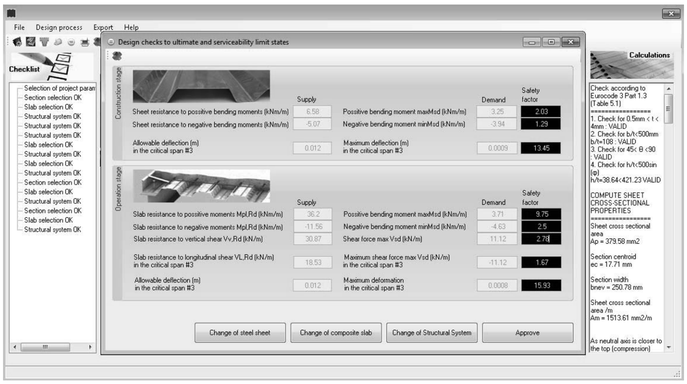


Fig. 18 Final design check against serviceability and ultimate limits states

as well as the existence of cantilevers at either end of the beam. In this way, the vast majority of the structural systems used in practice can be tackled. Note that, as before, all input data and analysis results are monitored and recorded in the logfile which appears on the right hand column of the program, while the user can visualize the calculated bending moments and shear forces to be developed during the construction and operation phases (Fig. 17). The deformed shape of the slab can also be determined. Finally, it is the designer's decision to modify the length of the spans or the value and distribution of the live loads to investigate the critical cases where the resulting action effects (bending moments, shear forces, and deflections) exceed the computed allowable resistance or deformation limits. Design verifications for the ultimate and serviceability limit states are performed by the software (Fig. 18) right after completing structural analysis. The checks are conducted for all spans of the structural system, given the fact that both the resistance to longitudinal shear and the allowable deflections are span-dependent.

By visualizing the developed capacity-to-demand ratio, in the form of safety factors, referring to flexural resistance, shear resistance, and deformations of the system, the user can re-design the system using his/her engineering judgment, either by providing a stronger sheet or thicker slab sections or by changing the structural system by introducing intermediate (temporary) supports. Moreover, the impact of different *m-k* values on the overall design process can be parametrically studied, as described in the following section.

7. Impact of *m, k* factors on the shear strength of the composite slab and the corresponding failure mode

Using the developed software and the experimentally defined values of the *m, k* factors, sets of design tables can be prepared, such as Tables 1 and 2, for different combinations of slab thickness, span length and structural system (different number of spans). The critical live load q in each case is determined based on the specific prevailing mode of failure (i.e., flexure under positive and negative bending, longitudinal shear, and vertical shear); note that the tables do not include values $q < 1$ kN/m^2, as these are not relevant for design. Such tables are useful for a quick selection of the composite slab configuration in practical applications.

Table 1 Design table of the composite slab for different values of live load q (kN/m^2). Case of single-span slab

Number of spans:	1
Sheet thickness (mm):	0.75
Concrete class to EC8:	C20/25
Steel grade:	S500
Reinforcement (slab top):	#Φ8/200

	Span length L(m)												
Slab thickness (cm)	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00
13	31.1	20.6	13.6	9.3	7.0	**5.1**	**3.8**	**2.8**	**2.0**	**1.5**	**1.0**		
14	33.2	23.0	15.0	10.4	**7.5**	**5.8**	**4.2**	**3.1**	**2.3**	**1.7**	**1.1**		
15	35.2	24.7	16.2	11.2	**8.3**	**6.2**	**4.5**	**3.3**	**2.5**	**1.8**	**1.2**		
16	36.8	26.7	17.5	12.3	**9.1**	**6.8**	**5.0**	**3.7**	**2.7**	**2.0**	**1.3**		
17	38.2	28.6	19.1	13.4	**9.9**	**7.4**	**5.4**	**4.0**	**3.0**	**2.1**	**1.4**		
18	39.8	30.6	20.5	14.4	**10.6**	**7.9**	**5.8**	**4.3**	**3.2**	**2.3**	**1.6**		
19	41.2	32.9	22.0	15.4	**11.3**	**8.5**	**6.2**	**4.6**	**3.4**	**2.4**	**1.7**		
20	42.7	33.8	23.3	**16.3**	**12.0**	**9.0**	**6.5**	**4.8**	**3.6**	**2.6**	**1.8**		

x	No intermediate support is needed during the construction stage
x	Intermediate support(s) are needed during the construction stage to prevent flexural failure

Table 2 Design table of the composite slab for different values of live load q (kN/m²). Case of two-span slab

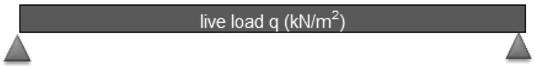
Number of spans:	2
Sheet thickness (mm):	0.75
Concrete class to EC8:	C20/25
Steel grade:	S500
Reinforcement (slab top):	#Φ8/200

	Span length L(m)												
Slab thickness (cm)	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00
13	25.0	16.0	10.3	7.0	**5.0**	**3.6**	**2.6**	**1.8**	**1.2**				
14	26.2	17.5	11.4	7.7	**5.5**	**4.0**	**2.8**	**1.9**	**1.3**				
15	27.4	19.2	12.5	**8.5**	**6.0**	**4.4**	**3.1**	**2.1**	**1.4**				
16	28.8	21.0	13.6	**9.2**	**6.6**	**4.8**	**3.4**	**2.3**	**1.6**				
17	30.1	22.4	14.5	**9.8**	**7.0**	**5.1**	**3.6**	**2.5**	**1.7**	**1.0**			
18	31.2	24.0	15.5	**10.5**	**7.5**	**5.5**	**3.9**	**2.6**	**1.8**	**1.1**			
19	32.2	25.0	16.4	**11.1**	**8.0**	**5.8**	**4.1**	**2.8**	**1.9**	**1.1**			
20	33.3	25.9	17.6	**11.9**	**8.5**	**6.2**	**4.4**	**3.0**	**2.0**	**1.2**			

x	No intermediate support is needed during the construction stage
x	Intermediate support(s) are needed during the construction stage to prevent flexural failure

Table 3 Comparison of the effect of determination of the '*m-k*' values on the critical live load q (kN/m²) of a single-span composite slab

Number of spans:	1
Sheet thickness (mm):	0.75
Concrete class to EC8:	C20/25
Steel grade:	S500
Reinforcement (slab top):	#Φ8/200
Slab thickness (cm)	20



	Span length L(m)												
(m,k)	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00
(91.04, 0.0117)	42.70	33.79	**23.81**	**16.29**	**11.94**	**8.79**	**6.35**	**4.56**	**3.32**	**2.35**	**1.45**		
(82.23, 0.028)	42.70	33.79	**23.31**	**16.29**	**12.01**	**8.99**	**6.55**	**4.83**	**3.60**	**2.59**	**1.76**		
(66.08, 0.022)	42.70	**27.40**	**17.80**	**12.22**	**8.84**	**6.44**	**4.51**	**3.11**	**2.10**	**1.34**			
(82.23, 0.028) vs. (91.04,0.0117)	0%	0%	-2%	0%	1%	2%	3%	6%	8%	10%	21%		
(66.08, 0.022) vs. (91.04,0.0117)	0%	-19%	-25%	-25%	-26%	-27%	-29%	-32%	-37%	-43%			

V_{vRd} (kN)	V_{LRd} (kN)
34.98	**16.53**
34.98	**17.29**
34.98	**13.83**

x	No intermediate support is needed during the construction stage
x	Intermediate support(s) are needed during the construction stage to prevent flexural failure

x.xx	vertical shear is the critical failure mode
x.xx	longitudinal shear is the critical failure mode

Moreover, an effort was made to investigate the effect of determining the *m-k* values on the basis of alternative assumptions, as discussed in Section 5. Some selected results are shown in Tables 3 and 4, which refer to the most critical case of a thick (200 mm) slab. Also shown (to the right of the main tables) are the calculated values of shear resistance, both transverse (V_{vRd}) and longitudinal (V_{LRd}). For the first two sets of *m-k*, it is seen that in most cases (typically for span length $L < 3.0$ m) the effect of different *m-k* determination is generally low as differences in the allowable live load do not exceed 10%. On the contrary, it becomes significant (at least in terms of percentage change) in case of long spans ($L > 3.0$ m). It has to be noted, however, that in this case, the live load that the slab can sustain is

Table 4 Comparison of the effect of determination of the ‘*m-k*’ values on the critical live load q (kN/m²) of a two-span composite slab

Number of spans:	2
Sheet thickness (mm):	0.75
Concrete class to EC8:	C20/25
Steel grade:	S500
Reinforcement (slab top):	#Φ8/200
Slab thickness (cm)	20

	Span length L(m)													V_{vRd}	V_{LRd}
(m,k)	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00	(kN)	(kN)
(91.04, 0.0117)	33.42	25.93	**17.92**	**11.94**	**8.54**	**6.18**	**4.28**	**2.86**	**1.86**	**1.04**				34.98	**16.53**
(82.23, 0.028)	33.42	25.93	**17.60**	**11.94**	**8.54**	**6.18**	**4.38**	**3.00**	**2.01**	**1.21**				34.98	**17.29**
(66.08, 0.022)	33.42	**21.13**	**13.51**	**8.96**	**6.28**	**4.33**	**2.77**	**1.67**						34.98	**13.83**
(82.23, 0.028) vs. (91.04,0.0117)	0%	0%	-2%	0%	0%	0%	2%	5%	8%	16%					
(66.08, 0.022) vs. (91.04,0.0117)	0%	-19%	-25%	-25%	-26%	-30%	-35%	-42%							

x	No intermediate support is needed during the construction stage
x	Intermediate support(s) are needed during the construction stage to prevent flexural failure

x.xx	vertical shear is the critical failure mode
x.xx	longitudinal shear is the critical failure mode

low (less than 2.0 kN/m^2) and such a solution would not be acceptable in most practical applications. It was also observed that when the span is short (i.e., L = 1.0), the prevailing mode of failure is ‘vertical’ shear, hence the alternative definition of the *m-k* values does not affect the design. However, in all other cases (L > 1.25 m, or shear ratio $\alpha_s \geq 7.8$) longitudinal shear governs the final design. For all these cases, the assumption regarding strength after loss of chemical bond leads to only a marginal increase in the maximum permissible live load. This can be attributed to the fact that, in contrast to other modes of failure, the longitudinal shear strength is a function of the span length. The use of the conservative estimate of *m-k* (66.08, 0.02) leads to significantly lower levels of maximum permissible live load due to its inferior (by approximately 20%) resistance to longitudinal shear. It is also noted that this effect becomes more significant for longer span lengths, independently of the number of spans.

8. Conclusions

In the light of the present experimental work, EC4 provisions related to longitudinal shear in composite slabs were critically assessed, focussing on the definition of: (a) the upper and lower limit of the applied 5000 load cycles, and (b) the failure load that should be considered in the determination of *m* and *k* factors. It was concluded that the limits $0.2W_t$ and $0.6W_t$ should be defined in a more pragmatic and unambiguous way, making sure that the load to be applied through the hydraulic jack is always realistic. Although the second interpretation of the cyclic load limits proposed herein (based on the measured ‘net’ failure load recorded during the monotonic test of the specimen) is more conservative, no loss of chemical bond during the application of 5000 cycles was observed. It is concluded, therefore, that factors 0.2 and 0.6 should be increased, if the intention of the cyclic loading is to induce loss of bond.

For the estimation of *m* and *k* factors the strength that should be taken into account should also be defined appropriately. In the framework of the present study three interpretations of the EC4 provisions were explored:

- Use of the load corresponding to maximum strength, developed just prior to loss of chemical bond.

- Use of 80% of the aforementioned value whenever the failure load does not exceed by at least 10% the load that corresponds to longitudinal slip equal to 0.1 mm.
- Use of the ultimate (post-peak) strength, related to friction mechanisms developing after loss of chemical bond, and manifested by local buckling of the steel sheet.

The second interpretation is the more conservative one, and the resulting *m-k* factors are lower. The third interpretation results in lower *k* factor and higher *m* factors than in the first interpretation.

To further demonstrate the implications of the above issues on the design process, an analytical approach implemented into a computer software was developed, based on Eurocode provisions and engineering judgment. It was found that the method of predicting the *m-k* factors has a visible influence on the design of composite slabs, since for medium and high values of the shear ratio, longitudinal shear governs the final design. It is therefore clear that this important point of EC4, related to the reliable estimate of longitudinal shear resistance, should be clarified in a future edition or amendment of the Code.

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