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Autor: Leskelä, Matti V.

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Composite Decking Unit of Thin-Walled Z Purlins and Thin Concrete Slab

Matti V. LESKELÄ
PhD, Civil Eng.
University of Oulu
Oulu, Finland

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

Summary

A prefabricated composite decking unit is described, in which Z and Sigma purlins are used as floor beams and are connected to a relatively shallow concrete slab. The concrete slab is cast on thin-gauge profiled sheet shuttering connected to the purlins by self-tapping and self-drilling screws in troughs in the sheeting. The behaviour of a typical decking unit and the properties of the screws as shear connectors are explained as observed in tests and in numerical analyses performed using layered beam elements.

1. Introduction

Prefabricated decking units composed of a thin concrete slab cast on thin-gauge profiled sheeting and Z and Sigma purlins represent a new development that has also been used to some extent in Finland. Their behaviour as composite structures and the behaviour of the self-tapping and self-drilling screws as shear connectors are discussed in this paper. Eurocode 4 (ENV 1994-1-1) can be applied to the design of these structures, and the validity of the design principles given in the Eurocode was verified by finite element calculation (method based on layered beam elements = LBE).

1.1 Principle of the decking unit

The load-bearing components of the unit are shown in Fig. 1. Concrete is cast on an assembly of steel components consisting of Z and Sigma purlins tied together by transverse low-profile steel sheeting and self-drilling and self-tapping screws, which also serve as shear connectors between the purlins and the concrete. It has been shown by tests that screws placed in troughs in the sheeting have enough strength and slip capacity to validate the use of partial and full shear connection designs according to the ENV-Eurocode 4 (ENV 1994-1-1 [1]). The size of the concrete cross-section is large enough to ensure of that the whole depth of the purlins is in tension or only slightly in compression so that no reduction has to be made due to instability and the simple theory of plasticity can be employed to evaluate the bending resistance according to Eurocode 4. For the resistance of self-tapping screws and similar devices, failure modes with respect to shank failure, ripping of steel, i.e. in the shuttering or the purlins, and failure of the concrete in the troughs should be considered. According to push-out tests, the resistance formula in ENV 1994-1-1, given as a function of the strength of the concrete is the most suitable for evaluating the strength of the screw connector.

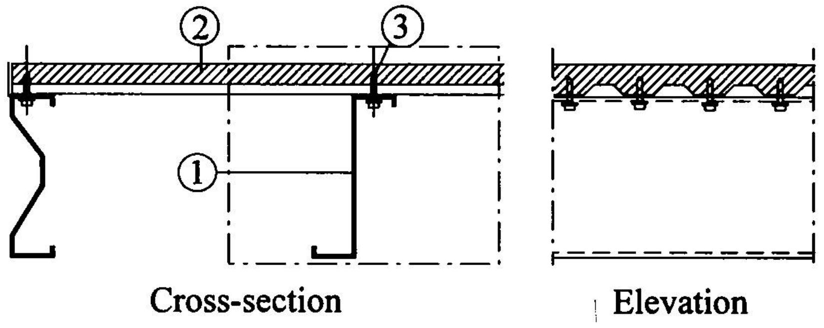

Fig. 1 Principle of the decking (showing structural components only): (1) purlins with a minimum thickness of 1.5 mm spaced at 600 mm, (2) concrete deck with a minimum thickness of 35 mm above the ribs (total depth 50 mm) and (3) screws as shear connectors.

2. Design properties

2.1 Properties of the screw connectors

The failure modes to be considered include (1) shank failure of the connector, (2) failure of the steel in the purlin flange, and (3) concrete failure in the trough. These can be evaluated according to Eurocodes 3 and 4 [1, 2]:

$$(1)\ F_{v.Rd} = \frac{0.6 A_{v.sh} f_{ub}}{\gamma_{Mb}} \quad (2)\ F_{b.Rd} = \frac{2.5 \alpha f_u d t}{\gamma_{Mb}} \quad (3)\ P_{l.Rd} = 25 \alpha \frac{A_v}{\gamma_v} \sqrt{f_{cube}(f_{cube} + 10)^{1/3}}$$

where d is the shank diameter of the screw, t the wall thickness of the purlin, f_{ub} the ultimate strength of the screw material and f_u the ultimate strength of the purlin material. Equations (1) and (2) are found in Eurocode 3, and equation (3) is a transformation from equation 6.14 of Eurocode 4. $A_{v.sh}$ is the shank area and A_v is an area calculated according to the stress diameter of the threaded section in the screw, and α may be taken as unity according to tests.

Since there is no way to evaluate the slip capacity of the connector theoretically, push-out tests according to section 10.2 of Eurocode 4 were carried out (Fig. 3). Two types of the screw described in Fig. 2 were tried, but there were no major differences in behaviour between them when the appropriate load-slip curves were considered in coordinates scaled to unity.

It was found out for the screws employed that only equations (2) and (3) should be considered, as no signs of shearing were observed in the shanks of the screws after the tests and the failure mode included both yield in bearing and failure of the concrete. In fact, (2) and (3) yield values of the same magnitude, when the grade of the concrete is at least C30/35. Equation (2) gave a slightly higher nominal resistance for the screw than (3), and it is thus recommended that equation (3) should be employed with respect to purlin materials not worse than S320 (f_{yp} = 320 MPa, f_{up} = 420 MPa).

For screws with a shank diameter of 5 mm (stress diameter 5.5 mm), the typical characteristic resistance from equation (3) varies between 6 and 7 kN, values which were exceeded by more than 10 % in the tests. The load-slip curves in the tests also showed that there is a large slip capacity for the resistance described above.

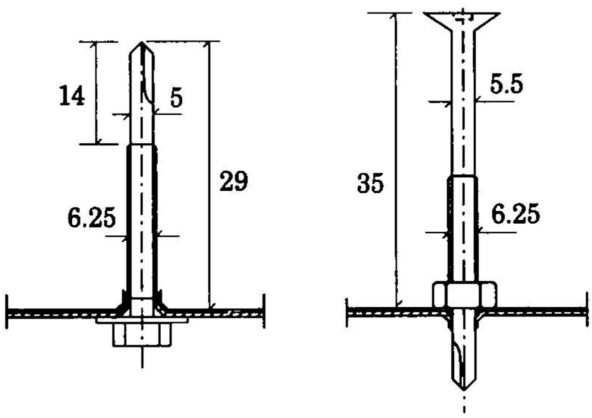

Fig. 2 Types of screws for which push-out tests were carried out. The screw on the right was used as a shear connector in the decking units tested for bending.

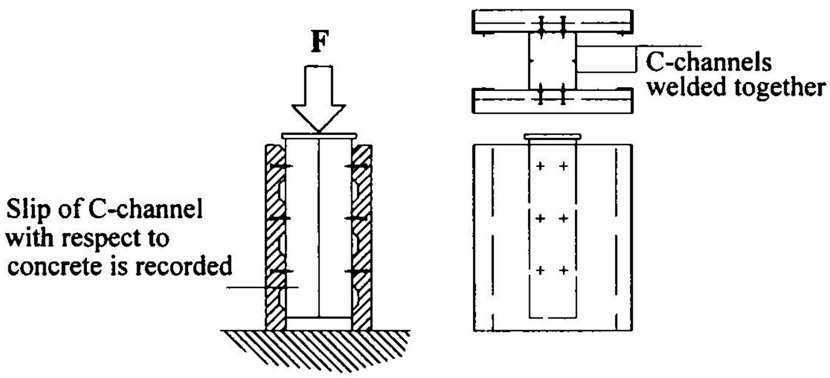

Fig. 3 Principle of the push-out test carried out to determine the load-slip properties of the screws as shear connectors. The thickness and material properties of the C profiles should be compatible with those of the Z purlins.

2.2 Composite cross-section

The span of the decking unit is always such that the whole width of the unit can be exploited for the effective concrete section. Assuming a full shear connection, a maximum value for the bending resistance is obtained. The stress state of the steel sections can be checked easily by applying the maximum design forces of the composite cross-section:

F_{tf} = maximum tensile force of the steel sections = $A_a f_{yp}/\gamma_a$
F_{cf} = maximum compressive force of the concrete section = $0.85 A_c f_{ck}/\gamma_c$

where A_a and $A_c = b_c h_{c1}$ are the total cross-sectional areas of the steel and concrete sections, respectively. Denoting that $\beta_x = x/h_{c1} = A_a f_{yp} \gamma_c/(0.85 A_c f_{ck} \gamma_a) \leq 1$ as the relative depth of the concrete section in compression, it may be easily seen that, for the purlins normally employed, $\beta_x < 1$ and the total steel section is in tension. The lever arm, z, for the evaluation of the bending resistance is $z = h_a/2 + h_{c2} + (1 - 0.5\beta_x)h_{c1}$ and $M_{pl,Rd} = zF_{tf}$. To obtain the maximum resistance, the number of shear connectors in the half span of the decking should be $N \geq N_f = F_{tf}/P_{l,Rd}$. Considering the partial safety factor for the shear connection, $\gamma_v = 1.25$, and the length of the span in the decking unit, this cannot normally be achieved, but the degree of the shear connection, $\eta = N/N_f$, is not far from unity.

2.3 Flexural loading tests for the decking units

Three test units consisting of three 200 mm deep Z purlins and a concrete slab with a width of 1500 mm were loaded in flexure with four equal line loads across the decking at the L/5 points on the span. The load-deflection curves for the specimens are shown in Fig. 4, together with the curve calculated by the method of finite elements (LBE) developed for the purpose [3, 4]. The load-slip properties for the shear connection were adopted from the push-out tests for the screw connectors and the stress-strain curve for the cold-worked steel in the purlins was modelled based on the coupon tests.

The test loadings were continued as far as possible so as to be sure that the full plastic resistance of the composite cross-section would be obtained. A stress diagram at the termination of loading, as defined from the strain measurements and material tests, is shown in Fig. 5, and the bending moment as integrated from the stress diagram complies with the maximum moment obtained, which is greater than the resistance $M_{pl.Rk}$ calculated according to simple plastic theory and stress blocks as in Eurocode 4.

It is seen in Fig. 5 that although not the whole of the steel section is plastic, $M_{pl.Rk}$ could still be reached, due to the lever arm of the internal forces being greater than $h_a/2$. The real distribution of stresses and their resulting force may even have a beneficial effect on the total force of the connection, i.e. a fully plastic shear connection is not required in reality to balance the tensile force of the steel section, and a considerable amount of the slip capacity is left in reserve.

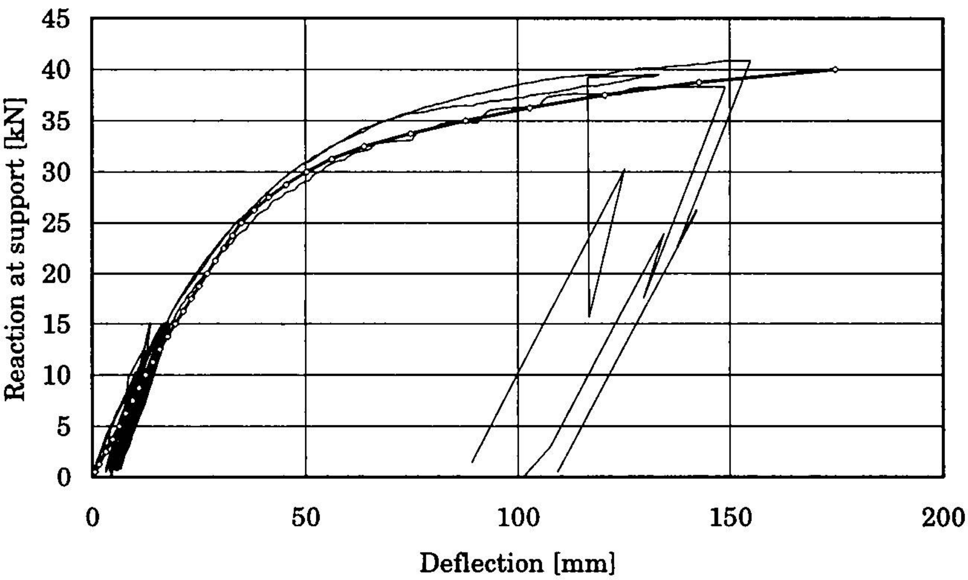

Fig. 4 Load-deflection behaviour of a decking unit having a span of 6 m, as observed in three loading tests (curves without markers) and a finite element calculation (curve with markers).

End slips were not measured in the flexural tests, but as monitored visually, they were in excess of 3 mm at the termination of loading. This would imply that the connector resistances were reached at least at the ends of the spans.

The loading in all the tests of Fig. 4 was terminated while the load could still be increased, once $M_{pl.Rk}$ had been reached. This was done due to the instability of the loading system caused by the excessive deflection.

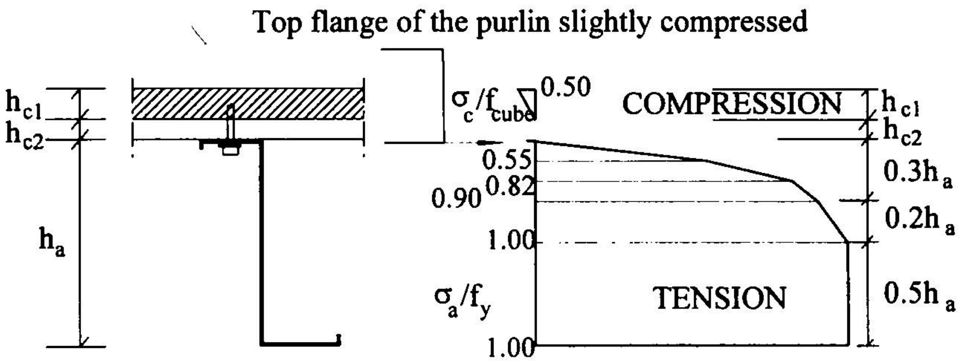

Fig. 5 Stress diagram at the termination of loading, as defined from the strain measurements.

2.4 Serviceability properties

The variation in the properties of the shear connection does not normally have any noticeable influence on the deflections in a composite beam, all the time the degree of the shear connection is close to unity. Unfortunately this is not true for the structure considered here, and the flexibility of the connection should be allowed for when evaluating the effective stiffness of the system. This may be done by increasing the nominal modular ratio, $n = E_a/E_c$, and $n_{eff} = 3n$ would be an appropriate selection for the test specimens and for considering the short-term behaviour. Even then the behaviour of the decking is satisfactory and no major design problems would occur.

To avoid discomfort due to vibration, the value of the lowest eigenfrequency, f, should be checked and should not be less than 4 Hz:

$$f = 500\pi \sqrt{\frac{E_a I_1}{m L^4}} \geq 4 \text{ Hz} \tag{4}$$

where $E_a I_1$ is the effective short-term bending stiffness of the system in MNm^2, L the span of the decking in metres and m the dead-weight of the decking in kg/m. For spans not greater than 6 m the above limit is easily exceeded, and the typical eigenfrequencies then are in excess of 6 Hz. The frequency requirement in equation (4) is not presented in Eurocode 4 and not even in Eurocode 3, but only in Eurocode 5 for timber structures. However, it is equally important to consider vibrations in composite structures.

3. Discussion

The decking unit described in this paper can be designed satisfactorily by following the methods of Eurocode 4. It is shown that the shear connection provided by the self-tapping and self-drilling screws may be considered ductile, and the normal degrees of the shear connection are not much below a full connection. The resistance of the screw connectors can be evaluated conservatively according to the formulae for headed studs in Eurocode 4, employing the stress area as the effective cross-sectional area of the connector. The strength of the screw material is such that no failures in the screws are to be expected and failure in the push-out tests took the form of simultaneous yielding in bearing of the steel and shearing in the concrete.

While the resistances for bending and vertical shear, and for longitudinal shear, will not normally cause any poblems, more attention should be paid to verification of the serviceability of the system. The additional deflection due to the flexibility of the connection can be considered by an application of the increased modular ratio, which should further be increased to allow for long-term effects. The decking may be precambered in connection with the concreting, by supporting the purlins as cantilevers. The effects of concrete shrinkage during drying and solidification of the slab concrete was monitored within the flexural test specimens, and no reduction in the precamber produced by the weight of the concrete was observed during the curing period.

Detailing of the system components is not discussed in this paper, but it should be pointed out that the normal problems emerging in thin-walled structures, such as web crippling at supports, need to be remembered. They can be considered by applying the rules of Eurocode 3.

4. Acknowledgements

The kind permission of Rautaruukki to publish material related to the tests discussed is gratefully acknowledged.

5. References

[1] Eurocode 4: ENV 1994-1-1, Design of composite steel and concrete structures, Part 1.1, General rules and rules for buildings. CEN 1992

[2] Eurocode 3: ENV 1993-1-1, Design of steel structures, Part 1.1, General rules and rules for buildings. CEN 1992

[3] Leskelä, M., Calculation Models for Concrete-Steel Composite Beams, Considering Partial Interaction. Acta Universitatis Ouluensis, Series C, Technica No 16. University of Oulu, Oulu 1986

[4] Leskelä, M.V., A finite beam element for layered structures and its use when analysing steel-concrete composite flexural members. Constructional Steel Design. World Developments (354-358). Elsevier Applied Science, London and New York 1992