STRENGTH REDUCTION IN HOLLOW-CORE SLABS WHEN SUPPORTED BY BEAMS AND GIRDERS

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ABSTRACT

Hollow-core slab units are widely used as decking systems in Finland. They were originally designed to be supported on walls, which provide a non-flexible support. In such cases the web of the slab is subject to a vertical shear stress and a longitudinal normal stress, caused by the shear force and the prestressing force of the slab respectively. When these slabs are supported by beams or girders, the shear failure condition is met at much lower load levels than is the case when they are assembled on non-flexible supports.

This paper discusses the estimation of the reduced shear strength of such slabs, due to inevitable involvement in the behaviour of a composite beam formed as the result of a structural decision of this kind.

A brief review of the flexural theory of composite structures is given to introduce the stiffness formulation employed.

Key words: *composite construction, hollow-core slab, shear strength, failure condition*

1. INTRODUCTION

Prestressed hollow-core slabs are a common part of decking systems, and are nowadays being used increasingly in composite floors. When assembled on beams, a portion of the slab becomes an effective part of the beam cross-section and the slab becomes inevitably affected by the composite interaction between the beam body and the slab units after the groutings and other concreting has been finished and the concrete in the joints has gained adequate strength.

For all loads acting after the interaction ability, the web of the slab is affected by an additional transverse shear force component, attributable to the longitudinal shear flow of the composite beam, which balances the changes in the normal stress resultants in the direction of the beam axis.

The shear flow of the composite action must be transferred through the web sections of the hollow-core units (FIG. 1) and thus the stress state of the webs in the slab is governed by two shear stresses, one due to the shear force of the slab and the other due to the longitudinal shear flow of the beam. As both of these tend to increase the tensile stresses in the web, the failure condition is met at an earlier stage of loading than would be the case if the slabs were supported on non-flexible surfaces. The shear flow is introduced in all beams, independent of the surface which supports the slabs, if the deformational compatibility condition is met. The level of the shear flow depends on the rigidity of the connection between the beam body and the slab units.

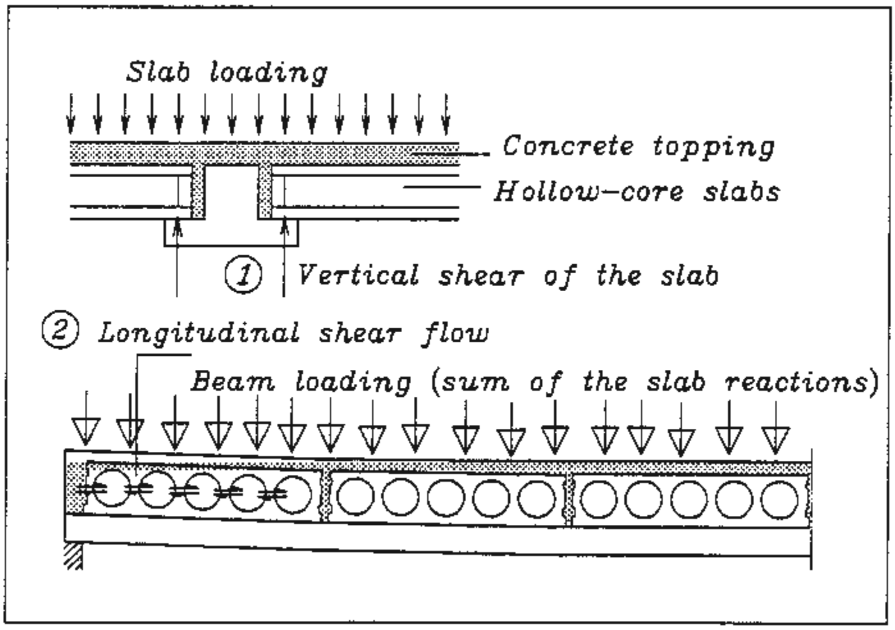

FIG. 1. *Shear forces acting on hollow-core sections when slabs are supported by beams: 1) vertical shear due to slab loading, 2) longitudinal shear flow due to composite beam behaviour.*

2. COMPOSITE BEHAVIOUR

A short review of the flexural theory of composite beams is given to show that composite behaviour can ensue even if not intended in the design. The only necessary condition for this is that the beam body and the slab units should be interconnected so that equal interface strains (complete interaction) or a known strain difference between the members can be produced at some common point of reference.

For the sake of generality, two randomly shaped members with indices 'c' and 's' are considered. Let the members be so assembled that they are in contact along the beam only on one level 'j'. The centroidal axes of the members are denoted by C_c and C_s when the members are bent separately (FIG. 2).

In the case of composite behaviour, an external flexural moment M is always composed of stress resultants M_c, M_s (internal moments) and Ne_i, where $N = N_s = -N_c$ is the axial stress resultant acting in opposite directions on the neutral axes of the two members (tension considered as positive)

$$M = M_c + M_s + Ne_i. \tag{1}$$

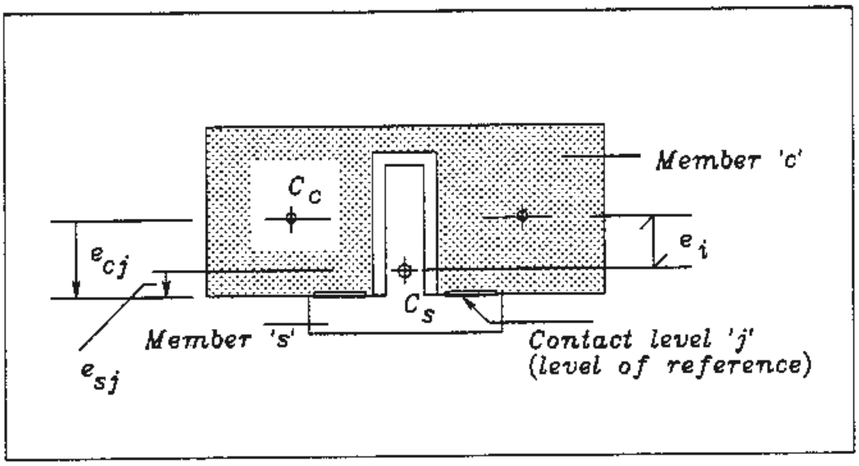

FIG. 2. *Composite cross-section of a flexural member where structural parts 'c' and 's' are in contact only on level 'j'.*

The curvature of the members is $\kappa = M/(EI)_{pi}$, in which $(EI)_{pi}$ represents the flexural stiffness of the composite structure.

The values of M_c and M_s are known on the basis of their common curvature

$$M_c = \kappa(EI)_c\,,\quad M_s = \kappa(EI)_s, \tag{2}$$

in which $(EI)_c$ and $(EI)_s$ are the flexural rigidities of the members and their sum denoted by $(EI)_{ni} = (EI)_c + (EI)_s$ represents the non-interactive stiffness. Thus

$$M = \kappa((EI)_c + (EI)_s) + Ne_i = \kappa(EI)_{ni} + Ne_i. \tag{3}$$

This equation is valid independent of the interaction rate, because the only condition applied is the common curvature formed on the basis that the members are in contact at all points along the beam axis. The strains of the interface 'j' are equal in the case of complete interaction, otherwise the slipping at the interface causes a strain difference (partial interaction). The interface strains or the strains ε_{cj} and ε_{sj} on the contact level are calculated by means of axial rigidities and curvature

$$\varepsilon_{cj} = -N/(EA)_c + \kappa e_{cj},$$
$$\varepsilon_{sj} = N/(EA)_s + \kappa e_{sj}. \tag{4}$$

The distances e_{cj} and e_{sj} are measured as being positive in a downward direction with respect to the centroids of the members (FIG. 3).

It is assumed that the strain difference $(\varepsilon_{cj} - \varepsilon_{sj})$ is known. To solve it from equations (4) requires the procedure below

$$N/(EA)_c + N/(EA)_s = \kappa(e_{cj} - e_{sj}) - (\varepsilon_{cj} - \varepsilon_{sj})$$

$$= \kappa e_i - (\varepsilon_{cj} - \varepsilon_{sj}),$$

$$N = [\kappa e_i - (\varepsilon_{cj} - \varepsilon_{sj})](EA)_c(EA)_s/(EA)_i, \qquad (5)$$

$$(EA)_i = (EA)_c + (EA)_s.$$

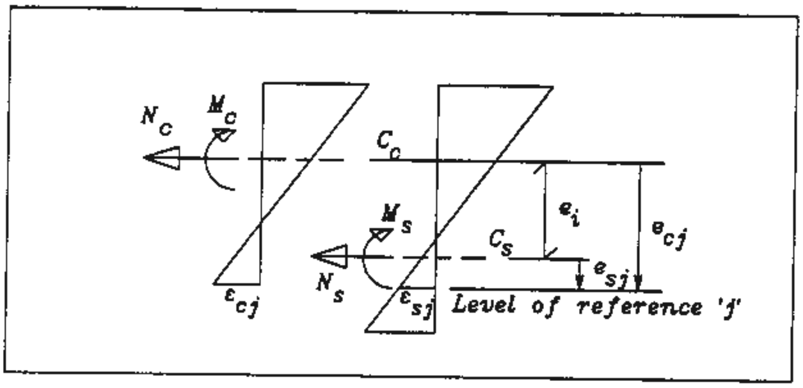

FIG. 3. *Strain diagrams for the members 'c' and 's'. The stress resultants are shown to act in their positive directions.*

The stress resultant N is a suitable measure for the interaction rate. The formulation above shows quite clearly that the maximum force is obtained when the strain difference vanishes.

To obtain the relation N - M, N must be inserted in equation (3)

$$M/\kappa = (EI)_{pi} = (EI)_{ni} + e_i[\kappa e_i - (\varepsilon_{cj} - \varepsilon_{sj})](EA)_c(EA)_s/(EA)_i$$

An abbreviation α_i can be used for a dimensionless term, the composite stiffness coefficient

$$\alpha_i = e_i^2(EA)_c(EA)_s/[(EI)_{ni}(EA)_i]. \qquad (6)$$

Applying this, the general formula for flexural stiffness is written briefly as

$$(EI)_{pi} = (EI)_{ni}[(1+\alpha_i) - (\varepsilon_{cj} - \varepsilon_{sj})\,\alpha_i/(\kappa e_i)]. \qquad (7)$$

Because the strain difference varies along the beam axis, equation (7) represent a non-constant stiffness, which obtains its maximum value (= $(EI)_i$, complete interaction) at the points of the zero moment and its minimum at the points of maximum moment. The average is always smaller than $(EI)_i$.

Composite beams and girders can be categorised according to the interaction rate. 'Short spans' refer to cases where the strain difference is able to accumulate as slip at the beam ends due to a flexible connection and the complete interaction rate cannot be reached. 'Long spans' refer to the opposite cases.

The approximate limit for traditional short beams is L < 20 m. With this knowledge it can be readily understood that most cases involved in the design of house constructions belong to the category of short spans, in which the complete interaction rate cannot be reached. Due to friction, however, the real interaction rate is always above zero, even though no efforts are taken to provide an adequate connection.

2.1 Shear connection and shear flow

A shear flow is induced to maintain the horizontal force equilibrium in any elements separated from the composite member. The longitudinal shear flow $v_l dx$ must balance the difference in the stress resultants formed over the length dx

$$v_l dx = \int_{A_c} d\sigma_c(y) dA_c = \int_{(EA)_c} d\varepsilon_c(y) d(EA)_{cn}. \tag{9}$$

This requires insertion of the strain $\varepsilon_c(y) = \kappa(x) y_c - N(x)/(EA)_c$, from which equation (10) follows.

$$v_l(x) = e_{cc}(EA)_{cn} d\kappa(x)/dx - [(EA)_{cn}/(EA)_c] dN(x)/dx. \tag{10}$$

e_{cc} is measured to the centroid of area A_{cn} using axis C_c as the origin, and $(EA)_{cn}$ is the corresponding axial stiffness. Equation (10) is the general formulation for the longitudinal shear flow, but it is not easy to use for design purposes unless complete interaction is assumed, in which case the curvature $\kappa(x)$ and stress resultant $N(x)$ are well known in a closed form. The shear flow is then written simply as

$$v_l(x) = e_c(EA)_{cn} V(x)/(EI)_i, \tag{11}$$

in which e_c is the distance from the neutral axis of the composite section to the centroid of area A_{cn} and $(EI)_i$ is the flexural stiffness based on complete interaction.

It is only for the case of complete interaction that the simple relation between the longitudinal shear flow and the vertical shear force of the beam holds exactly. Equation (11) always gives the upper bound value for the shear flow, however, and this can also be used to calculate the partial interaction when the axial stiffness $(EA)_{cn}$ is chosen appropriately (the deflections can also be calculated using the same values).

The proper stiffness values (i.e. the design widths of the compressive flange) can be evaluated by using more accurate methods of analysis for each type of structural system. These include FEM-systems which employ the flexible connections between the slabs and the beam body.

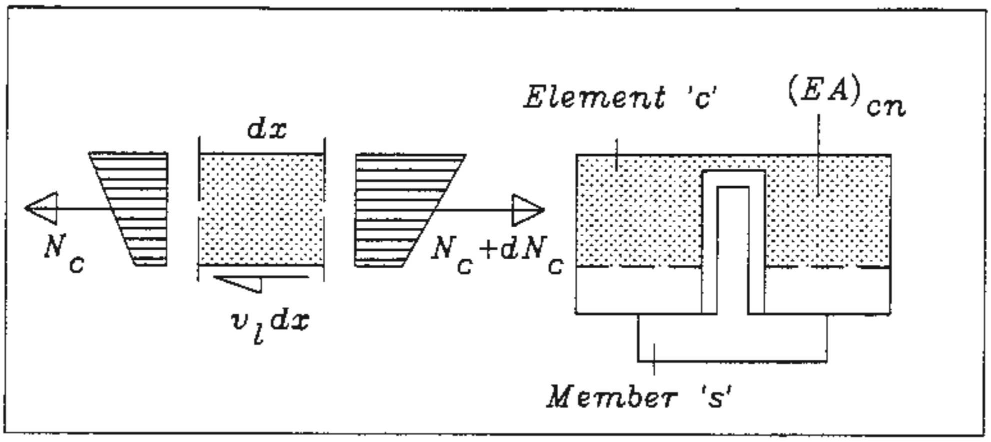

FIG. 4. *Horizontal equilibrium must always be maintained between the change in the normal stress resultant and the longitudinal shaar force $v_l dx$ in the element 'c'.*

2.2 Stress state in the webs of hollow core slabs when supported by beams

In the case discussed here, the webs of the slab units are stressed by two shear stress components (proportional to the loading of the slabs), and by one normal stress (constant, due to prestressing) in the vicinity of the slab ends. Since the maximum shear flow is encountered at the beam ends, the slabs are stressed most heavily there. The two shear stresses tend to reduce the level of the onset of shear failure and the shear resistance of the slabs is reduced, sometimes very drastically.

3. FAILURE CONDITION OF CONCRETE

Failure function of concrete developed by N.S. Ottosen /1/ is used here to study 'premature' shear failure in hollow-core slabs. The invariants required for the failure function are derived by means of the stress tensors involved. The function is normally written as

$$F(\sigma_{ij}) = AJ_{2\sigma}/f_c^2 + \lambda\sqrt{J_{2\sigma}}/|f_c| + BI_{1\sigma}/|f_c| - 1 \qquad (12)$$

where λ is a function of the deviatoric angle θ (octahedral stress theory, e.g. explained in /2/), calculated on the basis of

$$\cos 3\theta = (3\sqrt{3}/2)(J_{3\sigma}/J_{2\sigma}^3). \qquad (13a)$$

$$\lambda = K_1 \cos\{\frac{1}{3} \arc\cos(K_2 \cos 3\theta)\} \text{ with } \cos 3\theta \geq 0$$

$$\lambda = K_1 \cos\{\frac{\pi}{3} - \frac{1}{3} \arc\cos(-K_2 \cos 3\theta)\} \text{ with } \cos 3\theta \leq 0 \qquad (13b)$$

A, B, K_1, K_2 are parameters that are calibrated so that four specific failure states become exact points on the failure surface /2/. These include uniaxial compressive strength $|f_c|$ and tensile strength f_{ct}.

Five sets of parameters can be readily applied to the failure function. These are based on the experimental data found in references /1/, /3/, /4/, /5/ and /6/ (the parameters based on /5/ and /6/ are also summarised in reference /2/).

It is not easy to obtain new values for different qualities of concrete, because biaxial strengths are also required, and these cannot be tested using normal uniaxial compression loading machines. The problem of the lack of exact parameters is discussed in chapter 3.3.

TABLE 1. Parameters of the failure function of Ottosen

f_{ct}/f_c	A	B	K_1	K_2	
0.087	1.2259	3.3699	11.9298	0.9914	/1,3,4/
0.121	0.8653	2.6322	9.6653	0.9801	/1,3,4/
0.145	0.6262	2.1386	8.1620	0.9647	/1,3,4/
0.121	2.1868	2.8460	9.1854	0.9962	/5/
0.121	1.4426	2.7148	9.4710	0.9900	/6/

3.1 Determination of the invariants

FIG. 5 shows the stress components for which the invariants of the stress state must be derived. σ_{11} is the normal stress due to prestressing, σ_{13} (or σ_{31}) is the shear stress due to the vertical shear force of the slab, and σ_{23} (or σ_{32}) is the shear stress due to the longitudinal shear flow of the beam.

Following the eigenvalue procedure for the determination of the principal stresses σ_1, σ_2, σ_3, $I_{1\sigma}$ is found to be equal to σ_{11}. Now, as $\sigma_{oct} = \sigma_{11}/3$, the deviatoric stress state, in which the diagonal components s_{ii} are

$$s_{11} = \sigma_{11} - \sigma_{oct} = 2\sigma_{11}/3,$$

$$s_{22} = -\sigma_{oct}, \qquad (14)$$

$$s_{33} = -\sigma_{oct},$$

yields the invariants $J_{2\sigma}$, $J_{3\sigma}$

$$J_{2\sigma} = \sigma_{11}^2/3 + \sigma_{13}^2 + \sigma_{32}^2, \qquad (15a)$$

$$J_{3\sigma} = 2\sigma_{11}^3/27 - 2\sigma_{11}\sigma_{32}^2/3 + \sigma_{11}\sigma_{13}^2/3. \qquad (15b)$$

3.2 Calculation of the failure curves

When the failure condition is employed, it is relevant to use relative values for the stress components, proportioned to the tensile strength of concrete, f_{ct}

$$\eta_P = \sigma_{11}/f_{ct},$$

$$\eta_{hc} = \sigma_{13}/f_{ct}, \qquad (16)$$

$$\eta_{vl} = \sigma_{32}/f_{ct}.$$

It must be noted that compression is always considered to be negative. For convenience, the following abbreviations are used with the failure function

$$F_1 = \sqrt{J_{2\sigma}}/|f_c| = r_f\sqrt{\eta_{hc}^2 + \eta_{vl}^2 + \eta_P^2/3}, \qquad (17a)$$

$$F_2 = J_{2\sigma}^{3/2}/f_{ct}^3 = (\eta_{hc}^2 + \eta_{vl}^2 + \eta_P^2/3)^{3/2}, \qquad (17b)$$

$$F_3 = J_{3\sigma}/f_{ct}^3 = 2\eta_P^3/27 - 2\eta_P\eta_{vl}^2 + \eta_P\eta_{hc}^2, \qquad (17c)$$

$$r_f = f_{ct}/|f_c|. \qquad (17d)$$

These allow it to be written as .

$$F(\sigma_{ij}) = AF_1^2 + \lambda F_1 + Br_f\eta_P - 1, \qquad (18)$$

where λ is the same as in equations (13) and the angle θ is calculated briefly from

$$\cos 3\theta = (3\sqrt{3}/2)F_3/F_2. \qquad (19)$$

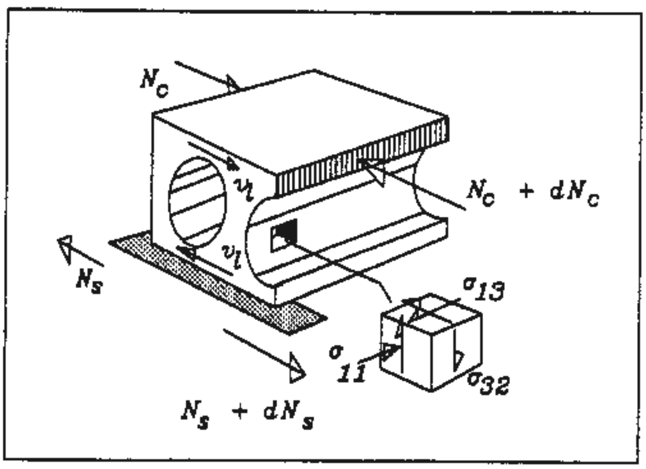

FIG. 5. *Stress components for the failure consideration.*

A short code for a computer program was written for use with equations (17) - (19). Different sets of η_{hc}, η_{vl} that satisfy the failure condition $F(\sigma_{ij}) = 0$ were examined while σ_{11} (or η_P) was kept constant. The results are shown in the diagrams in FIG. 6, for which the following data were used:

$r_t = 0{,}074$ (= 1/13.5)
parameters A, B, K_1, K_2 according to TABLE 1, row 5.

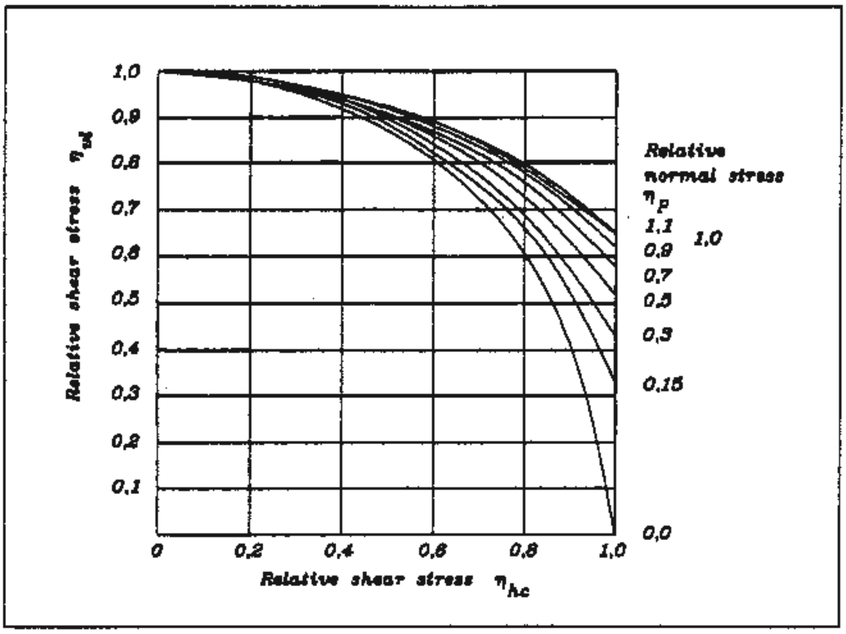

FIG. 6. *Shear failure curves for different values of normal stress σ_{11}. Relative normal stress is calculated with respect to the tensile strength of concrete, $\eta_P = \sigma_{11}/f_{ct}$.*

3.3 Discussion on the failure curves

As can be readily seen from TABLE 1, the parameters applied in FIG. 6 are not 'exactly valid', because the ratio of the uniaxial strengths r_t does not belong to any of the rows of TABLE 1. Row 5 was selected instead of row 1 because the data connected with the parameters chosen apply to modern concretes and these parameters yield slightly lower failure values than row 4.

At boundaries, where $\eta_{vl} = 0$ or $\eta_{hc} = 0$, the calculation repeatedly yields values $\eta_{hc} = 1.32$, when $\eta_{vl} = 0$, or vice versa. Consequently the curves in FIG. 6 were scaled with respect to the maximum stresses of the boundaries. All the time the compressive normal stress η_P is non-zero, it is correct to obtain values of $\eta_{vl} > 1$ or $\eta_{hc} > 1$ if only one shear stress is applied. In any case, in order to reach a more universal and conservative solution, it is better to scale all the values to give strengths $\eta \leq 1$.

The innermost curve in FIG. 6 also deserves special consideration. For this diagram the normal stress $\eta_p = 0$, and thus the curve must be symmetric with respect to a line drawn from the origin with an inclination of 45 degrees. It is seen that this condition is satisfactorily met. The outermost curve represents a failure state when there is considerable compression applied together with the shear stresses. Since the curves for $\eta_p = 1$ and $\eta_p = 1.1$ are quite close to each other, it can be assumed that the outermost curve approximately represents the failure envelope for all stress cases $\eta_p > 1.1$.

The failure curves are straightforward to use. Failure in pretensioned hollow-core slabs takes place in a region where there is always some prestress present. For a good approximation, it is assumed that an average of $\eta_p = 0.25 \ldots 0.3$ gives a sound, conservative basis for any calculation, although it has not been possible to check these considerations against representative test data. A design curve coming close to $\eta_p = 0.3$ has been adopted in Finland /8/ as the code of practice for composite beams supporting hollow-core slabs. This curve can be expressed as a quadratic function

$$(\tau_{vl}/f_{ctd})^2 + 0{,}84(\tau_{hc}/f_{ctd})^2 = 1 \qquad (20)$$

The calibration required for the use of the design curve (20) includes formulae for calculating the shear stresses η_{vl} due to the longitudinal shear flow. The difficulty in this lies in the fact that there are various rates of interaction in different types of beam. E.g. there are steel beams that have so far been designed omitting the interaction, as if doing so removes it totally. The beam body itself can be designed conservatively by means of this assumption of course, but the load must in all cases be transferred to beams from slabs and premature failure of the slabs must be taken into account.

4. DESIGN REQUIREMENTS

Beams categorised as having flexible connections refer to a full plastic design method, in which the redistribution of connection forces due to the plastic deformations requires ductility in the joint. Connections made with hollow-core slabs cannot be considered ductile, unless any efforts are made to reinforce the slab ends.

At our present state of knowledge, the connections must be designed as rigid, by calculating the shear stresses η_{vl} based on the formula (11). In addition, the full plastic analysis of the beam cannot be considered, and the ultimate limit state for the overall system is assumed to be reached when the elasticity limit of the strain in the beam is exceeded.

Systems comparable to those in FIG. 7 have attracted increasing interest because of their economy, but they cannot be designed employing the customary way of considering limit states. The essential reasons are shown clearly in Figures 5 and 6.

Defining the ultimate flexural limit state according to the first yielding of the beam body must not be considered as a disadvantage. It can be shown that all the design requirements involved are better balanced and yield admissible loads closer to each other than would be the case when also the pasticity is considered.

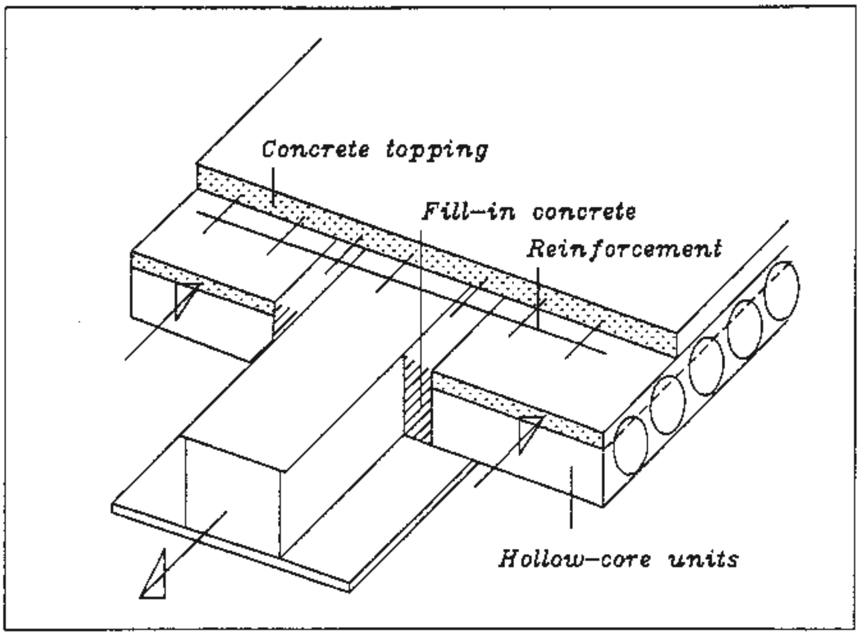

FIG. 7. *A typical flooring system employing hollow-core slabs. It is not essential to the failure of the slabs what materials the beams are made of. Since the changes in the stress resultants of the cross-sectional parts must be balanced by the shear flow, the webs of the slab units are forced to behave like shear connectors between the beam body and the compression section formed by the shaded area of the hollow-core members. One part of the shear flow is transferred by the reinforcement if assembled across the beam.*

4.1 Calculation of relative stresses η_{vl} and η_{hc}

Shear stress η_{hc} due to the vertical shear force of the slab can be calculated according to elasticity theory, and for the accuracy required for the design purposes, it is satisfactory to use the formula

$$\eta_{hc} = 1.6V_{hcd}/\{f_{ctd}(h_{hc}\Sigma b_i)\}. \qquad (21)$$

h_{hc} is the depth of the slab section and Σb_i the sum of the web widths in one slab unit. V_{hcd} is the design shear force of the slab.

For a shear stress η_{vl}, a certain distribution area A_{vl} for the longitudinal shear flow must used. This is represented as

$$A_{vl} = k_{vl} \varnothing_{hc} \Sigma b_i, \quad (22)$$

in which $\varnothing_{hc}$ is the diameter of the hollow cores (in the case of oval cores, the horizontal diameter is used). k_{vl} is a coefficient of distribution which must be calculated according to a more rigorous theory of elasticity, or based on test results, if available.

There are only a few full-scale tests which can be used to estimate the value of k_{vl}, and according to these, it seems quite sound to propose a design value $k_{vl} = 1.5$, as has also been adopted in the Finnish code of practice /8/, for cases in which more accurate values are not available. It must also be noted that the distribution coefficient is a function of the interaction rate, i.e. if the value proposed is used, the axial stiffness of concrete members must also be estimated accordingly, so that the effective flexural stiffness of the composite structure is modelled reliably.

It can be well understood from FIG. 6 that the location of the coordinate points η_{hc}, η_{vl} is sensitive to variation in the tensile strength of the concrete. he design strength f_{ctd} is calculated from a formula related to the cube strength K

$$f_{ctd} = k_f K^{2/3}/\gamma_c \quad (23)$$

in which k_f is given values of 0.2 ... 0.27 depending on the code applied. For prefabricated products it should be sufficient to use $\gamma_c = 1.35$. According to Eurocode 2/Part 1 it may be suitable to use the mean value of the tensile strength, which is obtained with $k_f = 0.27$. The strength data collected from the web sections of the hollow-core slabs confirm that $k_f = 0.25$ gives the best fit, hence this value has been the basis for deriving the failure curves ($K = 60\ MPa$, $f_{ctd} = 3\ MPa$) and checking the validity of the distribution coefficient $k_{vl} = 1.5$.

4.2 Reinforcement of the slab ends

The reduction of the shear resistance of the slabs in the structures tested so far has been so prominent that it becomes a determining design factor and clearly restricts the level of admissible loads on the slabs. To increase the area sheared, the webs should be made thicker, but this gives rise to demands for product changes, which are only a theoretical alternative. Another possibility for increasing the area A_{vl} is to fill an adequate length of cores so that restraint against the sway effect is obtained. One task for future research will be to provide the rules for effective and economical filling.

5. DISCUSSION

It has been shown clearly that there is a reduction in the shear resistance of hollow-core slabs when they are supported by beams. This is explained by composite interaction in which some volume within the slab units become an active section of the composite beam, intentionally or sometimes unintentionally.

The problems that arise with the use of hollow-core slabs were first recognized in beams having a low profile (partially or totally encased in concrete, i.e. slim flooring systems, FIG. 7), in which the slabs must be supported by the tensile layers of the beam. The soffit of the slabs is strained according to the deformation state of the supporting layer, and thus cracking emerges, some of which will be located on the strands. The development length of the full prestressing force will then increase, because the anchorage properties are affected by the cracking.

This phenomenon is taken into account in design by limiting the straining of the supporting surface in the serviceability limit state after commencement of the interaction to a maximum of $\Delta\epsilon_l \leq 0.7$ o/oo, which allows for virtually four equal cracks of surface width $w_k \leq 0.175$ mm per metre (or in practice per slab unit). This proposal can be considered conservative.

Cracks have been found in tests to form partly along the tendons, but mostly along the hollow cores, spaced so that the reduction in shear resistance due to the loss of prestress is not noticeable even at the ultimate limit state. The condition discussed will nevertheless become a determining one in many cases, and it is understood that the propped construction method cannot be employed. In any cases, the unpropped method is mostly favoured because of its ease of implementation.

More serious is the resistance loss in the slabs encountered at the beam ends. A typical structure may be considered as an example. Slabs supported by beams normally span a distance of 7 ... 8 m and have a characteristic dead-weight of 4 kN/m^2 (the design weight will then be roughly 5 kN/m^2). Admissible live loads to be carried by the slabs are determined on the basis of the failure mechanism first activated.

If no reduction in the shear resistance occurs, it will be flexural failure that takes place, becase the unreduced shear failure load for these slabs will be very much greater than the flexural failure load, say 1.5 times. Let the total shear failure load be 31 kN/m^2 and the relative shear stress η_{vl} for the design live load be 0.8 ... 0.9, a typical value for reasonably designed beams. According to FIG. 6 and equation (20) it is required that $\eta_{hc} \leq 0.6$ for acceptable design, i.e. the remaining effective part of the shear resistance is estimated to yield some 17 kN/m^2 before shear failure. This comprises the total design load, hence the affordable design live load will not be more than $17 - 5 = 12$ kN/m^2.

Let us now compare this with a case in which reduction is not considered. From the flexural failure load $31/1.5 = 21$ kN/m^2, we find that the contribution of the design live load is equal to 16 kN/m^2, which shows the extent of the reduction.

5.1 Flooring systems employing hollow-core slabs in Eurocode 4

Slim flooring systems (e.g. FIG. 7), or others employing hollow-core slabs for their concrete structure are not considered in Eurocode 4, part 1 /10/, and it seems that the problems involved in the design of these systems are not widely known. In general, very little concern has been shown for verifying the serviceability of structures and this seems to be the weak point of the Eurocode.

The Finnish guidelines /8/ state clearly the requirements regarding the elasticity of the bent structures (especially single span systems) in the serviceability limit state, because there are many cases where a design based on the ultimate limit state cannot guarantee functionality.

The Eurocode design principles rely entirely on the idea of the plastic ductility of systems. The beams themselves can be ductile enough to develop plasticity, but as the loads must be transferred to the beams from the slabs, which are not capable of developing ductile plastic behaviour in the system, the ductility of the beams is of no use. Thus, for the overall safety of a flooring system, a linear (or almost linear) behaviour must be required up to failure of the system components (i.e. the slabs).

5.2 Conclusions

It has been demonstrated clearly that a new way of thinking is required when designing flooring systems in which hollow-core slabs are to be supported on beams and girders.

Although full plastic analysis cannot be employed, these structures are still highly economical, because

- no propping is needed and in many cases it is not even allowed, in order to ensure appropriate behaviour of the decking slabs

- the slab units assembled can be used as a working plateau even better than can the profiled steel sheeting in the case of composite slabs

- long spans can be used for the slabs

- the extra costs required by fire-proofing can be minimised
- modern architectural forms can be easily put into practice
- working in cold weather requires less preparations to carry out the concreting and less energy for curing the concrete than with the composite slabs.

6. ACKNOWLEDGEMENTS

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