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Long Span Composite Slabs

Planchers mixtes à grande portée

Verbunddecke mit grossen Spannweiten

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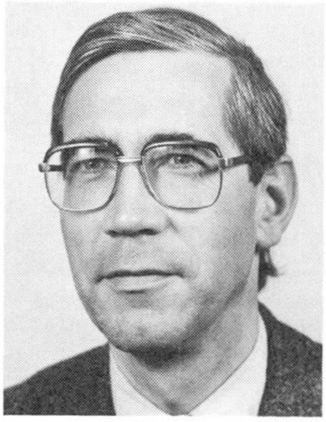

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SUMMARY

In this paper specific design aspects of long span composite slabs, with deep steel sections and relatively small ribs, are discussed. Results are given of a numerical study on the rotational capacity of such sections, leading to conclusions on the applicability of plastic analysis. A simplified method is given for the calculation of the ultimate moment. A prototype of a new special steel deck element for use in ground floors in housing is presented. This element provided with insulation is designed to span up to 5,50 m without temporary support.

RÉSUMÉ

Cette communication traite des aspects spécifiques du dimensionnement des planchers mixtes à grandes portée, à profil de grande hauteur et à petites nervures. Les résultats d'une étude numérique sur la capacité de rotation de tels profils sont présentés, ainsi que les conclusions sur la possibilité de leur appliquer une analyse plastique. Une méthode simplifiée de calcul des moments ultimes est proposée. Le prototype d'une plaque nervurée en acier pour planchers mixtes de maisons d'habitation est présenté. Cet élément, comportant une isolation thermique, est dimensionné pour franchir une portée de 5,5 m sans étai intermédiaire.

ZUSAMMENFASSUNG

In diesem Aufsatz werden Entwurfsaspekte von Verbunddecken mit grossen Spannweiten behandelt, die mit hohen Stahlblechprofilen und relativ schmalen Betonrippen ausgestattet sind. Es werden Ergebnisse einer numerischen Studie über Rotationsfähigkeit dieser Querschnitte mitgeteilt, die Schlussfolgerungen über die Anwendungsmöglichkeit der Plastizitätstheorie zulassen. Eine vereinfachte Methode zur Berechnung der Biegetragfähigkeit wird gegeben. Ein Prototyp eines neuen Deckenelementes für die Anwendung in Wohnhäusern wird präsentiert. Dieses mit Isolierung versehene Element wurde für Spannweiten bis 5,5 m entworfen.

1. INTRODUCTION

Two very important advantages of structural steel for buildings are simplicity and speed of construction. To exploit these advantages optimally, the design of the floor construction should fit within the total concept of the structural steel design. For office and appartment buildings an efficient structural solution is to use steel frames spaced at distances of 5-6 m. and composite floors directly spanning from main beam to main beam without secondary girders. The use of temporary supports (props) under the steel sheet in the construction phase should be avoided. This leads to sheets with deep profiles, relative small ribs and a minimal concrete cover (see f.e. Fig. 3). However, there are hindrances for the use of these types of floors:

- According to some standards the hight of the compression zone of the concrete may not exceed a certain maximum f.e. half the effective depth of the slab;
- The use of simple plastic design for the determination of the moment capacity is not generally permitted for deep decks. A more complicated non linear flexural strain analysis is then required;
- As the ribs are shallow, the vertical shear is more often critical. The design rules for the determination of the vertical shear capacity in most standards are overconservatif. The contribution of the steel sheet for the vertical shear is normally neglected, which is obviously not correct especially for deep decks with unpropped construction.

This paper presents results of a study undertaken as a contribution to level the first two hindrances.
For long spans other forms than trapezoidal profiled sheets are also possible. As an example, a prototype for such a special designed deck element for ground floors in housing will be presented.

2. ROTATION CAPACITY IN POSITIVE BENDING

If a reinforced concrete slab is "over-reinforced" the rotation capacity may be too small. This is caused by premature crushing of the concrete before the reinforcement yields. Therefore, the amount of reinforcement should be limited. In concrete codes this is achieved indirectly by setting a maximum for the hight of the compression zone of the concrete. Often the same rule is also adopted or refered to in specs for the design of composite slabs. For example BS 5950 - Pt 4 [3] states: 'Unless the slab has compression reinforcement, the depth of the stress block for the concrete should not exceed 0.5 d_s.' However, intuitively can be expected that with the same reinforcement ratio the rotation capacity of the considered composite cross-sections is more favourable compared with reinforced concrete. This is based on the following facts:

- The bending strength of the profiled steel sheet (reinforcement) itself is considerable, especially in case of deep decks for long spans;
- The self weight of the structure is carried by the sheet alone leading to considerable stresses in the sheet before composite action is effective;
- The yield stress (strain) of sheet material is normally smaller than for rebars.

To verify whether this holds true a numerical parameter study is carried out. With a special developed computerprogram based on non linear flexural strain analysis moment-curvature relations have been determined for a number of cross-sections. The basic assumptions of the calculations are:

- Plane sections remain plane after bending;
- Concrete in tension is neglected;
- After composite action is effective the interaction is complete (no slip);
- The stress-strain relation for steel is as shown in Fig. 1a;
- For the stress-strain relation for concrete two cases are considered as shown in Fig. 1b:
  . a bilinear diagram, according to the Dutch Concrete Code, with a maximum strain of $\epsilon_u = 3.5$ ‰;

because the numerical calculation should simulate physical tests as close as possible, a more realistic stress-strain diagram with a falling branch is also considered. This diagram is based on physical tests given in [4].

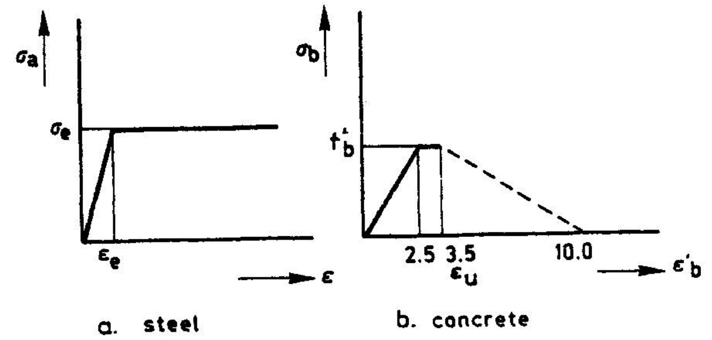


Fig. 1 Stress-strain relation for steel and concrete

In Fig. 2 the effect of the hight of the concrete cover h_b on the strain- and stress-distribution is illustrated qualitatively. Two cases are shown:

- h_b is sufficient great to cause yielding of the sheet over the full hight;
- h_b is so small that the hight of the compression zone x exceeds h_b. The steel is still partial elastic when $\epsilon_b = 3.5$ °/₀₀.

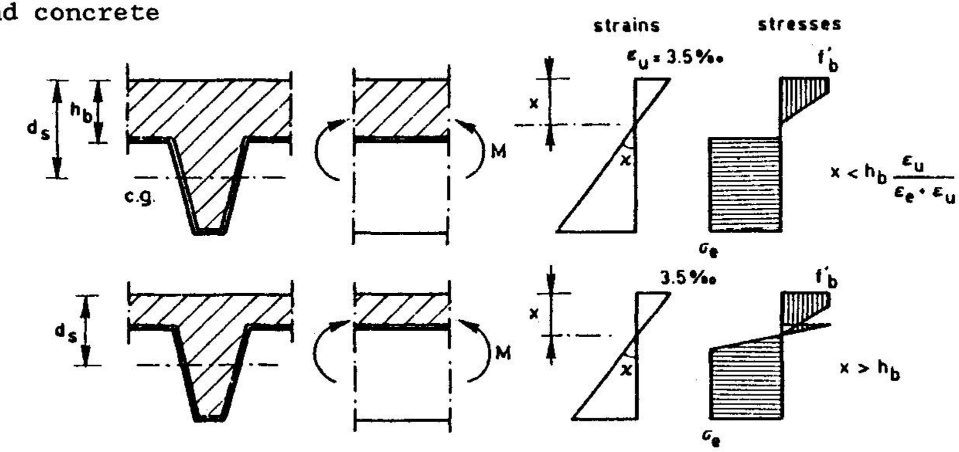


Fig. 2 Strain- and stress distributions

The parameter calculations have been carried out for the three sheet profiles shown in Fig. 3. The following parameters have also been varied:

- The concrete cover is varied from 50 to 150 mm;
- The concrete strength f'_b = 14, 18 and 24 N/mm2;
- The yield strength of the sheet: σ_e = 280, 320 and 500 N/mm2;
- The method of construction: propped resp. unpropped.

In fig. 4 the calculated moment-curvature relations for floor type II, with f'_b = 18 N/mm2 and σ_e = 500 N/mm2, are given for different hights of the concrete cover h_b. This is a rather extreme case with a relative low concrete strength and an unusual high yield strength of the steel sheet. The results for other cases are similar. Remarkable is that at maximum moment the concrete strain is greater than 3.5 °/₀₀ as indicated in Fig. 4. This means that althought the concrete strength decreases (falling branch in σ-ϵ diagram) the moment still increases. Very important is the fact

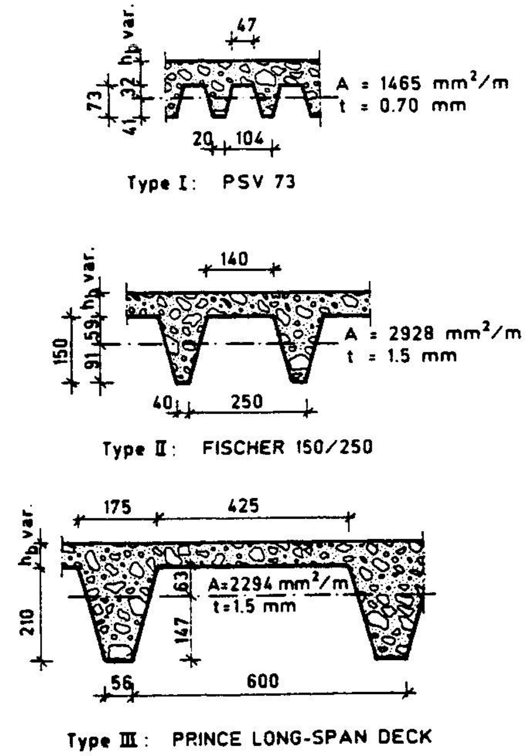


Fig. 3 Considered cross-sections

that both the curvature at ϵ_b = 3.5 °/₀₀ and the curvature at M_{max} are greater for the smaller values of h_b.
For the case given in Fig. 4 is $x > h_b$ for $h_b < 80$ mm.
This leads to the conclusion that from the point of view of rotation capacity, there seems to be no reason to put restrictions on the application of deep sections with minimal concrete cover. This conclusion should be confirmed by experimental evidence. In Fig. 5 the calculated moment-curvature diagrams are given for two cases with propped and unpropped construction. The ultimate moment in both cases is independent of the construction method as can be expected on theoretical basis [2]. The deformation capacity is greater for unpropped construction.

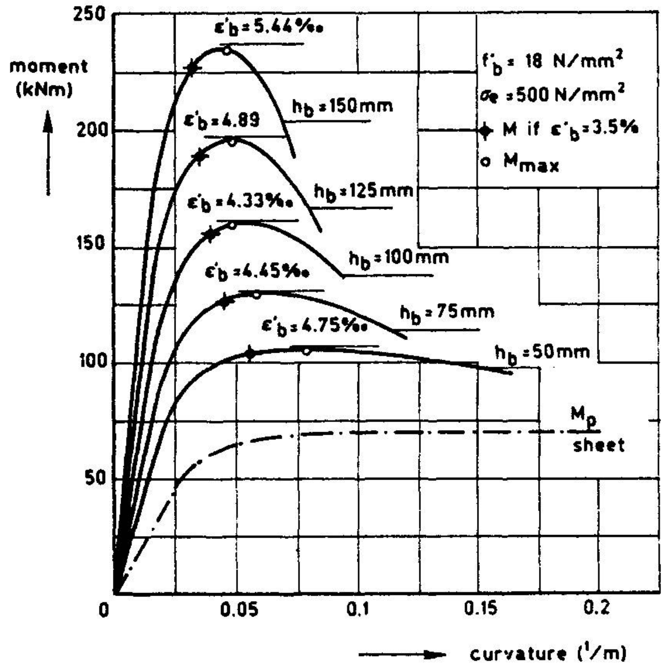


Fig. 4

Effect of variation of concrete cover h_b on moment-curvature relation

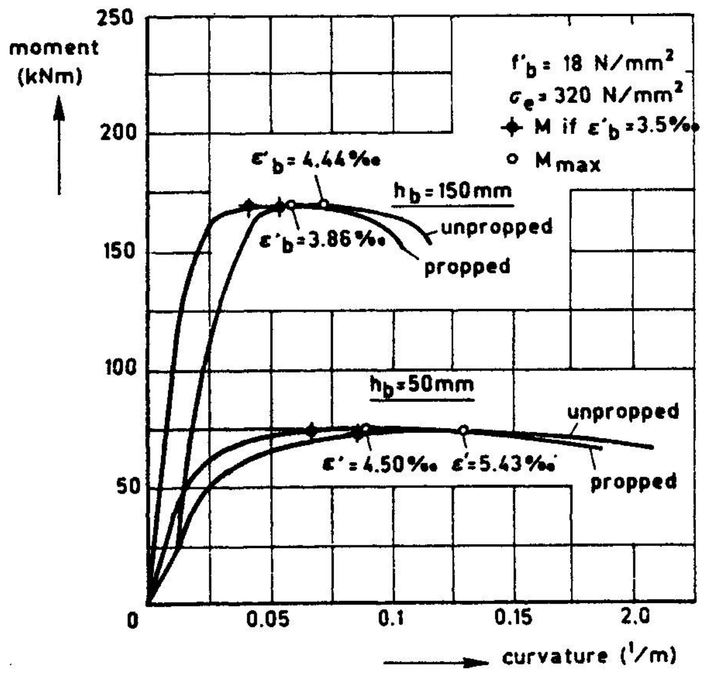


Fig. 5

Illustration of the influence of the construction method

3. SIMPLIFIED CALCULATION METHOD FOR ULTIMATE MOMENT

Use of non linear flexural strain analysis as described in paragraph 2 is not suitable for allday practical design work. Therefore a simplified method is developed. Two simplifications are introduced (see Fig. 6). The structural form is simplified by neglecting the concrete rib and a second simplification is that both steel and concrete are assumed to be ideal plastic materials. For steel this is a usual assumption. For concrete the difference between reality and idealisation is greater. To compensate for the effect of this unsafe idealisation, the design stress of the concrete f_b' is reduced with a factor k.

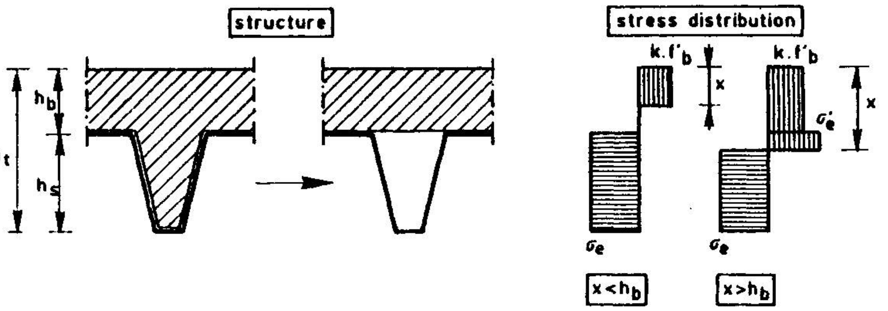


Fig. 6

Assumptions for simplified calculation methods

There are two possible cases to be considered, depending on the hight x of the compression stress block. For the type of floors under consideration practically ever is $x > h_b$. Although much simplified now, the calculation for this case is still elaborate. Therefore, a further simplification is introduced.

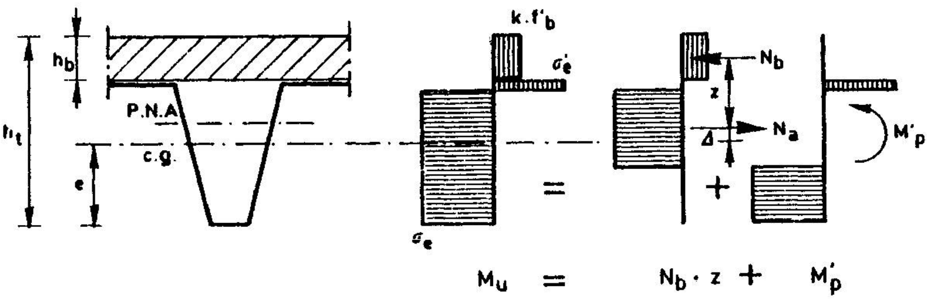


Fig. 7

Model for further simplification

As shown in Fig. 7 the stress distribution is split into two parts leading to two components of the ultimate moment. From equilibrium follows that the normal force in the sheet is equal to N_b. In this case $N_b < A\sigma_e$ so the steel section can resist an additional so called reduced plastic moment M_p'. The value of M_p' can be determined from the yield contour for combined effect of N and M on the cross-section. The yield contours have been calculated for 8 different types of sections. It has been found that the curves form a narrow band as shown in Fig. 8.

The curves can be approximated by the expression:

$$M_p' = 1.25\, M_p \left(1 - \frac{N_a}{A\sigma_e} \right)$$

where: M_p is the unreduced plastic moment.

The only unknown value is now the lever arm z. If $N_a = A\sigma_e$ ($x \leq h_b$) the line of application N_a goes through the centre of gravity. However, for smaller values of N_a the line of application shifts from the centre of gravity to the plastic neutral axis as illustrated in Fig. 9. If, as a very simple approximation is assumed that this shift is linear proportional to $N_a/A\sigma_e$ the distance e_{px} from

N_a to the bottom fibre follows from the expression:

$$e_{px} = e_p - (e_p - e)\frac{N_a}{A\sigma_e} .$$

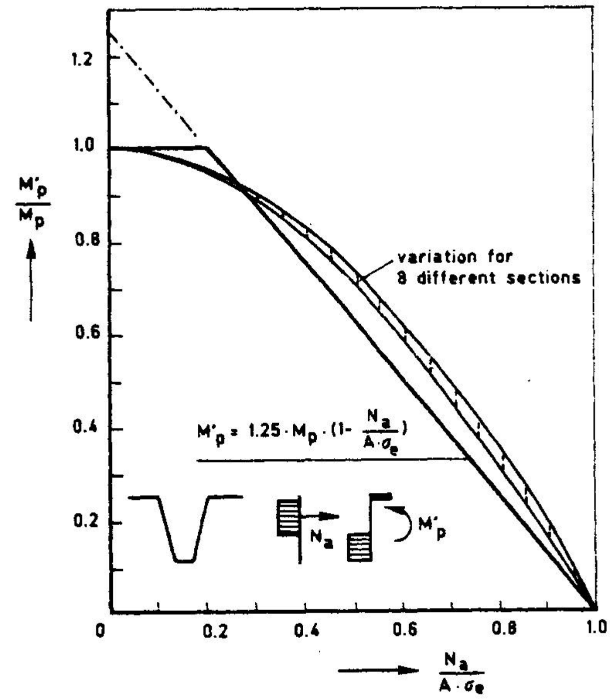


Fig. 8

Yield contours for combined effect of N and M on profiled sections

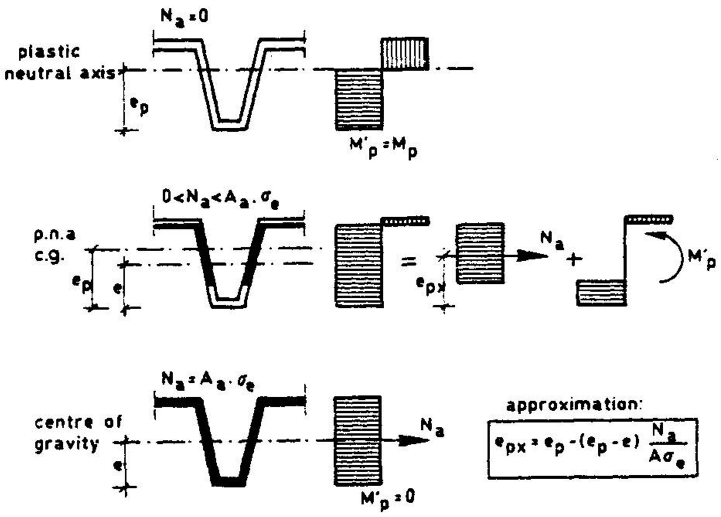


Fig. 9 Position of the tensile stress resultant N_a for various values of M.

The value of M_u can now be calculated with the following expressions:

$$M_u = N_b * z + M'_p$$

$$\text{where: } z = h_t - 0.5\,h_b - e_p + (e_p - e)\,\frac{N_b}{A\sigma_e}$$

$$M'_p = 1.25\,M_p\,(1 - \frac{N_b}{A\sigma_e}) \ngtr M_p$$

$$N_b = k\,f'_b\,h_b\,b$$

For the cases given in par. 2 the values of M_u have been calculated for $k = 1.0$, 0.9, 0.8 and 0.7. These values were compared with M_{max} as determined with the non linear flexural strain analysis. The following conclusions could be drawn:

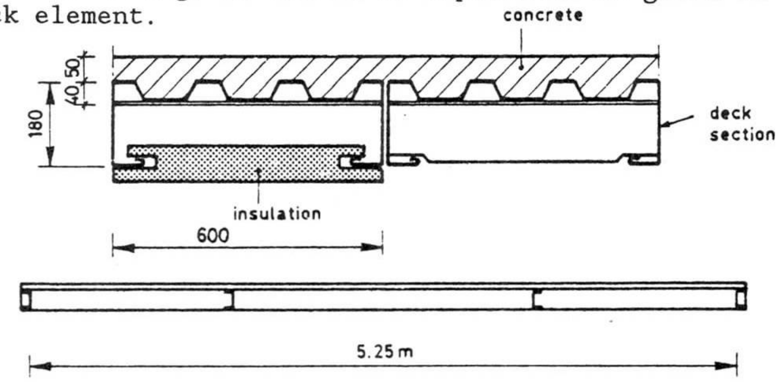
- a value of $k = 0.8$ gave the best average correspondence;
- for $k = 0.8$ the average difference between M_u and M_{max} was -1.6% and the standard deviation 1.5%.

A NEW FORM OF DECK ELEMENT

As a common effort of the Dutch steel and concrete industry a project is started to investigate possible use of composite decks in housing, especially for ground floors. This application requires some special properties:

- the possible span should be 5.50 m;
- no temporary supports;
- the element should be provided with thermal insulation;
- special attention should be paid to corrosion resistance.

Although use of trapezioded sheets described in the previous paragraphs is possible, ideas for new forms have also been developed. Just as an illustration of such a possible new form in Fig. 10 and 11 an impression is given of a prototype of a new deck element.



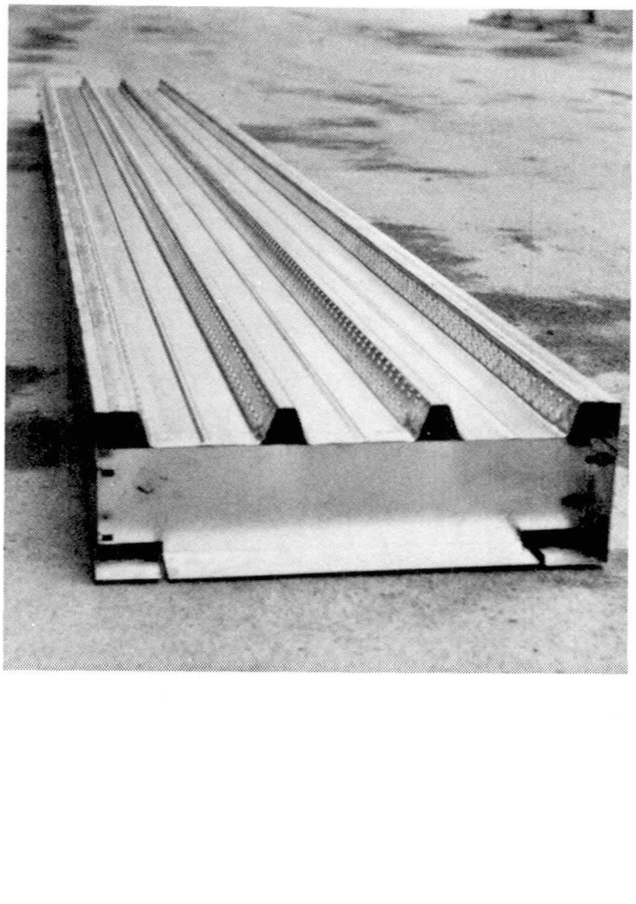


Fig. 10 Prototype of deck element

The basis is an existing trapeziodal profiled sheet with a hight of 40 mm. The webs are provided with two rows of burls to provide shear connection. Properties of this type of shear connection are given in [2]. By using a wider sheet than normal, two edge members can be formed. These edge members should provide strength and stiffness for this long span under dead weight and construction loads. To support the corrugations of the sheet and distribute loads in transverse direction, two intermediate transverse profiles are provided as shown in Fig. 10. A test programme is underway to determine the structural properties and of course also the economical feasibility is investigated.

Fig. 11 Composite deck element under test loading (field test)

ACKNOWLEDGEMENT

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