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Moment Redistribution in Profiled Sheeting

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MOMENT REDISTRIBUTION IN PROFILED SHEETING

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SUMMARY

This paper describes a simple procedure which determines the load capacity of profiled sheeting. This procedure is more accurate than existing design methods since it predicts the redistribution that occurs at interior supports.

1. INTRODUCTION

The present European Recommendations for the Design of Profiled Sheeting [1], prepared by the European Convention for Constructional Steelwork (ECCS), are based upon the effective width concept and empirically determined factors. The use of these factors is necessary in order to model the following factors :

- initial imperfections,
- residual stresses,
- elastic buckling and post-buckling behaviour of individual plate elements.

However, these factors are based upon tests which were performed on single-span specimens. When determining the flexural capacity of multiple-span specimens, calculations using these factors give conservative values. As a result, most manufacturers prefer testing to the ECCS design method.

Research investigating the nature of the reserve capacity of continuous span specimens has been carried out according to the following objectives :

- to determine the accuracy of the present ECCS design procedure,

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- to compare the ultimate load capacity of similar single and multiple span specimens using a simple test procedure,
- to determine the increase in ultimate load capacity which occurs in multiple-span specimens as a result of moment redistribution near interior supports.

2. EXPERIMENTAL INVESTIGATION

Tests have been conducted at EPFL on a wide variety of cold-formed steel sheetings to determine the moment redistribution factor at interior supports. The first of two test programs, completed in 1984, was conducted on so-called first generation profiled sheeting. First generation profiles are those that have no stiffeners in either the webs and flanges. The second test program, completed in 1986, was conducted on second generation profiled sheeting. Second generation profiles are those with stiffeners provided on the webs, flanges or both.

2.1 Test specimens

Profiled sheetings were chosen according to two criteria. Firstly, sections without embossments were used in order to reduce the number of assumptions necessary when calculating section properties and predicting behaviour. Secondly, profiled sheetings which have found common usage in Switzerland were employed. As a result, the following sections were chosen :

1.- First generation profiles tests (1984) :
   - Montana 57/0.80 mm (unsymmetric trapezoidal ribs) (t = 0.031 in.),
   - Hi-Bond 55/0.88 mm (trapezoidal ribs) (t = 0.035 in.),
   - Holorib 51/0.75 mm (dovetailed ribs) (t = 0.030 in.).

2.- Second generation profiles tests (1986) :
   - Montana 75/0.80 mm (trapezoidal ribs with one stiffener in one flange) (t = 0.031 in.),
   - Holodeck 74/0.80 mm (trapezoidal ribs with one stiffener in one flange and in each web) (t = 0.031 in.).

These five sections are shown in Figure 1. For each of the five types of sheeting listed above, three test series were performed.

This enabled a comparison of behaviour with similar single and multiple span specimens.

These tests are identified as series 1, 2 and 3. Load placement and span lengths were chosen such that test results could be directly compared with the ECCS design procedure. The coiled sheeting from which the specimens were formed was inspected both before and after the forming process. Each specimen was then marked prior to shipping. This minimised dimensional variation between test series on similar test specimens.

Test series 1 consisted of a single-span simply supported profiled sheeting, loaded symmetrically by two equal line loads. These tests determined the ultimate flexural strength of a single-span specimen in pure bending. The static system for this test series is shown in Figure 2 a).

Test series 2 consisted of a single-span profiled sheeting, simply supported and loaded at midspan. This test series determined the influence of a concentrated load in the region of maximum moment. The line load at midspan simulates the effect of the interior support of a two span specimen. The static system for this test series is shown in Figure 2 b).

Test series 3 consisted of a profiled sheeting continuous over two equal spans. Each span length was the same as that used in test series 1. Two line loads, of equal magnitude, were applied to each span at the same location as in test series 1. The static system for this test series is shown in Figure 2 c).

2.2 Test procedure

Before testing, the cross-sectional geometries and material thicknesses of all specimens were measured. All measurements were made in accordance with the ECCS Recommendations for the Testing of Profiled Metal Sheeting [2]. Six tensile test specimens were cut from the stock material used to form each profile. After the forming process, six additional specimens were cut from the center of flat plate elements and an additional six at the curved portion between these elements. All specimens were tested according to the Standard ISO procedure [3]. A more detailed review of the test procedure and the test specimens is contained in [6].

Test series 1. The procedure adopted for conducting these tests was a modification of the ECCS Recommendations [2]. The blockings, required by ECCS between all ribs under concentrated loads, were provided only at exterior ribs. This change was made for two reasons. Firstly, it is difficult to determine the distribution of the applied load on the specimen with more than two blocks and secondly, using only two blocks, it is much easier to assemble the test.

For the second test program, a 175 mm (6.89 in.) length of profiled sheeting, the same profile as the specimen tested, was placed beneath applied loads. This short length of profile was attached to the specimen by a number of screw fasteners placed near both edges transverse to the applied load. Its placement insured that premature shear failure near the concentrated loads would not occur.

Test series 2. The ECCS Recommendations were followed for this test series. Blocking is not required as failure occurs at the location of the applied load.

Test series 3. This series resembles test series 1. However, instead of exterior blocking beneath the concentrated loads, a 150 to 175 mm (5.91 to 6.89 in.) length, depending upon profile height, of profiled sheeting was placed between the specimen and the transverse spreader beam. This profiled sheet was the same shape and thickness as the specimens tested.

2.3 Test results

Series 1. The ultimate load of the test specimens were compared to the ultimate load calculated using the ECCS Recommendations. This comparison is shown in Table 1. A very good correlation between test results and calculated values was observed for the first test program; the maximum difference between theoretical and test values of ultimate moment was 5 %. For the second test program, the maximum difference between theoretical and test values was 19 %.

Series 2. An interaction diagram of moment and support reaction at ultimate load, calculated using the ECCS Recommendations, is shown in Figure 3. Test results are also presented on this figure. For the first test program, the maximum difference between theoretical and test values of ultimate moment was 9 %, the standard deviation being 0.045. For the second test program, these two values are, respectively, 25 % and 0.134.

Series 3. Ultimate loads from these test specimens and theoretical values calculated using the ECCS procedure are presented in Table 2. The theoretical values of ultimate load do not correspond to experimental results for both test programs. For the first test program, the maximum difference between theoretical and experimental values was 37 % and the minimum difference was 21 %. For the second test program, these values were 29 % and 39 %. The test values of ultimate load were always larger than those predicted by the ECCS procedure. The difference between the experimental ultimate load and the ultimate load predicted by ECCS represents the reserve capacity due to moment redistribution near the interior support. The redistribution factor, α, is defined as the reserve capacity of the specimen divided by the ultimate load predicted by ECCS.

3. ANALYSIS

The following analysis is used to determine the redistribution factor, α, for multiple-span cold-formed profiles using a semi-analytical procedure. Based on an elastic analysis, the compatibility equation at ultimate load for a beam with two equal spans, L, uniformly loaded, is given by :

$$\frac{M_o\ L}{3\ EI_o} = \frac{p_o\ L^3}{24\ EI_o} \tag{1}$$

M_o : moment at the interior support of a two span continuous beam assuming linear behaviour; this is calculated using the simple beam formula and the yield stress of the material,

I_o : moment of inertia of the entire cross section,

L : single-span length, both span lengths equal,

E : modulus of elasticity,

p_o : uniform applied load at M_o.

In equation (1), the left-hand term represents the end rotation in a simply supported span due to a single end moment, M_o. The right-hand term represents the end rotation in a simply supported span uniformly loaded. By equating these two components, an expression for the moment at the interior support of a two-span beam is obtained. To account for the additional capacity observed during testing, equation (1) may be rewritten to include the non-linear components of rotation at the interior support. Thus, this new expression is written as follows :

$$\frac{M_o\ L}{3\ EI_o} + \Delta\theta_{el} + \Delta\theta_p = \frac{p_u\ L^3}{24\ EI_o} \tag{2}$$

$\Delta\theta_{el}$: rotation at the interior support due to local buckling of individual flat plate elements,

$\Delta\theta_p$: rotation at the interior support due to the presence of a concentrated support reaction; this rotation is permanent,

P_u : uniform applied test load at failure or predicted ultimate load.

In expression (2), the left-hand terms represent three separate components of rotation. The sum of these three components equals the rotation which occurs at the interior support in a cold-formed profile. To define these three components of rotation, illustrated in Figure 4, the following assumptions are made :

- the negative-moment region near the interior support of a two span uniformly loaded specimen can be modelled by a single-span beam with a concentrated load at midspan,
- the effect of concentrated load is so localised that the magnitude of $\Delta\theta_p$ is independent of the span length.

Let us consider typical load-midspan deflection curves of a single-span beam with a concentrated load at midspan, as shown in Figure 5. Theoretically, a compact cross section with no initial imperfections will attain overall plasticity without instability taking place and therefore, complete moment redistribution. This behaviour is identified by curve OAB. In reality, both initial imperfections and local buckling cause failure at a lower applied load than that predicted by OAB. Curve OKN represents this behaviour. When a concentrated load is present at the same location as the maximum moment, a further reduction in capacity is observed due to web crippling. This is shown by curve OEF. As loading is increased, four different types of behaviour are predicted by curve OEF. In the first region, OC, linear behaviour prior to local buckling is observed. In region CD, nonlinearity, primarily caused by local buckling in the different flat plate elements of the specimen, is observed. In region DE, the effects of the concentrated load dominate behaviour. At point E, failure of the entire section occurs. Curve GJF is typical of the post-elastic failure behaviour of cold-formed sheeting. The nature of this curve has been investigated by several researchers [4] [5] [7]. Schardt [5], for example, has used a straight line. Unfortunately, this curve is difficult to obtain by theoretical means for cold-formed sheeting.

To obtain $\Delta\theta_{el}$ and $\Delta\theta_p$, the following procedure is proposed : rotation due to buckling of individual plate elements is dependent upon moment gradient, span length and moment-curvature relationship. Using the moment-curvature relationship from test series 1 and a finite difference model, $\Delta\theta_{el}$ can be plotted as a function of span length, for each sheeting. The magnitude of this component of rotation in terms of midspan deflection of the single-span beam with a concentrated load is shown in Figure 5 as HI. Rotation due to support reaction is localised and independent of span length. Thus, $\Delta\theta_p$ is determined by the non-linear rotations taken from testing, provided that the span length is sufficiently short to insure that elastic local buckling of the flanges does not occur. For the sections tested, this length is less than ten times the depth of the section. Component HI approaches zero and the remaining non-linear component of mid-span deflection is represented by IJ in Figure 5.

Using this procedure, two different tests obtain the two components of rotation, $\Delta\theta_{el}$ and $\Delta\theta_p$. These rotations are used to compute the ultimate load capacity of multiple-span profiled sheeting. The first test, series 1, establishes the moment-curvature characteristics of the section subject to bending moment alone. The second test, a small span with a single concentrated load, determines the effects of concentrated reaction. The non-linear components of rotation at the interior support can thus be expressed as :

$$\Delta\theta_{el} + \Delta\theta_p = \alpha \frac{p_o L^3}{24 EI_o} \tag{3}$$

Using equations (1) and (2) the ultimate load capacity, p_u, is expressed as :

$$p_u = (1 + \alpha) p_o \tag{4}$$

Values of α have been calculated using this procedure for the sections tested. These values are compared to the experimentally determined redistribution factors and are listed in Table 3. Conservative values for the redistribution factor are obtained, the average difference being 23 %.

4. DISCUSSION

It is instructive to compare the ECCS and AISI design codes. Philosophically, these two codes have one major difference. AISI uses working stress design, whereas the ECCS Recommendations has adopted an approach which is similar to LRFD (Load and Resistance Factor Design).

Otherwise, for profiled steel sheeting, the two codes have similar characteristics. The following points are the major exceptions :

- The AISI formulae for the effective width or effective area of compression flanges with intermediate stiffeners are based upon test results performed by Winter [8]. The ECCS method analyses such elements as a beam on an elastic foundation.
- Both AISI and ECCS have similar web crippling formulae. However, the ECCS formula does not include a term accounting for web slenderness (web height, s_w, divided by thickness, t).
- In the ECCS method, the von Karman effective width concept is applied to the compression zone of the web, in addition to the compression flange, when calculating the bending resistance. AISI limits web stress when calculating the bending resistance.
- The AISI code designates different safety factors for different members, e.g. 1.85 for web crippling and 1.67 for bending resistance, etc. In the ECCS Recommendations, safety factors do not change according to the element.

Bearing these in mind, the method proposed in this article could not integrate directly in AISI code. Further study would be appropriate if LRFD is adopted by AISI.

Test Series 1. Single-span test results and computed values for the second generation profiled sheeting are more scattered than those of the first generation profiled sheeting. This is due to two main factors :

1.- Nominal cross-sectional dimensions were used in calculations for the second generation; measured values were used for the first generation.

2.- The modeling of the intermediate web and flange stiffeners introduce deviations of predicted values for second generation profiled sheeting. Furthermore, the effective moment of inertia and cross-sectional area of these stiffeners are difficult to define accurately and may change as load is applied.

In general, the ECCS design method successfully determines the ultimate capacity of single-span profiled sheeting. However, it underestimates the limits of linear behaviour.

Test Series 2. Span lengths were chosen in order to obtain test results in the general vicinity of $M/M_u \leqslant 0.25$. As presented in Figure 3, this ratio corresponds to the ultimate resistance of the section when web crippling is the limiting mode of behaviour. Very few test results are available for this region. When such small span lengths are provided, the section behaves as though it is supported by an elastic foundation. In this analogy, the rigidity of the elastic foundation models the buckled rigidity of the webs. The results from this test series illustrate that ECCS values vary widely from observed behaviour. More realistic models of web crippling need to be developed. In the absence of such a model, and when failure is due to web crippling near a concentrated load, it is recommended that tests be performed to determine the ultimate resistance.

Test Series 3. The differences in reserve capacity, between predicted and test values of α (Table 3) are due to several factors. Presently it is believed that the following two components account for the majority of these differences :

1.- Although the same sheeting was used for test series 1, 2 and 3, dimensional differences between specimens exist. M_o was calculated using the measured section properties of the sheeting used in test series 1. Some difference in section properties is always present due to the flexible nature of these specimens.

2.- The deformation of the sheeting at the interior supports of a multiple-span specimen is larger than that observed under the concentrated load of test series 2 specimens.

Also, it should be noted that all tests were conducted on single panel widths of profiled sheeting. The exterior flanges and webs had greater freedom than they would have had if continuity was provided. This lack of continuity reduces the average load that can be applied to the section before failure occurs. Therefore, redistribution factors may be marginally higher in multi-panel systems. In addition, the sheeting is fastened at the supports in practice. The rotational capacity seems not to be influenced due to the nature of failure mechanism.

Since all our tests are carried out with 100 mm (3.94 in.) support length, more studies are needed in the field to clarify the effect of support length. Codes predict an increase of web crippling resistance when the support length is larger. Some supplementary tests showed that the rotational capacity seemed not to be affected by the same degree when the support length is increased.

To illustrate the effect of this redistribution factor, a load table can be established. For example, see Table 4. This table compares the designed values calculated by ECCS method with the proposed method. The table shows that the longer the span, the smaller the increase. Further work is needed to verify this trend.

5. CONCLUSIONS

The following conclusions are presented :

1.- Variations between measured and nominal dimensions for profiled sheeting can be substantial. To improve accuracy, actual section dimensions should be used when analyzing test data.

2.- Use of the present ECCS Recommendations results in successful prediction of the ultimate load capacity of single-span systems constructed using first generation profiled sheeting.

3.- The ECCS procedure underestimates the ultimate load capacity of both first and second generation continuous steel profiled sheeting by 21 % to 39 % because rotational capacity at interior supports is not considered.

4.- The rotational component at support, $\Delta\theta_p$, accounts for between 39 % and 79 % of the total rotation observed at the interior support for both the first and second generation steel profiled sheeting.

5.- The semi-analytical procedure outlined in this paper provides a conservative means of predicting ultimate strengths of multiple-span profiled sheeting. This procedure improves the accuracy of LRFD concepts.

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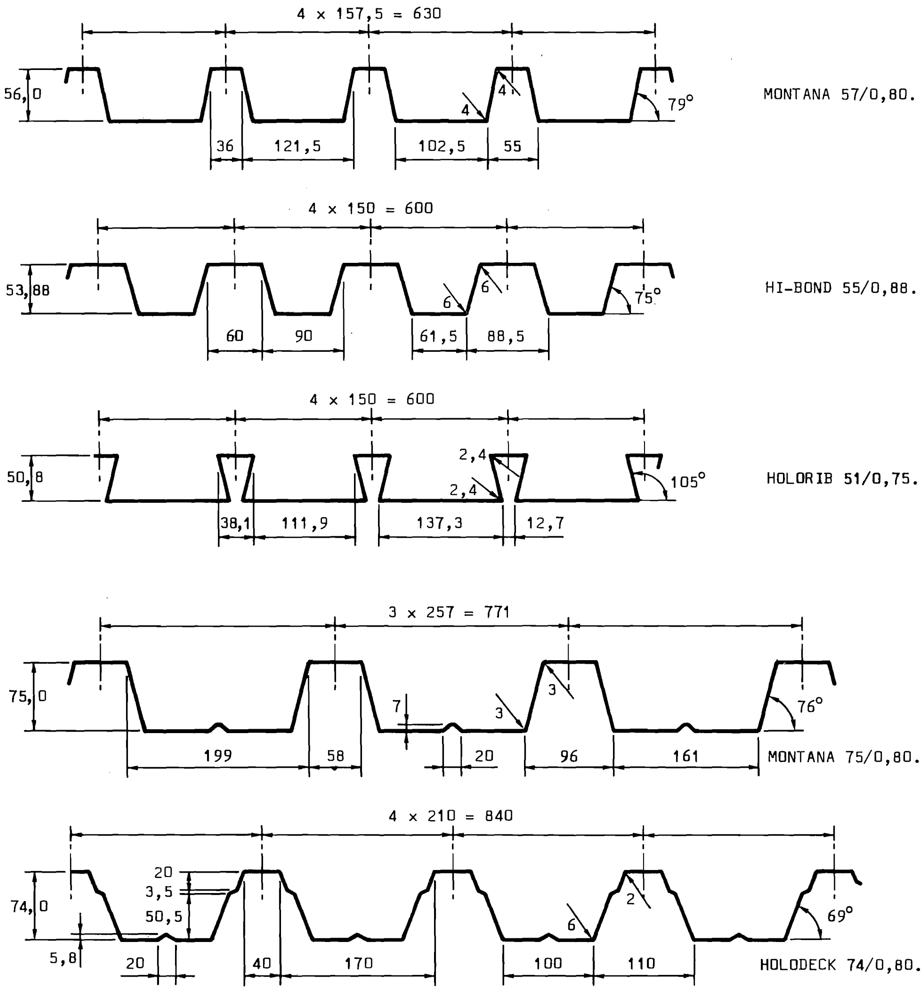
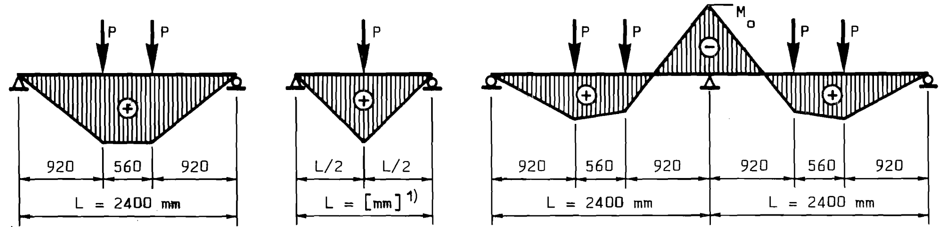


FIGURE 1

Profiled sheeting sections (1 in. = 25.4 mm).



1) Test program 1 : L = 560, 720, 1040, 1200, 1520, 2000 mm

Test program 2 : L = 275, 350, 900, 1200, 1600 mm

M_o = 0,355 P L

a) Series 1. b) Series 2. c) Series 3.

FIGURE 2

Static system for the tests (1 in. = 25,4 mm).

	PROFILE TYPE	TEST NUMBER	STATIC SYSTEM: $M_u^+ = 0{,}92\ P_u$		STATIC SYSTEM: $M_u^- = 0{,}92\ P_u$	
			$M^+_{u,series1}$	$M^+_{u,ECCS}$	$M^-_{u,series1}$	$M^-_{u,ECCS}$
TEST PROGRAM 1	MONTANA 57/0,80 f_y = 294 N/mm^2	MO22	3,243	3,321		
		MO21			3,114	3,124
		MO23			3,197	3,085
	HI-BOND 55/0,88 f_y = 306 N/mm^2	HI22	3,340	3,485		
		HI21			3,257	3,227
		HI23			3,243	3,163
	HOLORIB 51/0,75 f_y = 291 N/mm^2	HO23	3,229	3,257		
		HO21			2,995	3,106
		HO22			2,967	2,969
TEST PROGRAM 2	MONTANA 75/0,80 f_y = 276 N/mm^2	MP22	3,137	3,660		
		MP21			4,030	4,077
		MP23			3,956	4,050
	HOLODECK 74/0,80 f_y = 335 N/mm^2	KH22	5,130	4,770		
		KH21			5,722	5,570
		KH23			5,786	5,580

(1 ksi = 6,895 N/mm^2)

TABLE 1

Test results of series 1 in kNm/specimen (1 kip-ft = 1,356 kNm).

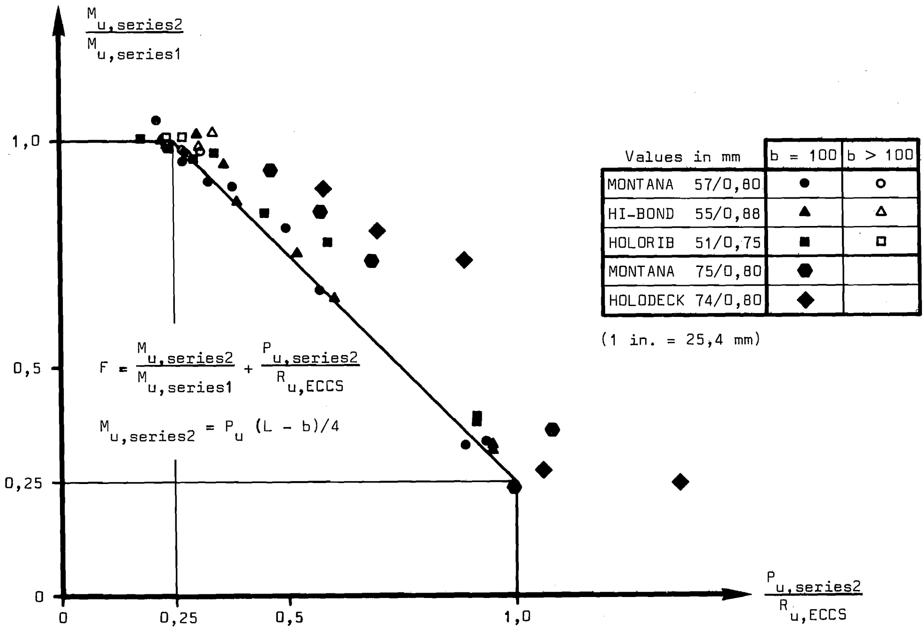


FIGURE 3

M-R interaction diagram.

PROFILE TYPE	f_y [N/mm^2]	$M^-_{u,series2}$ 1) [kNm/specimen]	$P_{u,series2}$ 2) [kN/specimen]	$P_{u,series3}$ 3) [kN/specimen]	$\alpha = \frac{P_{u,series3}}{P_{u,series2}} - 1$
MONTANA 57/0,80	294	2,871	3,370	4,264	0,265
HI-BOND 55/0,88	306	3,094	3,631	4,409	0,214
HOLORIB 51/0,75	291	2,868	3,366	4,623	0,373
MONTANA 75/0,80	276	3,369	3,954	5,120	0,295
HOLODECK 74/0,80	335	4,612	5,413	7,530	0,391

1) The ultimate moment is given by the interaction moment-reaction diagram in series 2 test.

2) $P_{u,series2} = M^-_{u,series2}/0,355\ L$ (elastic linear solution).

3) Average values of tests.

(1 ksi = 6,895 N/mm^2, 1 kip-ft = 1,356 kNm, 1 lb = $4,45 \cdot 10^{-3}$ kN)

TABLE 2

Test results of series 3 and redistribution factor.

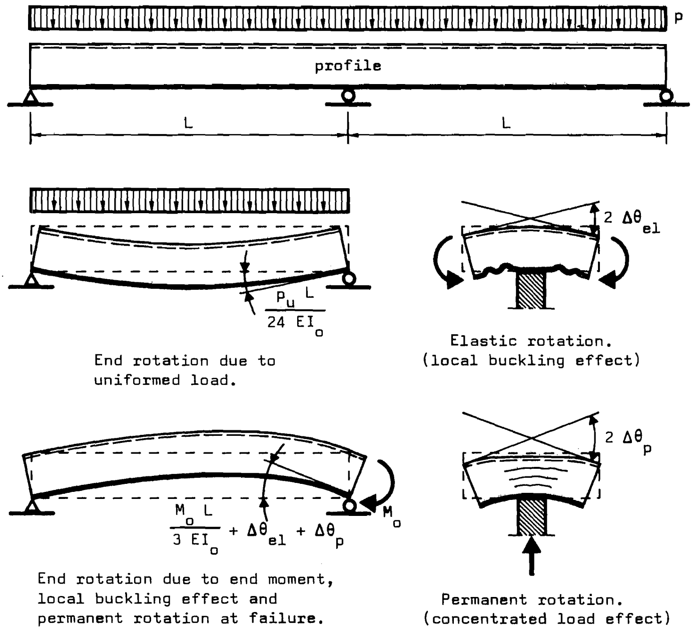


FIGURE 4

Components of compatibility equation (2).

PROFILE TYPE	$I_o \cdot 10^6$ [mm^4/specimen]	$\Delta\theta_{el} \cdot 10^{-3}$ [rad]	$\Delta\theta_p \cdot 10^{-3}$ 1) [rad]	α_{cal} 2)	α_{test}
MONTANA 57/0,80	0,386	3,31	3,75	0,228	0,265
HI-BOND 55/0,88	0,389	2,62	3,90	0,205	0,214
HOLORIB 51/0,75	0,420	3,72	3,93	0,284	0,373
MONTANA 75/0,80	1,020	–	2,95	0,234	0,295
HOLODECK 74/0,80	0,869	–	4,12	0,204	0,391

1) $\Delta\theta_p$ is obtained from a 560 mm span series 2 test

2) Calculated from equation (3)

(1 $in.^4 = 4,16 \cdot 10^5$ mm^4)

TABLE 3

Comparison of the tested and calculated redistribution factor.

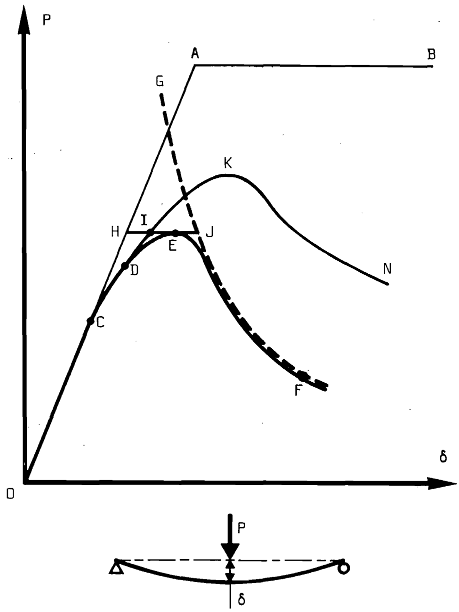


FIGURE 5

Typical load-midspan deflection curve.

SPAN [mm]	2500	3000	3500
ALLOWABLE LOAD WITHOUT REDISTRIBUTION [kN/m^2]	3,27	2,45	1,92
ALLOWABLE LOAD WITH REDISTRIBUTION [kN/m^2]	3,90 (α = 0,193)	2,81 (α = 0,150)	2,15 (α = 0,120)

(1 in. = 25,4 mm, 1 kip/ft^2 = 47,88 kN/m^2)

TABLE 4

Load table of MONTANA 75/0,80, double span with uniform loading. Comparison between ECCS and proposed methods (safety factor : 1,5 ; support width : 100 mm).