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Ductility in support regions of continuous composite beams

Ductilité dans les zones d'appuis des poutres mixtes continues

Duktilität im Auflagerbereich von durchlaufenden Verbundträgern

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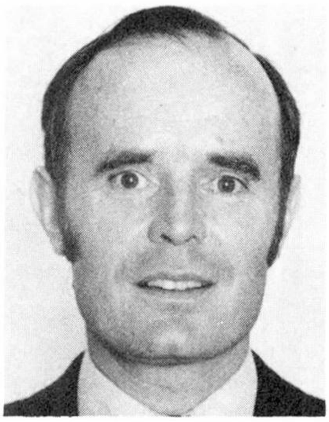

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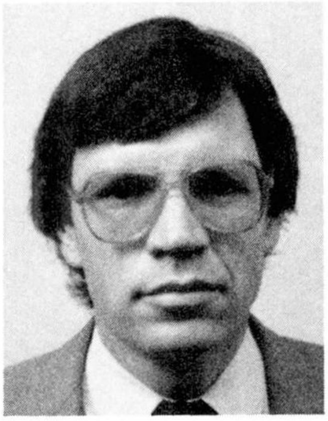

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SUMMARY

Two solutions are investigated for improving the ductility of support regions of continuous composite beams to permit plastic design without uneconomic prescriptions due to possible local web and flange buckling. The behaviour is assessed in three beam tests and compared with theoretical predictions. The economic implications are evaluated by comparitive cost studies. The benefits of a compositely-continuous, simply-supported beam are described.

RÉSUMÉ

Deux solutions sont recherchées pour augmenter la ductilité et la stabilité des zones d'appuis des poutres mixtes dans le but d'assurer la plastification totale de la section – donc d'éviter le voilement de l'aile ou de l'âme – sans avoir recours à des dispositifs trop coûteux. Le comportement de ces zones d'appuis est évalué à l'aide de trois essais de poutres et de leur comparaison avec les prévisions théoriques. Des études de comparaison de coûts permettent de prévoir les implications économiques de ces découvertes. Les gains qu'on obtiendrait avec des poutres mixtes continues, par rapport au poutres simples, sont estimés.

ZUSAMMENFASSUNG

Zwei Lösungen zur Verbesserung der Duktilität im Auflagerbereich von durchlaufenden Verbundträgern werden untersucht, damit plastische Berechnungsmethoden angewendet werden können, ohne dass unwirtschaftliche Vorschriften aufgrund des möglichen, lokalen Flansch- und Stegbeulens auferlegt werden. Das Verhalten wird mit 3 Balken-Tests belegt und mit theoretischen Voraussagen verglichen. Die wirtschaftlichen Folgen werden durch Kostenvergleiche bewertet. Die Vorteile eines einfach gelagerten, durchlaufenden Verbundträgers werden beschrieben.

1. INTRODUCTION

It is well established that continuous composite beams provide an efficient and economic solution in building structures, particularly if they are designed to develop their plastic moment capacities in both sagging (positive moment) and hogging (negative moment) regions. However, it is also recognised that in order to develop the plastic collapse mechanism in such continuous beams a large amount of redistribution of moment is necessary between the support and midspan regions. This implies a requirement for adequate ductility in hogging moment regions as reflected by the plastic plateau in representative moment-rotation curves (Fig.1). This requirement is particularly demanding in the case of composite beams.

Local buckling as described subsequently and, to a lesser extent, lateral buckling of the steel section critically influence the ability of composite beams in support regions to maintain their moment capacity during the hinge rotations which are necessary to develop a collapse mechanism. These modes of failure are particularly important in limiting the ductility because of the concurrent axial compressive force applied to the steel beam which balances the tension force in the longitudinal reinforcement in the slab. The applicability of simple plastic theory to continuous composite beams has been reviewed by Johnson [1] and assessed in quantitative terms by Johnson and Hope-Gill [2].

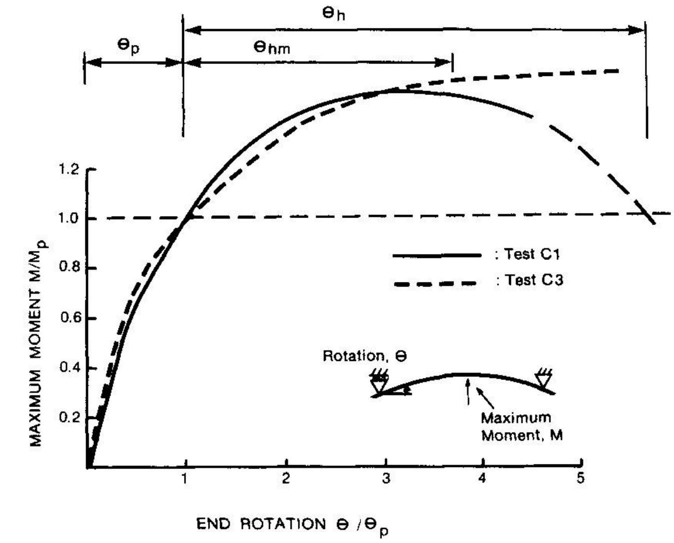


Fig. 1 Moment-rotation curves

Two solutions for improving ductility are described; the first incorporates the use of inclined stiffeners and the second involves designing and detailing a composite beam so that it is simply-supported prior to the concrete achieving its characteristic strength and continuous subsequently. The project was directed towards solving a hypothetical design problem and the cost implications are also evaluated and discussed.

2. THEORETICAL CONSIDERATIONS

2.1 Ductility

The two-span continuous beam illustrated in Fig. 2a provides the terms of reference in this paper for considering ductility, local buckling and cost comparisons. The limiting elastic and ultimate bending moment diagrams are shown in Fig. 2b and are based on the following assumptions:

- Ratio of flexural regidities (EI) in hogging (cracked) to sagging regions = 0,4
- Ratio of plastic moment capacities in hogging to sagging bending = 0,7

Once the moment at the support reaches its ultimate capacity, $M'_e = M'_p$ in the idealised elastic bending-moment diagram, a redistribution of moment is required from support to midspan region in order to develop the ultimate bending-moment diagram. This redistribution can be achieved if a sufficiently long, plastic-moment plateau exists in the relationship shown in Fig. 1 between support moment and rotation at the end of the hogging moment region (length L_h in Fig. 2). This ductility is normally assessed by the rotation capacity, R, which is defined as follows with reference to Fig. 1:

$$R = \frac{\Theta_h}{\Theta_p} \quad \text{or} \quad R_m = \frac{\Theta_{hm}}{\Theta_p}$$

2.2 Local Buckling

The inverted simply-supported beam arrangement shown in Fig. 3 forms the basis of the experiments described subsequently and represents the hogging region of length $2L_h$ between points of inflection of the beam shown in Fig. 2. Local buckling of the compression flange is the most significant cause of strain-weakening behaviour in continuous beams. Following the proposals of Lay and Galambos [3] , the local buckle is assumed to develop when the length of the yielded region of the flange (L_p in Fig.3) extends sufficiently far to accommodate the full wave-length of the buckle. Kemp [4] has rearranged the formulations and assumptions of Lay and Galambos [3,5] Southward [6] and Stowell [7] to give the following formulae which can be solved iteratively to give the ratio of yielded to half-span length $\ell_f = L_{pf}/L_h$, at the onset of local flange buckling in regions of moment gradient:

$$\left(\frac{b}{t_f}\right)^2 = \frac{4}{3\varepsilon_b - n_1\left(\dfrac{\pi t_f}{\ell_f L}\right)^2} \qquad (1a)$$

in which (b/t_f) is the ratio of flange width to thickness, $n_1 = 1$ for no web restraint (coincident web buckling) or $= 2$ for web providing restraint and ε_b is the longitudinal strain at buckling in the compression flange at the centre of the buckled length, given by:

$$\varepsilon_b = \varepsilon_y \ \{s + 0{,}5e\ell_f \ / \ (1-\ell_f)\} \qquad (1b)$$

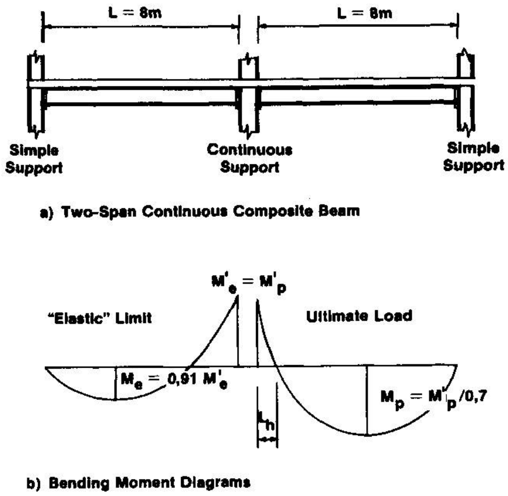
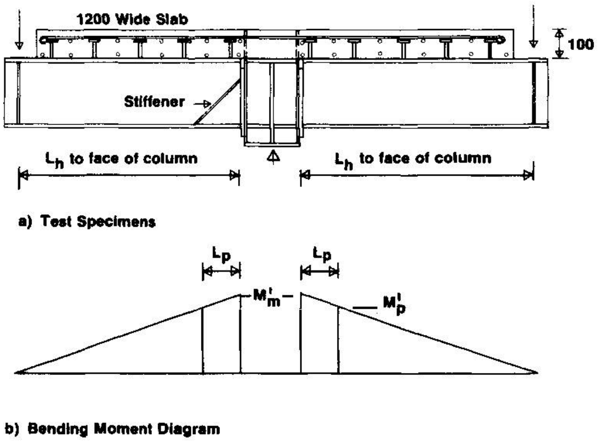


Fig. 2 Illustrative beam arrangement.



Test	C1	C2	C3
Size of steel section	305x102x25	305x165x41	305x102x25
Area of longit. reinforc.	470mm²	470mm²	940mm²
No. of studs/half-span	5	5	9
Detail of beam end plate at column connection (Grade 8,8 Bolts)	Rigid Continuous	Rigid Continuous	Flexible No tension transfer

Fig. 3 Test specimens

in which ε_y is the yield strain, s is the ratio of strain at the onset of strain-hardening to yield strain and e is the ratio of modulus of elasticity to strain-hardening modulus.

Web buckling is predicted to occur [8] when the ratio of yielded to half-span length, $\ell_w = L_{pw}/L_h$, extends far enough to satisfy the following relationship:

$$\ell_w = 27 \frac{t_w}{L_h} \{(2n^2_{wc} -1) - \sqrt{(2n^2_{wc} -1)^2 -1}\}^{0.5} \sqrt{240/f_y} \qquad (2)$$

provided $h_{wc} > 27\, t_w \sqrt{240/f_{sd}}$

in which $n_{wc} = h_{wc}/27t_w \sqrt{240/f_y}$, h_{wc} is the clear depth of web in compression, t_w is the web thickness and f_y is the yield stress.

Kemp [8] has identified in tests on plain steel beams that local flange buckling may be initiated under the conditions defined by Eqs. 1, but may not develop due to strain compatibility constraints across the flange, which introduce inhibiting forces. These are released by the onset of web buckling or lateral torsional buckling. A combined mode of local flange buckling and web buckling may therefore be expected to occur when the ratio of yielded to half-span length, $\ell_b = L_p/L_h$, is equal to the larger of ℓ_f (Eqs. 1 with $n_1 = 1$ for no web restraint) or ℓ_w (Eq. 2). This model has been shown [8] to compare favourably with test results, using the following simplified relationship between rotation capacity at maximum moment and plastic length ratio:

$$R_m = \frac{\theta_{hm}}{\theta_p} = \frac{h_w}{2h_{wc}} \ell_b \left(2s - 1 + \frac{e\ell_b}{1-\ell_b}\right) \qquad (3)$$

3. COMPOSITELY-CONTINUOUS, SIMPLY-SUPPORTED BEAMS

Code requirements to avoid local buckling of the web of continuous composite beams often lead to uneconomic steel sections due to the extended depth in compression which is required to balance the tension force in the longitudinal reinforcement. The need for such limitation has also been demonstrated in tests on plain steel sections [8] .

The authors decided to investigate the implications of applying to building structures the concept of "compositely-continuous, simply-supported beams", which was suggested by Fried [9] for bridges in Australia. These beams possess simple, welded end-plates for connection to the column with no provision for continuity of the top, tension flange but the facility for compression force to be transmitted in bearing from the compression flange and web, with the possible need for shim plates to allow for fabrication tolerances and erection clearances. Such a beam will behave as simply-supported prior to the concrete achieving its strength and continuous when the longitudinal slab reinforcement becomes effective. This procedure effectively upgrades the resistance of a simply-supported beam by using the longitudinal reinforcement provided over the support, which may only be the minimum amount required to control cracking. The reinforcement should be staggered and extend far enough to allow for both elastic and plastic positions of the point of inflection. This arrangement was studied experimentally and assessed in terms of relative cost.

3.1 Experimental Results

Three beams, C1 to C3, were tested in the arrangement shown in Fig. 3 in order to assess the ductility requirements further. These specimens were intended to represent the hogging region of the two-span continuous beam of Fig. 2 and were deliberately chosen to reflect relatively slender sections. The details of the test specimens are given in Fig. 3 and Table 1. Beams C1 and C2 possess conventional rigid end-plate connections appropriate to a continuous beam, whereas beam C3 is a compositely-continuous, simply-supported beam as described above with sufficient longitudinal reinforcement to give approximately the same moment capacity as beam C1.

The relationship between moment at the face of the column and the relative rotation between the column and end of the beam is shown in Fig. 1 for beams C1 and C3. Whereas C1 failed in combined local buckling of web and flange at a rotation capacity of 3,5, specimen C3 exhibited significantly larger rotations and the test was stopped due to failure of the slab in horizontal shear. Beam C1 just satisfied the intention in the South African code [10] of $R_m > 3$. Beam C3 more than met this requirement due to the greater flexibility in the simply-supported end connection. Furthermore local buckling was less likely in this case due to the smaller proportion of web depth in compression.

The observed plastic rotations, Θ_{hm} in Fig. 1, are recorded in Table 1 for the beam and end connection to the column, as well as for the beam alone. It is apparent that the rotation capacity is approximately doubled by the flexibility of the end connection. This beneficial effect is difficult to quantify for designs in general and has partly been allowed for in the relatively low rotation capacity required for plastic design (eg. $R_m \geq 3$). The plastic rotations predicted using the theoretical models of Eqs. 1 to 3 are given in the last line of Table 1 and compare favourably with the observed behaviour. The plastic rotations are described rather than the rotation capacities due to the difficulty in consistently identifying EI and thus Θ_p in the test beams due to large rotations in the end connection and variable amounts of concrete cracking. The beneficial effects of the diagonal stiffener (Fig. 3) which inhibits local buckling of the flange and web can be seen in the results of beam C2. Investigations are proceeding on how to quantify this benefit.

Test		C1	C2		C3
flange slenderness b/t_f		15,4	16,5		15,3
flange yield stress		378	341		378
flange length ratio L/t_f (Eq. 1a)		141	101		141
web slenderness n_{wc} (Eq. 2)		1,23	1,28		0,3
web yield stress		412	359		412
diagonal stiffener (Fig. 3)		NO	NO	YES	NO
Plastic rotation $\Theta_{hm} \times 10^{-3}$	observed : beam & end connection	22,6	29,9	47,2	>34,3*
	observed : beam only	12,0	14,8	23,0	> 9,7*
	predicted : beam only	12,8	16,6	n.a.	18,8

(*: beam C3 did not fail in web & flange buckling)

Table 1 Local buckling characteristics

3.2 Cost Implications

In order to determine how the requirements for ductility of the hogging moment region influence the cost of construction, the structure of a four storey building with beams spaced at 3m and supported on three rows of columns as shown in Fig. 2a was designed for clear beam spans L of 8, 10 and 12m. For each value of beam span, three different joint details were considered : simply-supported, compositely-continuous, and fully-continuous. At the two exterior columns the beams were assumed to be simply-supported. These structures were designed for each of two assumptions: steel beams propped until concrete has hardened, and steel beams unpropped from the start. The assumption was made that before hardening of the concrete the top flange of the steel beam is fully-restrained against lateral torsional buckling. Various steelwork contractors were asked to price each of the design variations.

The results of this study are shown in Table 2, where the cost/m² of the 8m span, simply-supported, propped beam construction was taken as unity. The cost/m² of floor relates only to those aspects which are not common to all beams: the steel beams with connections, shear connectors, reinforcing in the hogging moment region, and the cost of erection. The cost or nuisance effect of propping was not taken into account, the former is minor and the latter difficult to assess in general terms.

Span L of beam	8m	10m	12m
Simply-supported beams, propped:			
section	305x102x33 I	406x140x46 I	406x178x60 I
no. of stud connectors	32	24	64
cost/m² of floor	1,00	1,13	1,40
Compositely-continuous beams, propped:			
section	305x102x29 I	406x140x39 I	406x178x54 I
no. of stud connectors	22	27	34
mass of reinforcement (kg)	10,6	11,5	23,3
cost/m² of floor	0,94	1,11	1,31
Fully-continuous beams:			
section	305x102x29 I	356x171x45 I	406x178x54 I
no. of stud connectors	20	26	32
mass of reinforcement (kg)	3,5	5,4	7,8
cost/m² of floor	1,02	1,31	1,41
Simply -supported beams, unpropped:			
section	406x140x39 I	406x140x46 I	406x178x160 I
no. of stud connectors	20	24	64
cost/m² of floor	1,11	1,13	1,40

Table 2 Summary of design solutions

Table 2 shows compositely-continuous propped beams to be the least-cost solution for each of the three spans, assuming the cost of propping to be negligible. The question as to the performance of unpropped, compositely-continuous beams is answered by the fact that the steel sections would be the same as the simply-supported, unpropped beams for which the temporary criterion of supporting wet concrete and construction loads was found to be the limiting condition for the spans and loading under consideration. Propping (at low cost) is thus a requirement for the success of compositely-continuous beams in this study, in which the construction loads were 52% of the final value of dead plus live load. Further investigations, the results of which are not quoted in this paper, indicated that for certain realistic but lower values of the ratio of dead to live load, unpropped compositely-continuous beams may be used without having to pay any penalty in the weight of the steel section.

Fully-continuous beams proved in this study to be uneconomical, although the uncertainty of the cost of propping is avoided through the fact that deletion of the propping was found not to affect the choice of the steel section. A striking observation is that in the 10m span case a heavier section had to be chosen than for the compositely-continuous counterpart, in order to satisfy the codified web buckling provisions.

It needs to be said that partial interaction between steel and concrete was assumed where the strength of a full-interaction beam exceeded requirements, thus saving on the number of shear connectors needed. Furthermore, limitations in the range of available steel sections led on occasions to slightly unrepresentative results.

4. DISCUSSION

This study, including the experimental work, showed that local web buckling is significant in the design of continuous composite beams, although solutions have been proposed [11] which make no reference to these ductility requirements. It is further clear that satisfying the ductility requirements in continuous beams has cost implications, as relatively slender sections are not suitable for plastic design. The compositely-continuous beam as described above, is however, an economic way of providing continuity without ductility problems, provided either that the beams are propped until the concrete has hardened, or that the live loads are relatively high in comparison with the construction loads. The improved ductility of a compositely-continuous beam over a fully-continuous member with the same moment of resistance is due to the fact that less web depth is under compression, and these beams exhibit considerably larger rotations within the end connection between the beam and the column. This is illustrated by the observed rotations in beam C3 in Fig. 1.

Taking the importance of speed of construction into account, the unpropped simply-supported beam remains a viable alternative. There are, however, good reasons to favour compositely-continuous beams:

- they are less expensive (up to 6% in this evaluation);
- cracking has been experienced in the support regions of simply-supported beams [12] and reinforcing is recommended; thus the only remaining requirement to create compositely-continuous conditions is to ensure contact between the compression region of the beam and the column face, which contact need not be extremely tight for the unloaded beam;
- the deflection and propensity for objectionable vibrations of the floors will be reduced with continuous construction.

5. CONCLUSIONS

Codes for plastically-designed continuous composite beams should contain prescriptions on web slenderness in hogging regions which allow for the depth of web in compression. These provisions will often increase the cost of the steelwork. The use of the compositely-continuous beams is shown to satisfy these ductility requirements and provide an economic solution which merits further attention.

6. ACKNOWLEDGMENTS

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