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Ductile Steel-Concrete Composite Joints

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Summary

The paper treats the ductility requirements for partial-strength beam-to-column joints in composite construction. The slab reinforcement enhances the resistance of the joint but may limit the rotation capacity. The ductility needed to permit formation of a beam-type plastic hinge mechanism is examined and proposals are made for design. The sources of ductility in composite joints are also examined, and a method to predict the limiting rotation capacity of end-plate joints is demonstrated.

1. Introduction

Ductility is needed in continuous and semi-continuous construction if in-elastic methods are used for global analysis. These include approaches based on the attainment of a mechanism of plastic hinges. The need for ductility at hinge locations is expressed in terms of "rotation capacity", being the localised rotation to achieve the distribution of moments used in design. With partial-strength joints, the rotation must be accommodated by the joint itself, because it is components within the joint, rather than the adjacent members, which behave inelastically. Thus the rotation capacity of the joint becomes one of the key properties. For design, it is the capacity φ_{cd} available at the design moment resistance M_{cc} (Fig. 1). To deliver the necessary rotation capacity, some component(s) must yield in a dependable manner. For steelwork joints, appropriate components are plate elements in bending and column webs in shear.

Compared to bare steelwork, composite joints provide greater moment resistance; in a braced frame, through the tensile action of the slab reinforcement. However, they usually remain partial-strength relative to the composite beam, and tests show that the rotation capacity at the enhanced moment is reduced (Anderson (ed), 1997). The rotation capacity is curtailed by several possible modes of failure; some are peculiar to composite joints; others are more likely because the tensile action of the reinforcement increases the balancing compression around the bottom flange of the steel section, thereby encouraging buckling.

This paper examines the influence of failure modes on the rotation capacity of composite joints for braced frames, and indicates how this property can be predicted. The requirements though concern the rotation needed for the assumed distribution of moments. These needs are addressed first.

2. Required rotation capacity

2.1 Analysis methods

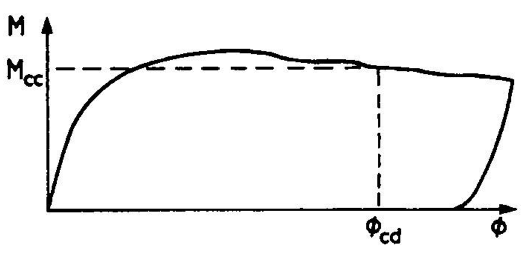

Fig. 1 :Rotation capacity

Numerical methods have been developed to determine the requirements (Najafi, 1992; Li et al, 1995). These were developed independently; comparison of results therefore strengthens confidence in both approaches and provides evidence to judge the appropriateness of simple plastic design for frames including partial-strength composite joints. The analysis due to Najafi is now outlined.

The analysis extends one by Johnson for fixed-ended composite beams, by including semi-rigid and/or partial-strength joints. Moment-curvature relationships for the beam's cross-section in hogging and sagging bending are determined from geometric data and material properties. An iterative procedure is then followed to determine a bending moment distribution which satisfies equilibrium and, through the moment-curvature relationships, ensures compatibility. The shear connection is assumed to provide full interaction, and plane sections are assumed to remain plane. The stress-strain curves for concrete in compression and reinforcement are as given in BS5400 (1978). To account for a proportion of load to be long-term, the concrete strains (except the final limiting strain) may be multiplied by a creep factor. Concrete in tension is assumed to crack and no account is taken of tension stiffening. If the slab is formed with profiled sheeting, the contributions of the decking and of concrete within the troughs are both neglected. The relation for structural steel is linear elastic-plastic, followed by strain-hardening at a modulus of E/33 once the strain exceeds eight times the yield value. To determine design values, the partial safety factors for concrete, reinforcement and structural steel were taken as 1.50, 1.15 and 1.0 respectively.

To obtain the moment-curvature relationships, values of curvature are varied. For each value, the position of the neutral axis is altered until equilibrium of direct forces is achieved; the moment is then calculated. The maximum value is taken to correspond to a limiting stress in the steel of 1.3 times the yield value or to a limiting concrete strain in compression of 0.0035.

To analyse a beam, values of end moment are varied. For each value, the distribution of bending moments is altered (whilst maintaining equilibrium relationships) until integration of the corresponding curvatures satisfies the condition of zero slope at mid-span. Equilibrium then determines the load level. Failure is assumed once a moment exceeds the maximum value for which moment-curvature data is available.

To account for end connections, the integration of curvatures includes an end rotation determined from the moment-rotation characteristic specified for the joint. If the characteristic has a plateau, then provided the mid-span region can resist further moment, the analysis continues by incrementing that moment. Equilibrium considerations and integration of curvatures lead to the corresponding connection rotation. The procedure permits reductions in increments as limiting conditions are approached, to determine the response with good accuracy.

The analysis distinguishes between propped and unpropped construction. In the former, no bending action is assumed until full interaction has been achieved. In the latter, account is taken of bending in the steel section. By specifying a stiffness for the steelwork connection, account is taken of semi-rigid joint action during construction.

2.2 Parametric studies

Those by Najafi covered the following ranges:

Beam span (L) : 6.0, 9.0 and 12.0m
Span : depth ratio (L/D) (based on depth of steel section) : 20 - 30
Steel sections : Universal Beams in S275 and S355 grades
Effective breadth of slab : Sagging region 0.175L
Hogging region 0.125L
Overall depth of slab : 120mm
Depth of decking : 46mm
Concrete : Cube strength 30 N/mm^2
Reinforcement : 1% of the effective area of concrete above decking; S460 grade.

The design resistance of the connection, M_{cc}, varied between 70% and 25% of the maximum design moment capable of being resisted by the composite beam in sagging bending, M_{pc}. Propped and unpropped construction was considered.

2.3 Results

The following results were observed from the studies:

1. The rotation required to develop a beam-type mechanism was independent of the connection stiffness, provided that the first plastic hinge forms in the joint; this has been confirmed by Najafi using a graphical presentation and proved by Li using energy theorems.

2. Required rotation increased approximately linearly with reduction in M_{cc} (Fig. 2).

3. Increase in L/D from 20 to 30 caused the required rotation to rise by approximately 30%.

4. As plastification spreads in the mid-span region, the demands on rotation capacity increase sharply. Curve (i) in Fig. 2 shows the rotations required to develop the maximum sagging moment permitted by the analysis. If only 95% of this value is required, then the required rotation capacity is given by (ii). A reduction of at least 30% is obtained.

5. Compared with a uniformly-distributed load, the required rotation can increase by the order of 50% if point loads act at the one-third points.

6. The rotational requirements increase if unpropped construction is adopted.

2.4 Proposals for design

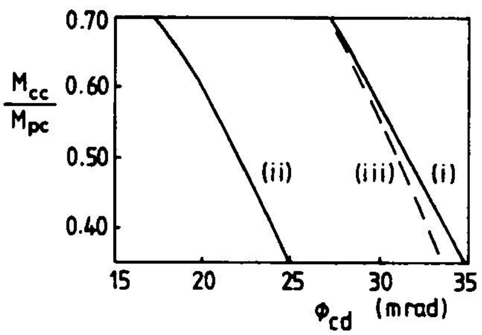

Fig. 2 : 305x165UB40 S355 9m span UDL (Propped construction)

As the required rotation capacity rises sharply if very extensive plastification is required at mid-span, it is appropriate to limit the sagging resistance moment in design to less than the maximum permitted. The sensitivity is shown not only by comparing curves (i) and (ii) in Fig. 2, but also by considering the results of Li et al (1996). The latter computed the maximum resistance moment by rectangular stress-block theory, as permitted by design codes. For the case examined in Fig. 2, the rotation capacity to obtain 95% of this value is shown by curve (iii). This corresponds closely to the present authors' results for the maximum moment - but the latter was determined by elastic-plastic analysis with the limitations given in (2.1).

An upper limit for the required rotation in propped construction can be determined from Fig. 2, which corresponds to the maximum L/D likely to be suitable in practice (Lawson and Gibbons, 1995) and to the highest steel grade in common use. For a uniformly-distributed load and a relative connection moment of 0.5, 22mrad is needed to attain a mid-span moment equal to 95% of the maximum permitted by the authors' analysis. By comparison with results by Li, this is equivalent to 90% of the resistance from BS5950 (1990) for the case shown. For point loads at one-third points, the requirement would be approximately 33mrad.

For unpropped construction, the requirements will depend on the relative load resisted while in this condition, and on whether simple or semi-continuous design is used for that stage. Preliminary studies indicate that the mid-span moment should be limited further, to the order of 85%, to limit rotation requirements to the above values.

3. Ductility available in composite joints

3.1 Failure modes

Possible modes include fracture of the reinforcement, loss of anchorage, failure of the shear connection, failure of the concrete slab by bearing or transverse splitting, and local buckling of steelwork. As for steel joints, some components must deform in a dependable manner to provide an assured rotation capacity. The likelihood of this can be judged from tests representative of composite joints envisaged in practice (Lawson and Gibbons, 1995).

Tests on joints with steel web and flange cleats have demonstrated the importance of slip between the cleats and the steel beam section (Altmann et al, 1991; Aribert et al, 1994; Davison et al, 1990; Xiao et al, 1994). The slip is erratic though and, when prevented, the ductility of the joint is compromised (Davison et al, 1990). A common failure mode was buckling of the column web in transverse compression. If loading continued, further rotation would occur but a significant reduction in moment could be experienced. The tests showed that with only welded mesh for reinforcement, fracture was likely at very limited rotations. In contrast, provision of high-yield reinforcing bars gave 28mrad before fracture, even though slip had been prevented (Davison et al).

With steel fin plates, Xiao et al (1994) have shown that composite joints can develop rotation capacities of over 40mrad. The steel connection derives flexibility from bolt deformation in shear and bolt hole distortions. However, such deformations are dependent on whether or not bolts slip into bearing and calculation of deformation is not straight-forward.

Xiao et al have also shown that joint rotations of the order of 40mrad can be achieved without substantial loss of moment, if partial-depth end plates are attached to the beam clear of the bottom flange. The capacity comes partly from the initial clearance between the lower part of the beam and the face of the column. The tests were terminated as buckling of the beam web was occurring. Although the rotation capacity could be regarded as adequate, the position of the end plate limits the resistance that can be developed.

In contrast, a joint with a full-depth ('flush') end plate maximises the lever arm for the tensile force in the reinforcement, and can provide a worthwhile stiffness during unpropped construction. Tests by Anderson and Najafi, Aribert et al and Xiao et al are taken as representative. This configuration lacks the slip deformations and distortion of bolt holes developed by other joint arrangements. In some tests the maximum recorded rotation was below 30mrad because loading ceased with the onset of buckling; fracture of the reinforcement did not then occur. Other tests showed that if column web buckling was prevented and the shear connection was ductile, rotation capacities of over 35mrad were achieved with a maximum of 1% high-yield reinforcement in the slab. Failure occurred by fracture of the reinforcement, or the tests were terminated at high deformations caused in part by substantial slip of the shear connection or by local buckling of the compression region of the beam. The likelihood of the latter is influenced by the tensile resistance that can be developed in the upper part of the steel connection; the use of thinner end plates or column flanges can lead to fracture of reinforcement occurring at a lower rotation due to a more limited balancing force causing buckling.

Provided that local buckling of the beam only occurs once the design moment resistance of the joint has been attained, these tests have shown that high rotations (> 40mrad) are achievable with only moderate reduction in peak resistance. However, the extent of buckling deformations is influenced by imperfections whose magnitudes are unknown to the designer. With slip of the steelwork connection and bolt-hole deformations excluded, either by the joint configuration or by the difficulty in ensuring these occur in a predictable manner, yielding of the reinforcement remains as the main source of predictable deformation capacity.

3.2 Rotation capacity due to deformation of the reinforcing bars

As composite joints may require substantial rotation capacity, it is important that the deformation of the reinforcing bars prior to fracture can be calculated. A method taking account of tension stiffening has recently been proposed (Anderson (ed), 1997). The results shown in Table 1 compare the proposal with three of the authors' tests (Anderson and Najafi, 1994) in which the only variable was the amount of reinforcement. The specimens were constructed propped. In the test designations, the figure(s) refers to the number of 12mm diameter high-yield bars included in the slab. The calculations are based on measured material properties. The additional rotations given in Table 1 relate to slip of the shear connection (Aribert, 1996) and an allowance for plastic compression in the beam flange immediately adjacent to the joint, where justified by the level of ultimate moment. In S8F and S12F the compressive force, required to balance the tensile action of the reinforcement and the bolts as the connection approached failure, exceeded the yield resistance of the flange by a substantial margin. It is assumed that the strain in the flange reached eight times the yield value (at which strain-hardening was taken to commence in (2.1) above) and that this remained constant over the distance of 130mm from the connection to the point along the beam at which the rotation was being measured.

	Rotation (mrad)				
Test	Rebars	Slip	Compression	Total	Experiment
S4F	20	5	-	25	27
S8F	24	6	4	34	36
S12F	26	8	4	38	55

Table 1: Predicted and measured rotation capacity

The calculated result for S4F includes allowance for mesh, whose contribution was significant in this lightly-reinforced specimen. Good agreement was obtained for this test and for S8F. The experimental rotation corresponded in both cases to fracture of the reinforcement. In S12F substantial deformation arose from local buckling of the bottom flange of the beam adjacent to the connection and the rebars did not fracture. The test was terminated at 55 mrad.

4. Conclusions

Composite connections with full-depth end plates provide a suitable configuration for beam-to-column joints. The moment resistance is substantial; this limits the demand for rotation capacity in plastic design. For the most critical load case expected in practice, a capacity of at least 33mrad should be provided. Using common grades of structural steel, this will permit 90% of the design sagging moment of resistance to be attained in a composite beam constructed as propped.

Tests have shown that such rotation capacity is readily obtainable, but is dependent partly on reinforcement of appropriate quality and quantity being provided. Calculation methods are available to determine these requirements. The resulting combinations of beam section and connections provide significant economies compared to nominally-pinned construction.

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