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A study of continuous stem girder systems

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Abstract. A new beam system comprising two cantilever stems and an interspan composite beam has been developed and its design philosophy is described in this paper. The system provides the equivalent of a semi-continuous beam without the requirement to calculate the moment rotation capacity of the beam-to-column connection. The economy of braced frames using the system has been investigated and compared with simple, continuous or semi-rigid systems. It is shown that the costs of the proposed system are similar to the semi-rigid system and cheaper than both the simply supported and rigid beam systems. Two tests have been carried out on 6 meter span beams, which also incorporated an asymmetric flange steel section. The behaviour of the system is presented and the test results are compared with those obtained from the theory.

Key words: continuous stem girder system (csgs); asymmetric steel section beam (asb); economy of braced frames; semi-rigid connection.

1. Introduction

In steel construction the method of connections can have a large impact on the fabrication and erection costs and therefore the total costs. It has been estimated that the fabrication and erection costs (SCI 1996) are often 30-50% of the overall structural costs although they vary with the types of structures, fabricators and locations. Over 60% of the fabrication costs (Nethercot 1998) is directly influenced by the fabrication of the connections including time spent for arrangement, handling and assembly of steel, as well as transportation of the steel.

The provision of connections should take into account the strength required, the simplicity of their design and fabrication, and the economy. In Eurocode 3 (1992) beam-to-column connections (joints) are classified as pinned, rigid and semi-rigid according to the rotational stiffness. In simple (pinned) connections beam ends are generally assumed as pins and transmit only vertical shear to a column. No moments are transmitted, as shown in Fig. 1(a) and therefore full rotation is permitted. These connections can be made using angle web cleats, fin plates or thin end plates and are normally designed for braced steel frames, whose overall stability against lateral loading is provided by

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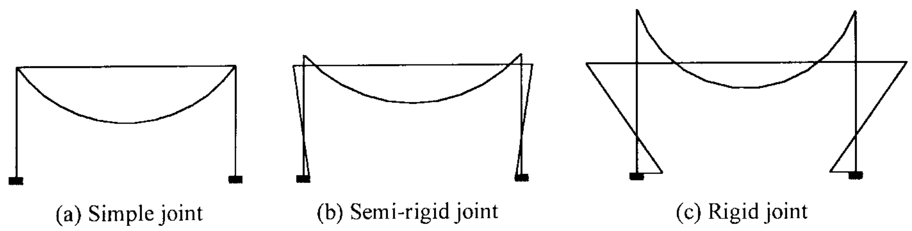


Fig. 1 Bending moment diagrams for frame structures

additional bracing systems.

In rigid design methods a beam is assumed to be fully fixed to a column so as to achieve full continuity and transmit the moment of the beam to the column as shown in Fig. 1(c). It is assumed that there is no rotation between the beam and the column. Rigid connections can be made using extended end plates or by welding the beam to the column and may require column web stiffeners. These connections are generally designed for unbraced steel frames, although rigid connections are also used for braced frames if saving in beam depth is desired.

However, a real joint never behaves in an infinitely rigid manner. The real behaviour of the joint is not ideally ‘simple’ or ‘rigid’. It lies somewhere between the two extremes. The design of joints should therefore be based on the real behaviour of the joints. Furthermore the rigid joints have to be very stiff and this leads to high fabrication costs. For optimised design of the joints and economy for steel framed buildings, semi-rigid joints have been proposed. This form of connection has a finite rotational stiffness and strength, but insufficient to develop full continuity. The moment distribution in a typical frame using semi-rigid joints is shown in Fig. 1(b). Semi-rigid connections may be made using top and seat cleats along with web angle cleats as used in the United States. (Bjorhovde and Colson 1991) Alternatively, full-depth end plates are more commonly used in Britain (Couchman 1997).

Recent studies (Anderson *et al.* 1993, Couchman 1997, Weynand *et al.* 1998) have shown that semi-rigid connections with a finite rotational stiffness and strength could avoid the complexity of rigid connections and hence bring economical benefits. A study by Lawson *et al.* (1997) has shown that up to 24% reduction in the weight of an individual beam could be achieved using semi-continuous connections. Investigations of the economy of steel building frames (Weynand *et al.* 1998) have shown that semi-rigid joints could bring economic benefits leading to possible savings of 20-25% in material, fabrication and erection costs for unbraced frames and of 5-9% for braced frames.

Despite such economical benefits, one of the disadvantages of semi-rigid connections is a complicated design method. The following system, Continuous Stem Girder System (CSGS) may be thought of as an adaptation of the semi-rigid method, however its design philosophy is relatively simple.

2. Continuous stem girder system

This system has two stems and an interspan beam as shown in Fig. 2. The stems are welded to the column in the fabrication shop and the interspan beam is connected to the stems with simple splices on site. Column stiffeners are optional depending on the rotation capacity of the joint

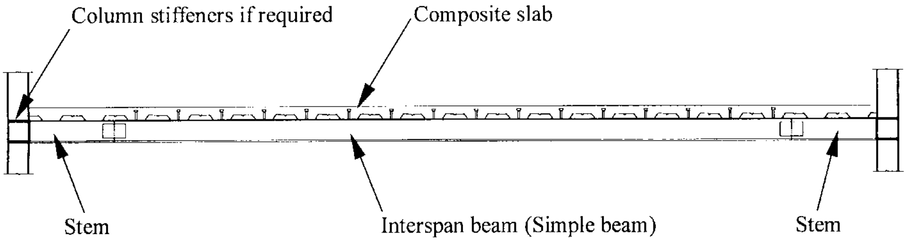


Fig. 2 The continuous stem girder system

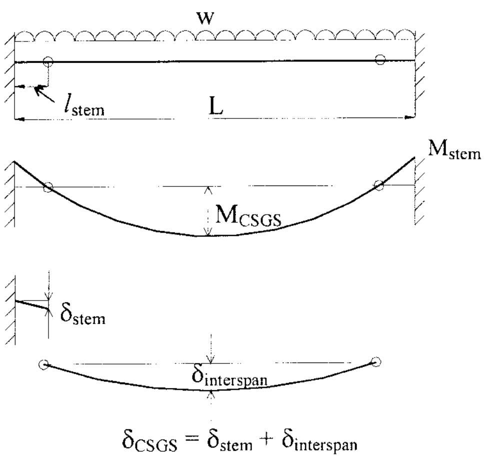
required. The system could be applied for both bare steel structures and composite structures. However to maximise the economy of the system composite beams are assumed here. Once the steel frame is erected on site, metal deck is placed across the steel beam and shear connectors are then welded to the top flange of the beam through the metal deck in the interspan beam region only. No shear connectors are used on the stems since the stems are in the negative regions and it is assumed that concrete is cracked in the region and therefore has no contribution to composite action.

Fig. 3 shows the design concept of the CSGS. It may be assumed that the interspan beam is a simple beam and the stems are cantilevers fixed to the columns. The mid-span moment M_{CSGS} of the CSGS subjected to a uniformly distributed load can be determined by:

$$M_{\text{CSGS}} = \frac{w(L - 2l_{\text{stem}})^2}{8} \tag{1}$$

The moment will vary, depending on the length of the stem. It may be larger than that of the fully fixed-end beam with span L, however it will be always less than that of the simple beam.

The maximum moment M_{stem} acting on the stem can be obtained from the moment combination by the following two loads: the uniform load acting on the whole stem, and the shear load P of the interspan beam acting on the edge of the stem.



$$\frac{wL^2}{24} < M_{\text{CSGS}} < \frac{wL^2}{8}$$

$$M_{\text{stem}} < \frac{wL^2}{12}$$

$$\frac{1}{384}\frac{wL^4}{EI} < \delta_{\text{CSGS}} < \frac{5}{384}\frac{wL^4}{EI}$$

Fig. 3 The design method of the continuous stem girder system

$$M_{\text{stem}}=\frac{wl_{\text{stem}}^2}{2}+Pl_{\text{stem}} \tag{2}$$

The total deflection δ_{CSGS} of the CSGS can be calculated by summing the deflections at the stem (δ_{stem}) and the mid-span of the interspan beam ($\delta_{\text{interspan}}$).

$$\delta_{\text{CSGS}}=\delta_{\text{stem}}+\delta_{\text{interspan}} \tag{3}$$

where

$$\delta_{\text{stem}}=\frac{1}{3}\frac{Pl_{\text{stem}}^3}{EI} \tag{4}$$

$$\delta_{\text{interspan}}=\frac{5}{384}\frac{w(L-2l_{\text{stem}})^4}{EI} \tag{5}$$

The length of the stem is generally governed by fabrication capacities and transportation. The maximum stem to stem length is recommended to be no more than 2.5 m for road transport (Taelim 1993). A study of the stem length on the moments (Kim 1999) has shown that M_{stem} will exceed M_{CSGS} if the stem is longer than 1.75 m when a uniform load of 40 kN/m is acting on a 12 m span beam. This results in an unwelcome solution since the negative moment at the stem will govern the size of the beam. In addition, many fabricators will find difficulty in handling too short or too long stems. The optimum length of each stem is therefore thought to be between 0.45 m and 1.0 m where M_{stem} is less than half the support moment of the continuous beam.

Under such an assumption M_{CSGS} and M_{stem} always lie between the moments of a simple beam and a continuous beam, as shown in Fig. 3. δ_{CSGS} also always lies between the deflections of a simple beam and a continuous beam.

3. Costing programme

An investigation into the economy of the CSGS has been carried out and compared with those of other connections. A three-story, three-span braced structure for an office building was chosen as shown in Fig. 4. In order to maximise the structural efficiency and economic benefits of the system 12 m span girders with three secondary beams and 10m girder spacing with 4 m story height were chosen. The overall stability of the structure against lateral loading was assumed to be provided by additional bracing systems. The beams were therefore designed for vertical loading only.

The following five beam-to-column connections were employed for the chosen model and the systems were analysed in accordance with BS 5950: Part 3 (1990).

- Case 1. Simple connection with angle web cleats
- Case 2. Simple connection with fin plates
- Case 3. Semi-continuous connection
- Case 4. Continuous connection
- Case 5. Continuous Stem Girder System

All beams in the Y-direction, B1 and B2 in Fig. 4 were designed as simple beams with double web angle cleats in all cases. If B2 has a moment connection to the web of the column in Case 3

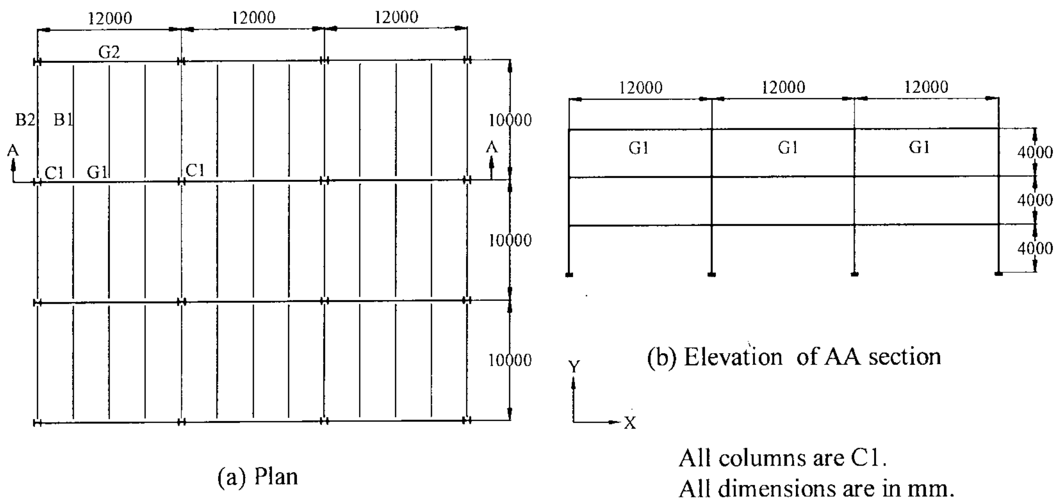


Fig. 4 A model for cost comparisons

and Case 4, there may be difficulty in bolting from one end plate to the other end plate through the column web. In Case 5 it is unlikely for a column to have four stems because of the limit of fabrication capacities and transportation. Both B1 and B2 steel sections are therefore the same in all cases. Double angle cleats of $90 \times 90 \times 8$ with five Grade 8.8, M20 bolts were used for the simple connections of B1 and B2 in all cases except Case 2 in which fin plates were used.

3.1 Case 1. Simple Connection With Angle web cleats (SCWA)

All beams in Case 1 have simple connections using double angle web cleats. These connections are designed to transmit only shear force to columns. B1 and B2 are connected to primary beam webs or column webs. G1 and G2 are primary beams connected to the column flanges using double angle cleats of $100 \times 100 \times 10$ with seven Grade 8.8 M20 bolts as shown Fig. 5(a). All the beams and columns are cut, drilled in the shop and then bolted on site.

3.2 Case 2. Simple Connection With Fin plates (SCWF)

The use of a bolted fin plate connection is becoming common in Britain (SCI and BCSA 1991) because of its simplicity to fabricate and ease of use on site. These fin plate connections were employed for all beams in Case 2 instead of the angle web cleats of Case 1. Fin plates of 10 mm thick were connected to the webs of primary beams using 10 mm welds, and fin plates of 20 mm thick to the column flanges using 20 mm welds. The primary beams (G1 and G2) and the secondary beams (B1 and B2) were then connected to the fin plates using nine Grade 8.8 M20 bolts and four Grade 8.8 M20 bolts respectively on site. Shear forces can be transmitted to the columns by the short fin plates in Fig. 5(b). The beam and column sections used in Case 2 are the same as used in Case 1. The connections were calculated based on the references by SCI and BCSA (1991), and Owens and Cheal (1989).

3.3 Case 3. Semi-Continuous Connection (SCC)

The semi-continuous connections employed in Case 3 are based on Couchman's design methods (1997). It should be noted that Couchman's semi-continuous connections were applicable to frames with non-composite beams. It is however assumed that the general philosophy of the design methods could be applied to the frames with composite beams.

For the beam-to-column connections extended end plates of 15 mm thick with eight Grade 8.8 bolts of M24 were used for both G1 and G2. The connections were made using 10 mm welds. Fig. 5(d) shows the details. No column stiffeners were used.

3.4 Case 4. Continuous Connection (CC)

Continuous connections are rarely used for braced structures unless particular circumstances require them (for example, to satisfy limited beam depth or storey height.). This example was, however, included in order to provide relative cost comparisons. The beam-to-column connections were designed to be rigid. Firstly, an extended end plate of 35 mm thick was welded to the end of a steel beam in a shop and then the beam was bolted to the column flange using twenty Grade 8.8 bolts of M24. Column stiffeners with the same thickness of the beam flange were used so as to avoid column web buckling and to transfer the full tensile force through the beam-to-column connection. The detail for this connection is shown in Fig. 5(e).

Beam-to-column connections should resist the large negative moments of a beam and transmit

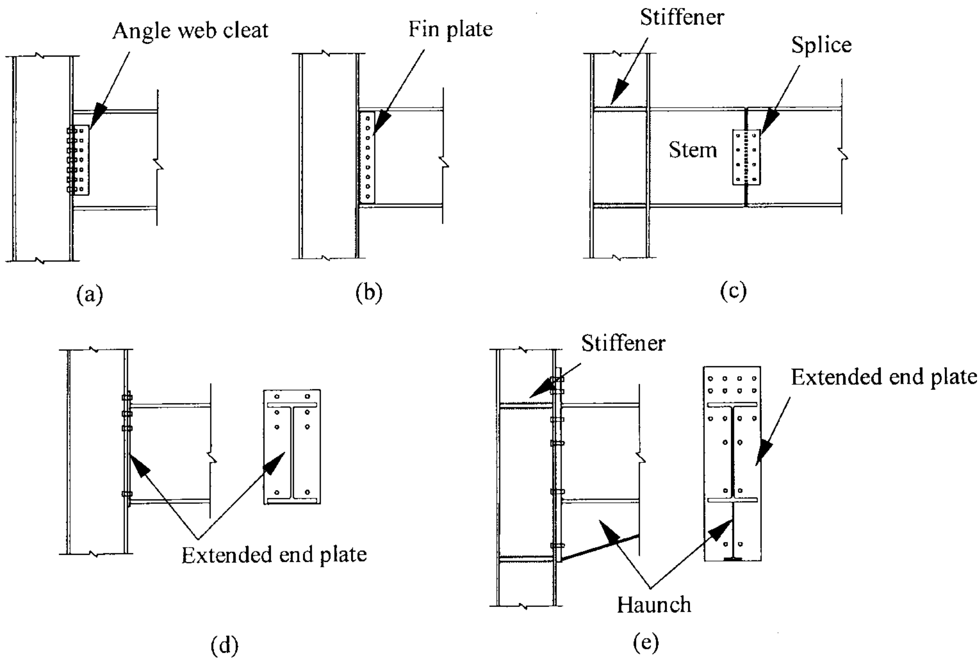


Fig. 5 Details of beam-to-column connections: (a) Simple connection with angle web cleats; (b) Simple connection with fin plates; (c) Continuous Stem Girder connection; (d) Semi-continuous connection; and (e) Continuous connection

them to an adjacent column. The large negative moments increased the number of bolts needed in the tension region, so two lines of four bolts and one line of four bolts were used above and below the top flange of the beam respectively. However, they were still insufficient because of the short lever arm. Haunches of 305×102 UB 28 kg/m were used to increase the lever arm. A larger column section of 356×406 UC 393 kg/m (compared with 356×368 UC 177 kg/m used in the other cases) was used so as to satisfy the shear of the column web due to the bolt load.

3.5 Case 5. Continuous Stem Girder System (CSGS)

Stems of 0.6 m were fully welded to columns in the shop and then the stems were bolted to the interspan beam using double splices with eight HSFG bolts of M24 on site. Column stiffeners were unnecessary in the design calculations. However stiffeners with the same thickness of the beam flange (CSGS1) were used in order to determine the effect of the use of the column stiffeners on the total costs, in comparison with unstiffened connections (CSGS2). The detail for this connection is shown in Fig. 5(c).

3.6 Cost comparisons

Fig. 6 shows that the CSGS and semi-continuous system lead to lighter steel beams. G1 and G2 can be reduced up to 25% (59 kg/m) and 28% (48 kg/m) respectively in comparison with braced frames using simple or continuous connections. This is because of the reduced mid-span and end moments.

Cost evaluations were based on the costing programme developed by Leonard (1997). The costs of fabrication were based on the estimation of the times that were required for the various operations during fabrication. They consisted of the costs of materials, cutting, drilling, assembly and welding. Fig. 7 shows the cost of the structures. It is seen that savings of 15% in steel weight alone can be made using the CSGS and its total costs are 9% less than those of systems using

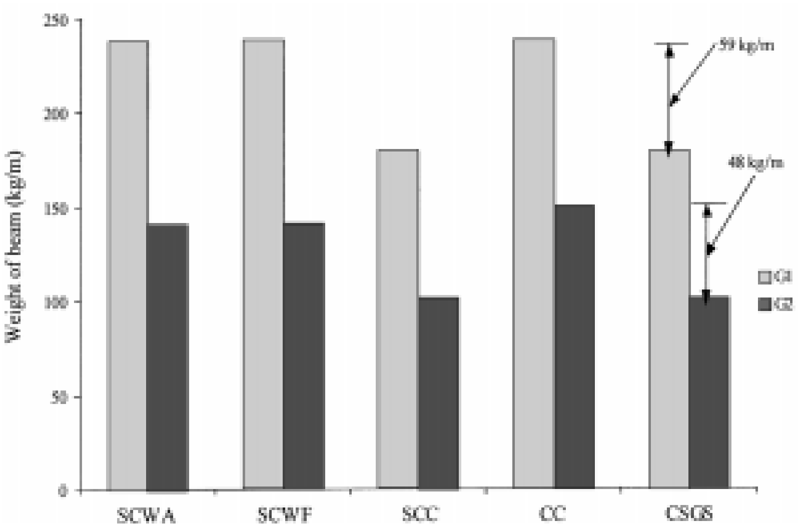


Fig. 6 Comparison of the weight of the primary beams

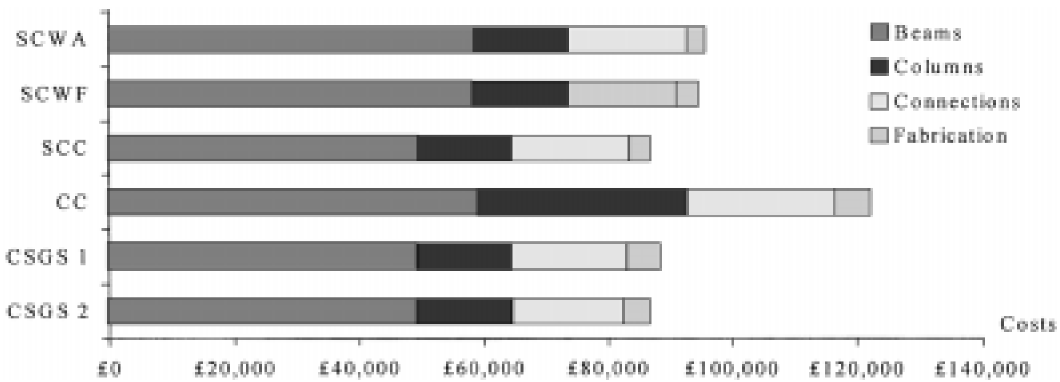


Fig. 7 Costs of structures when the labour cost of £ 15.00/hr was employed

simple connections. Whilst, they were 2.1% larger than those for systems using the semi-continuous connection when the column web was stiffened by stiffeners (CSGS1). This is due to the higher fabrication cost of the CSGS. However, the total costs of both cases were approximately the same (0.1% difference) when no column stiffeners (CSGS2) were used for the CSGS in Case 5.

4. Experimental programme

The costing analysis in the preceding section has shown that the CSGS had potential economy. This encouraged the Authors to study the behaviour of the CSGS. The Authors therefore carried out two beam tests in order to validate integrity of the system. Two specimens of 6 m span and 0.71 m high columns with concrete slabs of 0.9 m wide and 75 mm thick incorporating profiled steel sheeting were fabricated and tested in the Heavy Structures Laboratory of the University of Strathclyde in Glasgow.

In practice an economical span for composite beams is thought to be around 8 to12 m and the most efficient floor arrangement for composite slabs is 3 m in a range of a slab thickness from 110 to 150 mm (SCI 1996, Wright 1990). However, the size of a test specimen is often limited by the condition of the laboratory or the maximum loading capacity available. This experimental study was limited by both. The maximum loading capacity in the Heavy Structures Laboratory of the University of Strathclyde is 250 kN. The test specimen therefore had to be half full size. Two composite beams of 6 m span with a 75 mm thick composite slab were fabricated as a half-size of a 12 m span beam with a 150 mm thick slab.

The objectives of this experiment were to determine:

· the integrity of the proposed system,
· the stiffness of the composite system,
· the moment resistance of the asymmetric steel section composite beam,
· the strength of the shear connection of the asymmetric steel section composite beam,
· the behaviour of the non-composite stem in the hogging moment regions,
· the behaviour of the stem-to-interspan beam connection, and
· the behaviour of the stem-to-column connection.

178×102 UB 19 kg/m of 0.5 m long was used for the stems and an asymmetric steel beam of

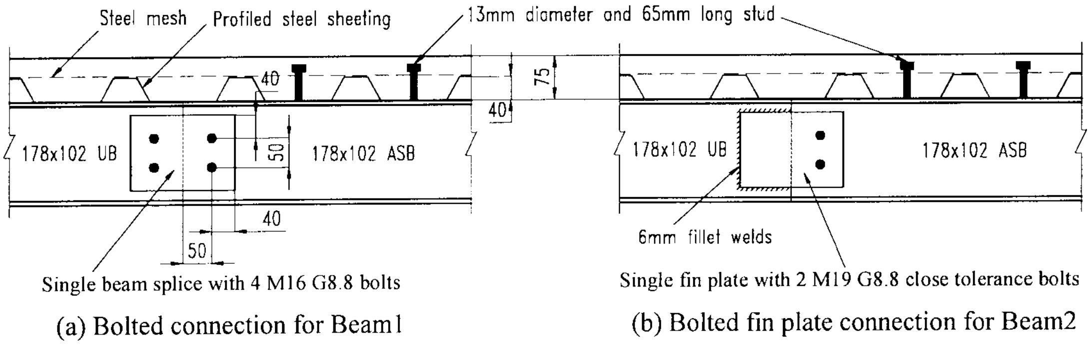


Fig. 8 Stem-to-interspan beam connections

5 m long for the interspan beam. Two options of asymmetric steel beams, ASB1 and ASB2 were employed. Their top flanges were 75% and 50% of the bottom flange respectively. The stems were welded to the column flange using 6 mm welds and then connected to the asymmetric steel beam using a single splice with four Grade 8.8 M16 bolts (see Fig. 8a). A standard tolerance of 2 mm was used. It was however observed in the first beam test that a vertical discontinuity occurred at the stem-to-interspan beam connection and this was a major cause of deflections. To overcome this problem the connection was modified for the second beam. The beam splice was welded to the stem and then bolted to the ASB as shown in Fig. 8(b). In addition, two Grade 8.8 19 mm close fitting bolts were used. The size of a hole is therefore the same as the bolt diameter (17.5 mm). Close-tolerance bolts are often used to improve the slip resistance between the web of a steel section and a plate in bolted connections (SCI 1996).

A composite slab of 900 mm wide and 75 mm deep incorporating profiled steel sheeting was used. Plain profiled sheeting 40 mm deep was laid across the steel beam and 13 mm diameter and 65 mm long headed stud shear connectors were welded to the top flange of the ASB through the sheeting. The studs were placed within the interspan beam only. Reinforcement of A142 (6 mm bars, 200 mm each way) was placed on the top of the sheeting (see Fig. 9). The concrete depth of 75 mm allowed for 15 mm concrete cover from the stud head after welding.

Concrete was made using ordinary Portland cement and 10 mm normal-density aggregate. The

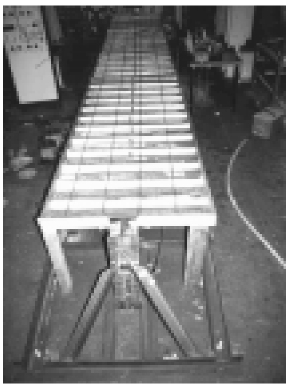


Fig. 9 Test specimen before casting concrete

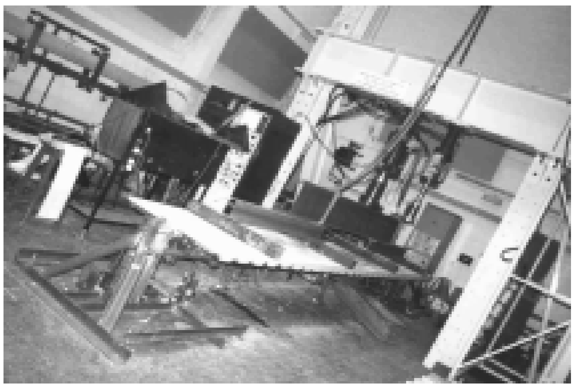


Fig. 10 Loading arrangement

water/cement (W/C) ratio was 0.55, and the slump was 100 mm for Beam 1 and 80 mm for Beam 2. The formwork stripping and the removal of the props were carried out 7 days after the casting. The specimen was cured and tested 14 days after the casting.

5. Discussion

The beams were tested using four point loads, as shown in Fig. 10, which were assumed to be approximately equivalent to a uniformly distributed load. It took approximately four hours to undertake each beam test. To avoid sudden failure displacement-control was used for the jack.

5.1 The moment resistance of the asymmetric steel section composite beam

The moment capacities of the composite beams incorporating asymmetric steel sections obtained from the tests are given in Table 1 and compared with theoretical moments of resistance, which were calculated using BS 5950: Part 3 (1990) and also confirmed using Eurocode 4 (1992). The theoretical results are based on the fact that the interspan beam was assumed to be a simply supported composite beam. The interspan beam is therefore subjected to a positive moment only. The theoretical moments in Table 1 were derived from the calculations of composite beams using asymmetric steel sections. As shown in Table 1, the test results were up to 18% higher than the theoretical results.

Fig. 11 shows bending moment diagrams. Elastic bending theory was used for the calculation of the bending moments of the simple beam and continuous beam. The bending moments of Beam 1 and Beam 2 were calculated on the basis of the elastic strains obtained from the tests and the elastic section modulus (Z): the section modulus of the steel beam for the stem and the section modulus of the composite beam for the interspan beam.

The mid-span moments of both Beam 1 and Beam 2 lay between those of the simple beam and the continuous beam, as shown in Fig. 11. The value of Beam 1 was close to that of the simple beam (92% of the values for the simple beam), whereas the value of Beam 2 was close to the continuous beam (157% of the values for the continuous beam). This was due to the stem-to-beam connection. The bolted connection resulted in Beam 1 behaving like a simple beam and the fin bolted connection resulted in Beam 2 behaving more like a continuous beam. Whilst, the negative moments of Beam 1 and Beam 2 on the stem were 68% of that of the simple beam and 60% of the continuous beam respectively. The design philosophy of the CSGS introduced in the earlier this paper can therefore be applied and Eqs. (1) and (2) can be used. This includes an extra safety margin of 40% at the mid-span.

Table 1 Properties of the test beams and moment capacities of the composite beams

Test beam	Steel		Concrete		Moment capacity (kNm)		Comparisons
	Y_s N/mm^2	E_s kN/mm^2	f_{cu} N/mm^2	E_c kN/mm^2	Test	Theory	Test/Theory
Beam 1	308	189	35.7	20.9	108.4	94	1.15
Beam 2	308	189	34.8	20.4	118.0	100	1.18

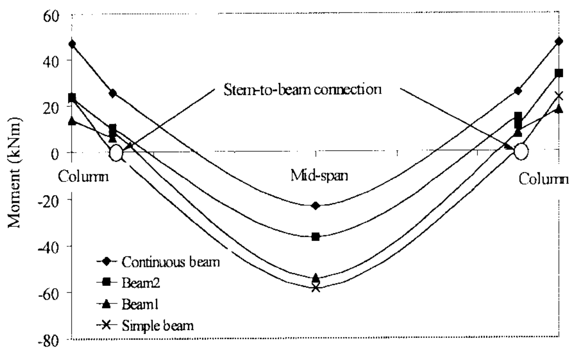


Fig. 11 Bending moment diagrams

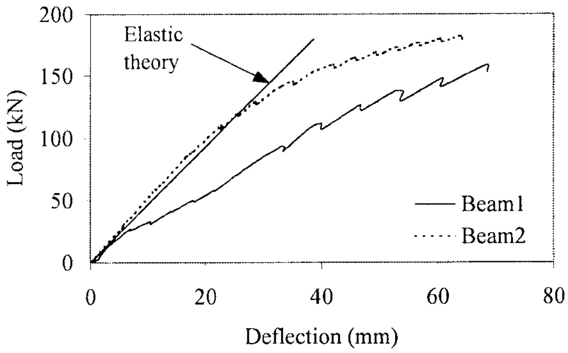


Fig. 12 Deflections at the mid-span

5.2 The deflections of the continuous stem girder system

As shown in Fig 12, more deflections occurred at the mid-span of Beam 1 at low loads than in Beam 2. This was because the deflections at the end of the interspan beam of Beam 1 suddenly increased at a load of 33 kN due to the slip at the single beam splice stem-to-beam connection. The modification of the connection to the bolted fin plate undertaken for Beam 2 improved the resistance to slip. The deflections at the end of the interspan beam of Beam 2 were therefore reduced in comparison with those of Beam 1. This resulted in relatively a stiff line of the mid-span deflection at the elastic stage and a higher stiffness (22%) at failure.

The theoretical deflections of the CSGS calculated on the basis of Eq. (3) were compared with the experimental results in Fig. 12. It is seen that the theoretical deflections were approximately 8% more at the elastic stage than those of Beam 2. This implies that Eq. (3) could be used, giving 8% marginal safety.

5.3 The load-slip relationship of the asymmetric steel section composite beam

The movement of the composite slab relative to the steel beam (slip) was measured by the dial gauges, which were attached close to the steel top flange. The magnetic part of the dial gauge was attached close to the steel top flange and then its dial pin was attached to the aluminium angle glued to the bottom flange of the profiled steel sheeting. The dial gauge readings along the beam near the failure are plotted in Fig. 13. It should be noted that the applied loads for Beam 1 and Beam 2 were 160 kN and 181 kN respectively. In Beam 1 the dial gauges at both stems, which were non-composite regions, were detached at low loads due to the separation of the slab from the steel beam shown in Fig. 14. In Beam 2 the movement of the non-composite stem relative to the slab was up to three times larger than the maximum slip of the composite beam, which was 1.3 mm. This is because there were no studs welded between the stem and the slab, thus allowing large slip between them.

5.4 The behaviour of the non-composite stem in the hogging moment regions

No studs were welded to the stems since in the hogging moment regions concrete is assumed to be cracked. It is thought in Beam 1 that due to the absence of studs, the composite slab started to

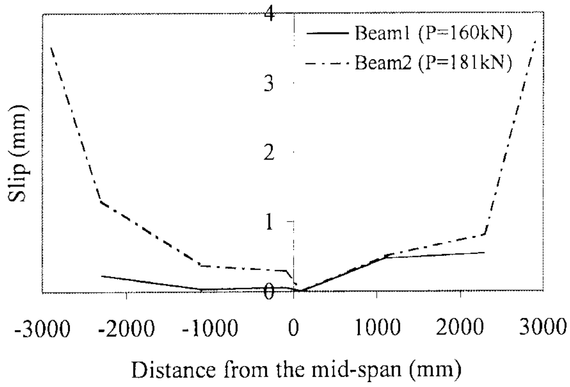


Fig. 13 Load-slip curve near failure

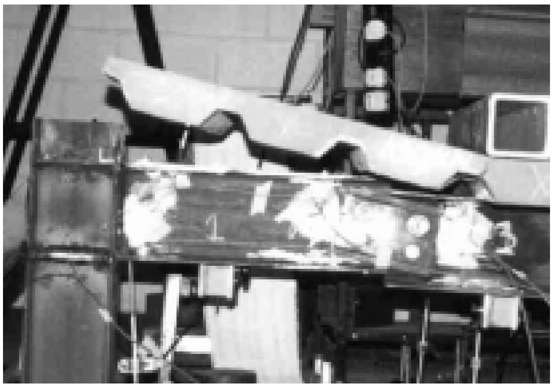


Fig. 14 The separation of the slab (Beam 1)

separate from the steel beam at low loads. This also resulted in more deflections in the interspan beam since the self-weight of the slab was added to the interspan beam, and more rotations between the stem and the column since the stems were free from the slab. Fig. 14 shows the separation of the slab near failure.

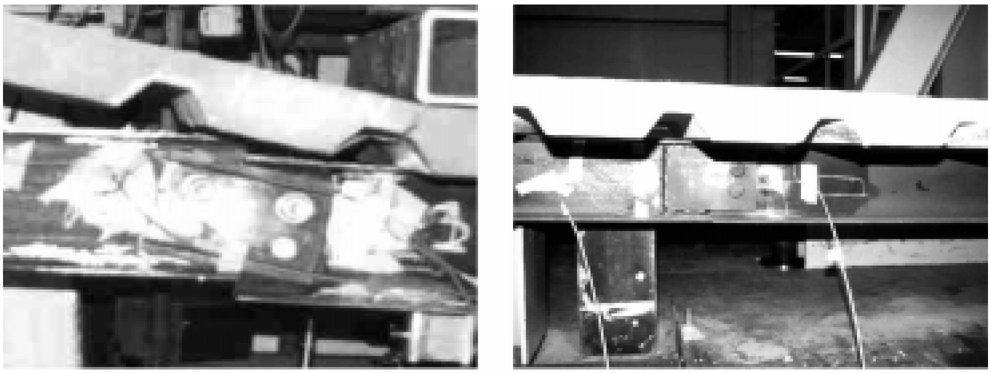
However, in Beam 2 little separation between the slab and the steel beam occurred even at failure despite there being no studs. It is clearly found that the major cause of the separation was the connection between the stem and the interspan beam. When the applied load increased, the beam splices of Beam 1 slipped towards the interspan beam and because of the slip the slab were lifted up as shown in Fig. 14.

5.5 The behaviour of the stem-to-interspan beam connection

The stems were connected to the interspan beam using a single beam splice for Beam 1 and a single fin bolted plate for Beam 2. It was clearly observed from the tests that the type of the stem-to-interspan beam connection greatly influenced the deflections, the separation of the composite slab from the steel beam described in the previous section and the vertical discontinuity at the connection as shown in Fig. 15. The maximum vertical discontinuity near failure was as much as 25 mm in Beam 1, whereas it was only 1.74 mm in Beam 2. The connection of Beam 1 behaved like a pin connection, although it was not absolutely free to move due to the stiffness of the beam splice, whilst the connection of Beam 2 behaved more like a continuous one.

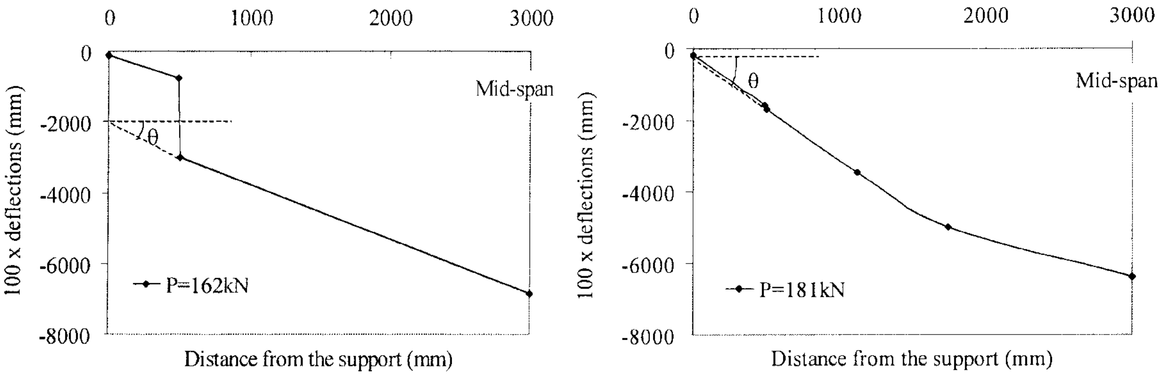
Due to the vertical discontinuity occurring at the stem-to-interspan beam connection the curvature or rotation of the interspan beam would differ from that between the stem and column discussed in the following section. Fig. 16 shows the extrapolation for the rotation of the interspan beam at failure. The values on the X-axis are the distance from the support and the values on the Y-axis are the deflections, which were multiplied by 100 times the actual ones. The deflection line is extended straight at the end of the beam till it reaches the Y-axis. The angle of the straight line to the X-axis is assumed to be the rotation of the interspan beam. Values of 0.016 rads and 0.029 rads were obtained for Beam 1 and Beam 2, which were 15% and 25% higher than those between the stem and column. This implies that in the CSGS the rotation of the interspan beam is more crucial than that between the stem and the column.

After the tests the beam splices of Beam 1 were removed from the steel beam and the steel beam



(a) Beam 1 (b) Beam 2

Fig. 15 Deformation at the stem-to-interspan beam connection near failure



(a) Beam 1 (b) Beam 2

Fig. 16 Extrapolation of the rotation of the interspan beam

is shown in Fig. 17. When the vertical discontinuity occurred at the connection, the splices tended to slip towards the lower part, the interspan beam and then the bolts could move within the tolerance of 2 mm each bolt. Once the discontinuity exceeded the limit of the tolerance, the bolts could still support the connection, but against the splices. So the holes in the web of the steel beam started deforming towards the opposite direction of the splices, namely towards the stems.

5.6 The rotation of the stem-to-column connection

In order to measure the rotation between the stem and the column two dial gauges were attached to the inner flange of each column at 100 mm spacing vertically. The upper dial gauge was expected to give higher readings than those of the lower dial gauge since the column tends to deform towards the beam direction and the upper part of the column is free to move, whilst the lower part of the column is restrained by the footing. However negative horizontal movements occurred in the dial gauges attached to the lower part of the column of Beam 2, although no horizontal forces were applied. This could be because the direct forces at the end of the interspan beam were pushing the stem. Due to these negative movements the rotations of Beam 2 were higher

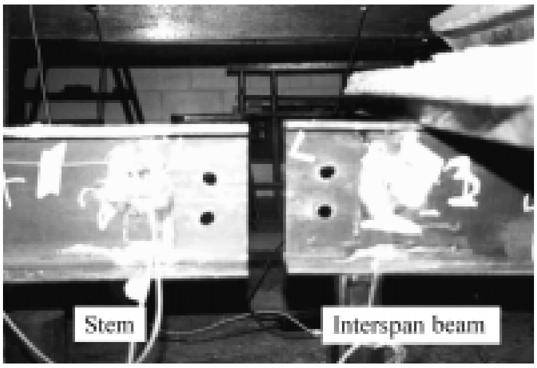


Fig. 17 Deformation of the beam web (Beam 1)

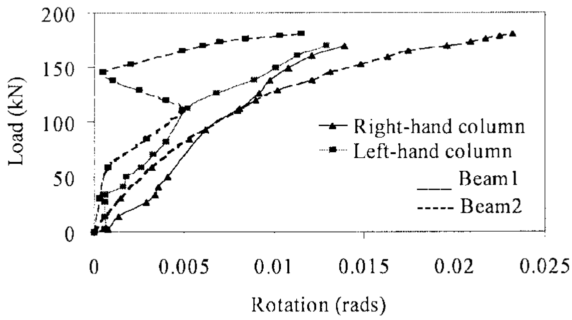


Fig. 18 Rotation between the stem and column

since the difference in the horizontal movement of the column was higher, or showed slightly a different line from those of Beam 1 (see Fig. 18). The maximum rotation was 0.0139 rads in Beam 1 and 0.0232 rads in Beam 2 respectively and regarded still within the range of rigid connections.

5.7 Failure modes

Fig. 19 shows the test specimen (Beam 1) just before failure. The steel beams tended to be twisted to the direction of the web connected to the beam splices. This twisting might be due to the torsional loading, which was inadvertently applied at 8 mm off-centre in the transverse direction. Torsional resistance of the asymmetric steel beams, Beam 1 and Beam 2 was calculated (SCI 1989) and compared with that of the Universal Beam in Fig. 20. The torsional rigidity, GJ of the asymmetric steel beam was decreased by around 21%. However its warping rigidity, EH decreased 41% in Beam 1 and 78% in Beam 2. Its total torsional resistance, T_r was 38% in Beam 1 and 29% in Beam 2 of that of the Universal Beam.

On the other hand, in the tests of Beam 2 no particular failure modes were found apart from the yielding of the bottom flange of the steel beam at the mid-span. No great deformation was found at the stem-to-beam connection as shown in Fig. 15(b).

Little deformation was found in the shear connection between the steel beam and the composite

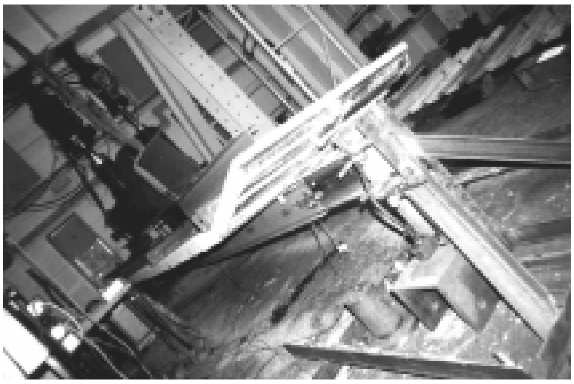


Fig. 19 Deformation near failure (Beam 1)

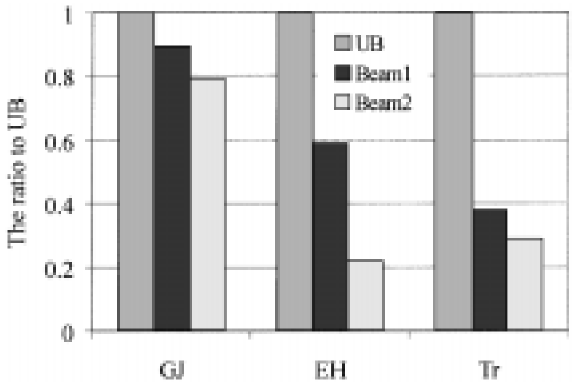


Fig. 20 Torsional capacity

slab in the composite interspan beam region in both Beam 1 and Beam 2.

6. Conclusions

The Continuous Stem Girder System has been introduced and its design philosophy developed in this paper. It has been found that the system provides the equivalent of a beam with semi-rigid connections, but its design is simpler. The costing analysis has shown that the system is cheaper than the simple connections or the continuous connections and only 2.1% more expensive than when semi-rigid connections are used. The costs were however approximately the same when no column stiffeners were used.

The overall conclusion, which can be drawn from the tests of the CSGS is that the type of the stem-to-interspan beam connection greatly influenced the vertical discontinuity occurring at the connection, which resulted in unexpected deflections and the separation of the composite slab from the steel beam in the non-composite stem region.

The experimental results showed that the composite interspan beam of the CSGS contained enough strength (40% extra safety margin) and ductility (8% extra safety margin) at the elastic stage when the vertical discontinuity at the stem-to-interspan beam connection was prevented by using a fin plate. The design philosophy of the CSGS introduced in the earlier section can therefore be used.

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