Oct 20th, 12:00 AM

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Eleventh International Specialty Conference on Cold-Formed Steel Structures
St. Louis, Missouri, U.S.A., October 20-21, 1992

INFLUENCE OF DEFORMED METAL DECKING COMPOSITE FLOORS TO BEAM-COLUMN CONNECTIONS

By

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SUMMARY

A series of tests designed to investigate the interaction of a variety of different steel beam to column connection details with a composite metal deck floor is described. The main emphasis is on assessing the connections' moment capacity, rotational stiffness and rotation capacity. The full details of the experimental behaviour of the flush end plate and partial depth end plate connections will be described in this paper. A simple method for moment capacity calculation based on the test results is proposed herein.

1. INTRODUCTION

Cold formed metal decking floor systems have become increasingly popular in recent years. A major structural advantage is to integrate the structural properties of cold formed steel decking and concrete by ensuring composite action through the use of through deck welded shear studs. This type of construction has now became a common feature in multistory steel frame buildings in several countries[1-4]. A number of researchers have made ad hoc studies of the influence of the composite floor on the behaviour of beam to column connections.[5-6] It was found that the composite floor had a strong influence on the moment capacity and rotational stiffness of the joint. Neglecting the contribution to joint properties from composite action may result in either an over conservative or an unconservative assessment of the actual behaviour of the composite frame. However, a general basis for the design of composite joints has not yet been established and the large number of variables present means that composite joints need further study. A test programme supported by the Building Research Establishment in the U.K. has been conducted at Nottingham University. The main emphasis is on assessing the three key indications of performance: moment resistance, rotational stiffness and rotation capacity. Knowledge of these fundamental properties is necessary before the true semi-rigid and partial strength nature of the connections can be properly incorporated into more realistic methods of frame design. Specimens were therefore selected to include several currently used joint types as well as to investigate slab parameters expected to influence the connection behaviour directly.

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2. TEST SPECIMENS AND SET-UP

2.1 SPECIMEN DESCRIPTION

The conventional metal deck flooring system comprises a concrete slab supported by profiled metal decking sitting on steel beams. Shear connection is typically provided by through deck welded shear studs. All tests were on internal 'cruciform' specimens as shown in Fig.1, with four different types of steel joint being used as shown in Fig.2. Beams were 305x165x40 UB and columns were 203x203x52 UC in grade 43 steel. Reinforcement was A142 mesh, supplemented in some cases by T10 or T12 rebars. PMF CF46 deformed metal decking was used as bottom shuttering. Full details of the setting-up and casting procedure are given in ref.7, which also contains a description of the test rig, instrumentation and test procedure.

3. TEST RESULTS AND ANALYSIS

The results for the cleated and web side plate specimens have been presented and discussed elsewhere[7]. Thus attention will be focussed on the completed tests on flush end plate and partial depth end plate connections. General information and the main results for all specimens are summarized in Table1.

3.1 COMPOSITE FLUSH END PLATE CONNECTION

All flush end plate connections were generally as shown in Fig.2(a). Specimen SCJ3 used only A142 mesh in the concrete and was designed to look into the effect of a more rigid steel detail on the strength and stiffness of the composite connection. Compared with cleated and web side plate specimens SCJ1 and SCJ2, the specimen had a relatively high initial stiffness. The final strength of the joint was 84 kN·m; much stronger than SCJ1 and SCJ2. The failure mode in all three cases was mesh fracture. A comparison of the behaviour of three different joints each with the same type of mesh reinforcement is shown in Fig.3, from which it can be seen that the flush end plate connection has the highest moment capacity. Decrease of stiffness caused by the cracks in the slab and the onset of bolt slip occured comparatively early for both cleated and web side plate connections. The flush end plate connection was not affected by bolt slip due to its bolt orientation.

The ratios of reinforcement of SCJ4 and SCJ5(with column web stiffeners) were designed as 1.0% rebar in the concrete. Flexural cracking in the slab started from the tips of the column's flange. Compared with specimen SCJ3, a more uniform pattern of cracking was obtained. Unlike cleated joints, this joint was able to maintain the applied loading long after the onset of cracking. The loss of stiffness was not very large compared with specimen SCJ3. The moment-rotation response of SCJ4 is shown in Fig.4. Failure was associated with excessive deformation of the column flange and buckling of the web of the column. Strain gauge readings indicated yielding of the reinforcement. The maximum moment achieved was 202.9 kN·m for SCJ4. Specimen SCJ5 had the same reinforcement ratio as SCJ4. However, at the level of the bottom flange of the beam, its column web was stiffened by two horizontal steel plates as indicated in Fig.5. The cracking pattern was similar to that of specimen SCJ4. Both strength and rotational capacities were clearly enhanced by the presence of the web stiffeners. Ultimate moment for the joint was 240.8 kN·m. Instead of excessive column flange deformation, failure of the joint was due to local buckling of the bottom flange of the beam. No buckling of the column web or stiffeners was detected. The connection's ability to resist compressive force in the lower part was increased by the web stiffening which resulted in the moment capacity increasing. This arrangement permits full utilization of the steel sections and is also in line with the design philosophy of restricting failure to beams rather than columns.

SCJ6 with the same ratio of reinforcement as SCJ4 was designed to check the effect of a different shear/moment ratio on the moment-rotation behaviour. The load position was moved closer to the column at 0.8m to produce the high shear. Compared with SCJ4, the general behaviour was quite similar but the resistance moment of SCJ6 was decreased to 157.6 kN•m as indicated in Fig.5. The eventual failure was by buckling of the column web and excessive flange deformation. The presence of the high shear force decreased the moment capacity by 22%.

SCJ7 was designed with two framed-in beam stubs connected to the minor axis of the column rather than two horizontal web stiffeners. By doing so, it was expected that the column web would be stiffened by the beam stubs, acting as de facto stiffeners. The test was terminated due to excessive column flange deformation. The ultimate resistance moment was close to that of SCJ4 as shown in Fig.6. This arrangement without the usual web stiffeners only changed the failure mode from web buckling to excessive flange deformation. No clear moment enhancement was found. After the test, the concrete floor was cut into two pieces to expose the shear studs with the concrete removed, the longitudinal cracks were continuous through the shear connector area. Shear connectors were found that had been bent severely as indicated in Fig.7. Although the reinforcement had been increased to 1.2% in SCJ7, the moment capacity was still determined by the shear connector strength.

The tensile zone of the composite connections were significantly enhanced due to the presence of the concrete floor. If the shear resistance of the shear connector is sufficient, the main variable will be the reinforcement ratio in the slab because the influence of the concrete will be negligible after cracking. Reinforcement ratios from 0.2- 1.2% were covered for the first and second phase work. Test results showed that the initial stiffness and moment resistance were increased as the ratio increased even with only the mesh reinforcement. The rotational capacity was improved as the eventual failure changed from a brittle type to a ductile one. Use of an appropriate ratio of reinforcement in the connection clearly improves the joint properties of moment-resistance, initial stiffness and rotational capacity.

Three different eventual failures(in column, beam, reinforcement respectively) were detected in the flush end plate connections with different ratios of reinforcement and web stiffening present. The tests have shown that such composite connections (i.e. with thin end plates) possessed a good initial stiffness and suffered comparatively little loss of stiffness after cracking of the concrete floor and yielding of the reinforcement (because of the relatively high rigidity and rotational capacity of this steel detail). Although the column stiffening should be avoided whenever possible for ease of construction, its use is clearly advantageous in terms of beam-column connection strength enhancement and failure mode control. Framed-in beams in the orthogonal direction did not increase the moment resistance, but effectively prevented column web buckling. The column flange enhancement will be further studied in the third phase of the work.

3.2 COMPOSITE PARTIAL DEPTH END PLATE CONNECTION

The partial depth end plate as shown in Fig.2(b) is normally considered as being sufficiently flexible to be considered as a simple connection in its bare steel state. It is current practice for the end plate to be welded to the top portion of the beam as this helps increase stiffness. However this was thought to be incompatible with composite connection design principles due to the availability of the stronger tensile zone. Three different positions of the partial depth end plate connection with the same ratio of reinforcement (0.8%) were chosen to study this influence.

Specimens SCJ8, SCJ9, SCJ10 represented different locations of end plate welded to the top, middle and bottom portion of the beam. The moment-rotation curves of all three specimens are shown in Fig.8. Moment capacity dropped down after the peak load and increased again as the web of the beam came into contact with the flange of the column. The test curves of the two specimens SCJ8 and SCJ9 are quite similar. The cracking pattern was limited to several large cracks. But the initial stiffness and moment capacity were both increased for specimen SCJ9 compared with SCJ8. The eventual failures for the SCJ8 and SCJ9 were both beam web buckling as indicated in Fig.9. SCJ10 exhibited a different behaviour compared with the above two specimens. The stiffness and moment resistance further increased as shown in Fig.8. The eventual failure changed from the beam to the column web buckling as indicated in Fig.10. More fine cracks appeared in the concrete floor. The test result indicated that the position of the partial depth end plate is crucial for the stiffness and moment capacity of the composite connection. When the concrete floor is in place, the current practice of placing the partial depth end plate near the top of the beam will cause a smaller resistance moment and stiffness. A more appropriate position would be to weld the partial depth end plate towards the bottom portion of the beam.

4. MOMENT CAPACITY CALCULATION FOR COMPOSITE END PLATE CONNECTION

At present no generally applicable methods exist for the design of composite connections. Separate calculation procedures have been developed for web side plate, flush end plate and partial depth end plate composite connections that consider the composite influence on the steel joint . These are consistent with the current BS 5950; Part 3.1 approach[8] for composite beams and composite slabs. The plastic analysis concept has been adopted within the framework of ultimate limit state design. Tensile resistance of the cracked concrete is neglected and strain hardening of the reinforcement is not considered in the calculation. In this method only the contributions of the beam section, reinforcement and connecting components are included. The shear and compression zone of the column web should also be checked using EC3[9]. Comparisons between the calculated and test values for the different connections are listed in Table2. The procedure for the moment calculation for a partial depth end plate composite connection will be described herein as an example for end plate connections.

The partial depth end plate connection shown in Fig.2(b) is assumed to have only two bolts in each bolt-row. This proposed method is valid for the cases where the end plate is welded to the bottom flange of the beam and/or to the beam web. Initially the reinforcement in the top of the section is assumed to yield. On this basis, a calculation is performed to determine the neutral axis depth as indicated in Fig.11. If the calculated neutral axis depth is too close to the top of the concrete section (because the reinforcement ratio is excessive), it is then assumed that the reinforcement has not reached yield and a new equilibrium condition is established to determine the neutral axis position again.

Compressive force F_b of the beam end connected to the end plate is:

$$F_b = p_y(l - t_{bf})t_{bw} + p_y f_l t_{bf} \tag{1}$$

in which
- l is the length of the partial depth end plate
- p_y is the yield strength of steel
- f_l is the beam flange width
- t_{bf} is the thickness of the web of the beam
- t_{bw} is the thickness of the web of the beam

Tensile force F_s of the top reinforcement is:

$$F_s = f_y A_f \tag{2}$$

in which
- $A_f = \rho A_c$
- f_y is the yield strength of the reinforcement
- ρ is the ratio of the reinforcement
- A_c is the effective composite floor section area

CASE ONE Neutral axis is within the concrete.This case is associated with the condition $F_s > F_b$ which means a part of concrete is in compression. Tensile strength of the concrete in tension is not taken into account.

Depth of the compressive concrete is:

$$x_p = \frac{F_s - F_b}{0.4\ b_c\ f_{cu}} \tag{3}$$

Compressive force F_c of the concrete section is:

$$F_c = F_s - F_b \tag{4}$$

All the beam web and flange connected to the steel plate are in compression.
Depth of the centroid Z of the compressive portion (from the bottom flange of the beam) is:

$$Z = \frac{\frac{1}{2}(1^2 - t_{bf}^2)\, t_{bw} + \frac{1}{2} f_l\, t_{bf}^2}{(1 - t_{bf})\, t_{bw} + f_l\, t_{bf}} \tag{5}$$

Moment capacity M_p of the connection can be obtained by taking moments of the connection about the centroid of reinforcement as

$$M_p = F_b\,(D - Z + d_{cc}) + Fc(d_{cc} - \frac{1}{2} x_p) \tag{6}$$

in which D is the steel beam depth

d_{cc} is the effective depth of concrete slab

M_p can be calculated by substituting for the appropriate terms from formulae (1), (3), (4) and (5) into formula (6).

CASE TWO Neutral axis in the beam web.

This case is associated with the conditions (i) $F_b > F_s$ and (ii) $F_b - F_s < p_y P_1 t_{bw}$ which means the neutral axis is still beyond the first row of the bolts. P_1 is the edge distance of the first row bolts.

Depth of the web of the beam not in compression is:

$$x_p = \frac{F_b - F_s}{p_y\, t_{bw}} \tag{7}$$

x_p can be calculated by formulas (1), (2) and (7).

Depth of the centroid Z_1 of the compressive area (from the bottom flange of the beam) is:

$$Z_1 = \frac{\frac{1}{2}(1 - t_{bf} - x_p)\, t_{bw}(1 - x_p) + \frac{1}{2} f_l\, t_{bf}^2}{(1 - t_{bf} - x_p)\, t_{bw} + f_l\, t_{bf}} \tag{8}$$

Moment capacity M_p can be obtained by taking moments about the centroid of the compressive stress block

$$M_p = F_s\,(d_{cc} + D - Z_1) \tag{9}$$

M_p can be calculated by substituting appropriate terms (2), and (8) into formula (9).

CASE THREE Neutral axis is below the first row of bolts

As neutral axis moves down so certain rows of bolts will be in tension. Two thirds of the depth of the bolts' pitch will be assumed to be in tension below the row of the bolts adjacent to the neutral axis. The number of the bolts in tension m can be derived from the internal force equilibrium as follows:

$$F_s + 2\,A_t P_t m = p_y\ f_l\, t_{bf} + p_y\ t_{bw}(l - t_{bf} - x_p + (m - \frac{1}{3})P) \qquad (10)$$

The above equation can be rearranged to give:

$$m = \frac{p_y\ f_l\ t_{bf} - F_s + p_y\ t_{bw}(l - t_{bf} - x_p + \frac{1}{3}P)}{2A_t\, p_t + p_y\ t_{bw}} \qquad (11)$$

Depth of the centroid Z_2 of the compressive area (from the bottom flange of the beam) is determined from:

$$Z_2 = \frac{\frac{1}{2}(l - t_{bf} - P_1 - (m - \frac{1}{3})P)t_{bw}\,(l - t_{bf} - x_p - (m - \frac{1}{3})\,P\,) + \frac{1}{2} f_l\, t_{bf}^2}{(l - t_{bf} - x_p - (m - \frac{1}{3})\,P)t_{bw} + f_l\, t_{bf}} \qquad (12)$$

Moment capacity M_p of the connection can be derived by taking moments about the centroid of the compressive stress block.

$$M_p = F_s\,(d_{cc} + D - Z_2) + m\,2\,A_t P_t\,(l - Z_2 - P_1 - \frac{m-1}{2}P) \qquad (13)$$

M_p can be obtained by substituting the appropriate terms (2), (11), and (12) into the formula (13).

5. CONCLUSION

A test programme designed to systematically study the resistance moment and rotational capacity of composite connections has been described. The following conclusions can be drawn from the analyses of the tests for the flush end plate and partial depth end plate:

i) The initial stiffness, moment resistance and rotational capacity were dramatically affected by changes to the reinforcement in slab. Use of an appropriate ratio of reinforcement will improve the behaviour of the joint and change the form of the eventual failure.

ii) Variation of the position of the partial depth end plate can also change the strength and stiffness of the connection. Following the current practice for bare steel joints does not provide the best behaviour for the composite connection. Placing the endplate at the level of the lower flange is suggested such composite beam-column connection design.

iii) The presence of the column web stiffener not only prevents the failure in the column but also increases the moment capacity of the connection without having much influence on the rotation capacity of the beam-column connection. Beam stubs with endplates on the column web took over the role of web stiffeners to prevent local buckling of the column web.

iv) The moment capacity and rotational capacity were affected by the moment-shear ratio. The moment capacity was decreased under the condition of a high shear load.

The work described in this paper covers the first and second phases of the project, the third phase of the experimental work covers tests of minor axis connections and edge joints.

6. ACKNOWLEDGEMENT

The project was funded through a grant from the Building Research Establishment as part of their role in calibrating the forthcoming Eurocode4 for use in the U.K.. This financial support is gratefully acknowledged.

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Table.1 Speciemen details and test results

Specimen	Joint Type	Web Stiffening	Reinforcement Ratio(%)	First Crack Moment (kN• m)	Ultimate Moment (kN• m)	Ultimate Rotation (mRad)*	Maximum Rotation (mRad)	Failure mode*
SJ1	Seating cleat	None	None	—	39.5	> 59.8	—	A
SCJ1	Seating cleat	None	A142 mesh (0.2%)	15	43.1	14.3	21.7	B
SCJ2	Web side plate	None	A142 mesh (0.2%)	12	29.6	16.4	30.4	B
SCJ3	Flush end plate	None	A142 mesh (0.2%)	30	85.7	7.2	26.6	B
SCJ4	Flush end plate	None	T12 Rebar (1.0%)	40	202.9	23.4	41.1	C,D
SCJ5	Flush end plate	Web stiffeners	T12 Rebar (1.0%)	45	240.8	26	35	G,E
SCJ6	Flush end plate	None	T10 Rebar &A142 Mesh (1.0%)	36	157.6	11.5	23	C,D
SCJ7	Flush end plate	Plate stiffening	T12 Rebar & A142 Mesh (1.2%)	37.5	204.5	26.5	46.9	G,D
SCJ8	Partial depth end plate	None	T10 Rebar &A142 Mesh (0.8%)	22.5	84	29	44.5	F
SCJ9	Partial depth end plate	None	T10 Rebar &A142 Mesh (0.8%)	31.5	107.5	27	43.9	F
SCJ10	Partial depth end plate	None	T10 Rebar &A142 Mesh (0.8%)	35	147.8	16.5	30	C,D
SCJ11	Seating cleat	None	T10 Rebar &A142 Mesh (0.7%)	33.8	169.5	14.3	21.7	C,D
SCJ12	Web side plate	None	T10 Rebar &A142 Mesh (0.7%)	22.5	101.3	39	79	H

*Note: 1mRad = 0.05729 Degree

A --- Excessive deflection of beams
B --- Fracture of the mesh reinforcement
C --- Excessive deformation of column flange
D --- Buckling of column web
E --- Buckling of beam flange
F --- Buckling of beam web
G --- Shear studs failure
H --- Web side plate twisting

Table2 Comparison of calculated and test moment values

Specimen	Test Moment (kN• m)Mt	Calculated Moment (kN•m) Mc	Comparison Ratio Mt/Mc
SCJ3	85.7	52.3	1.64
SCJ4	202.9	153	1.33
SCJ5	240.8	266.4	0.90
SCJ6	157.6	153	1.03
SCJ7	204.5	182.7	1.34
SCJ8	84	80.1	1.05
SCJ9	107.5	80.1	1.34
SCJ10	147.8	170.4	0.88

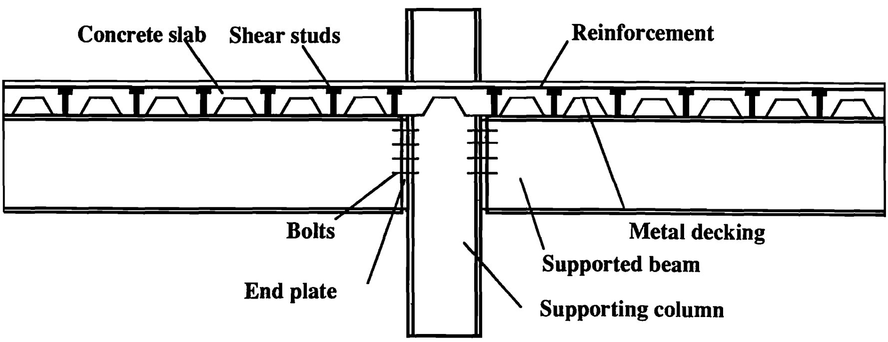
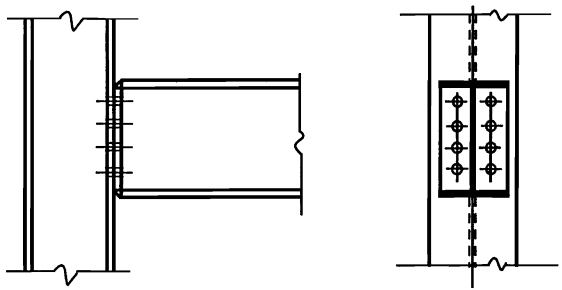
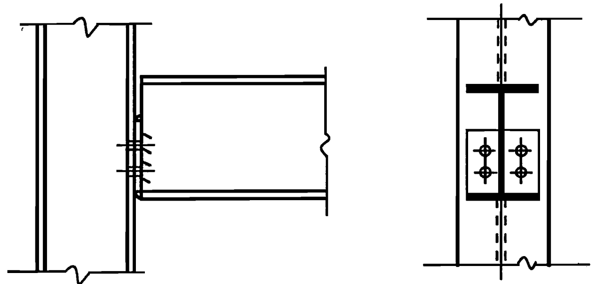


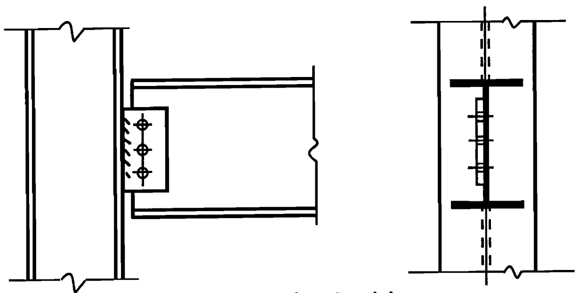
Fig.1. Specimen construction



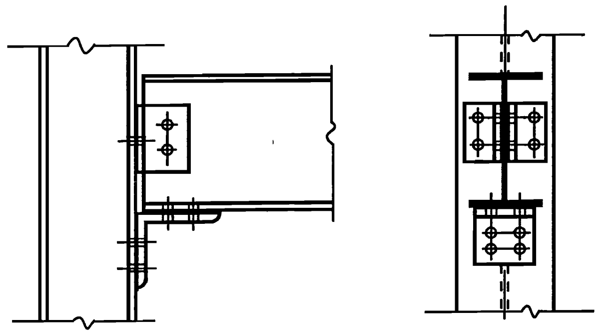
Type1 Flush end plate joint
(a)



Type2 Partial depth end plate joint
(b)



Type3 Web side plate joint
(c)



Type4 Seating cleat joint
(d)

Fig.2 Four Types of Steel Joint

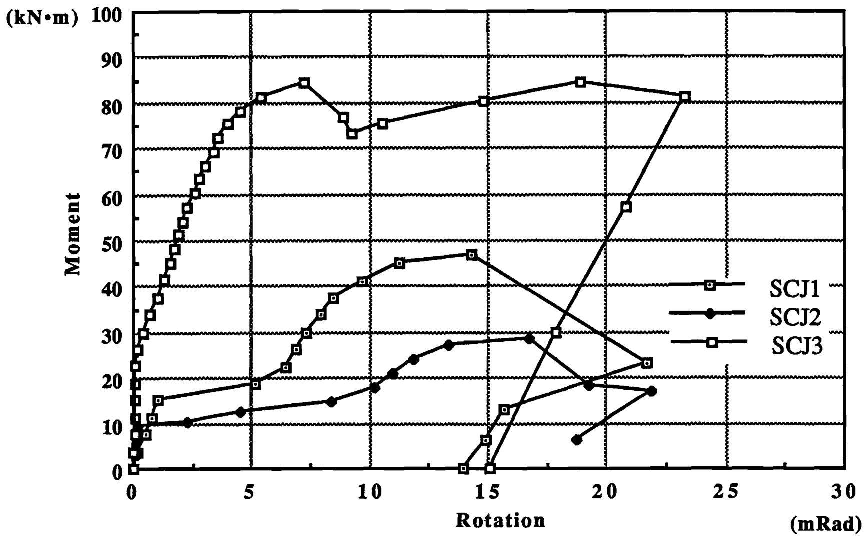


Fig.3 Moment-Rotation Relations of Different Steel Details With A142 Mesh(0.2%)

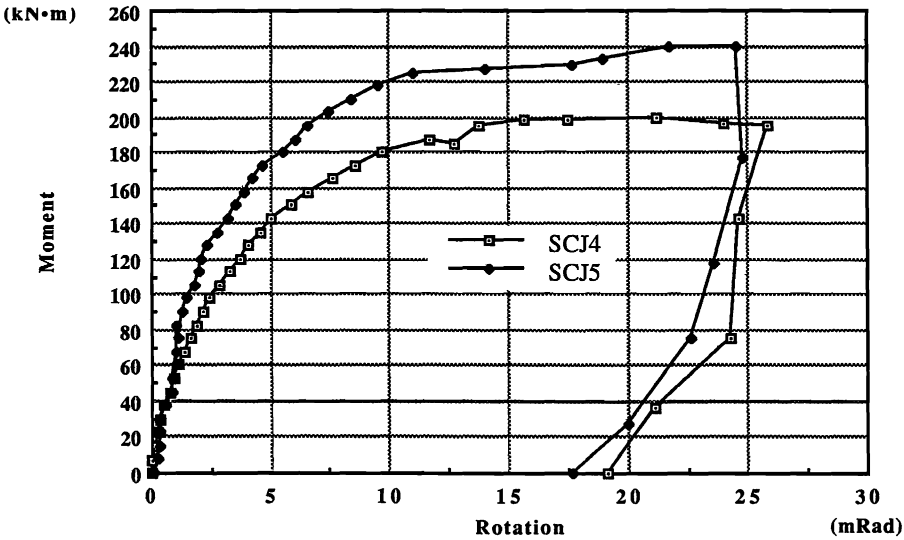


Fig.4 Effect of the presence of web stiffener on moment-rotation relation

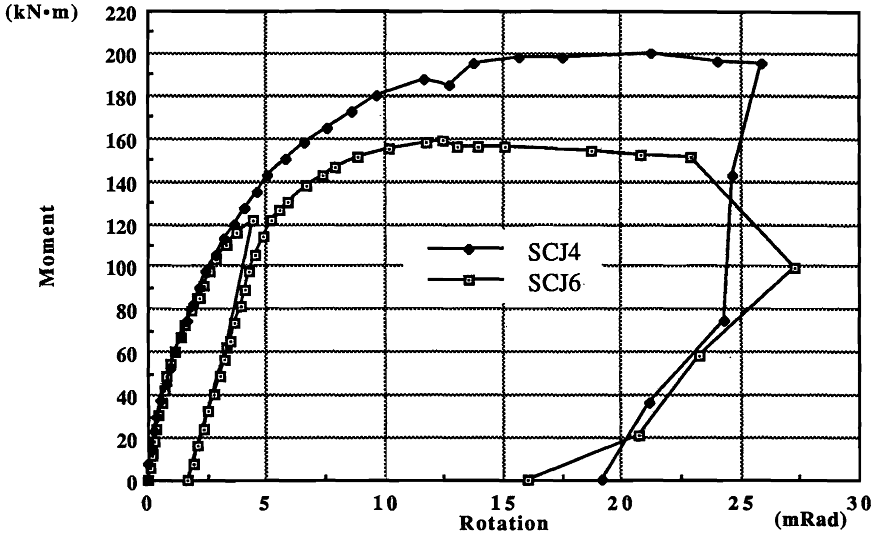


Fig.5 High shear force decreased the moment capacity of the flush end plate connection

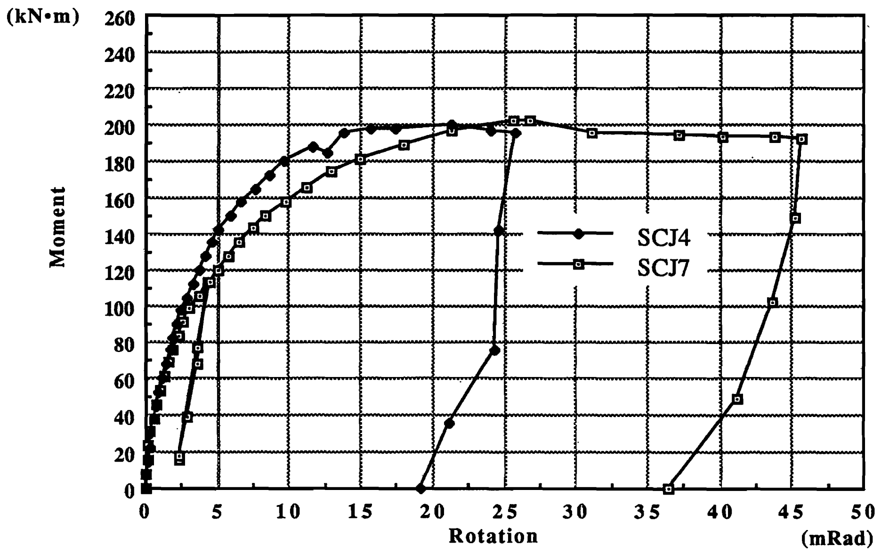


Fig.6 Influence of the beam stubs in orthogonal direction on the connection

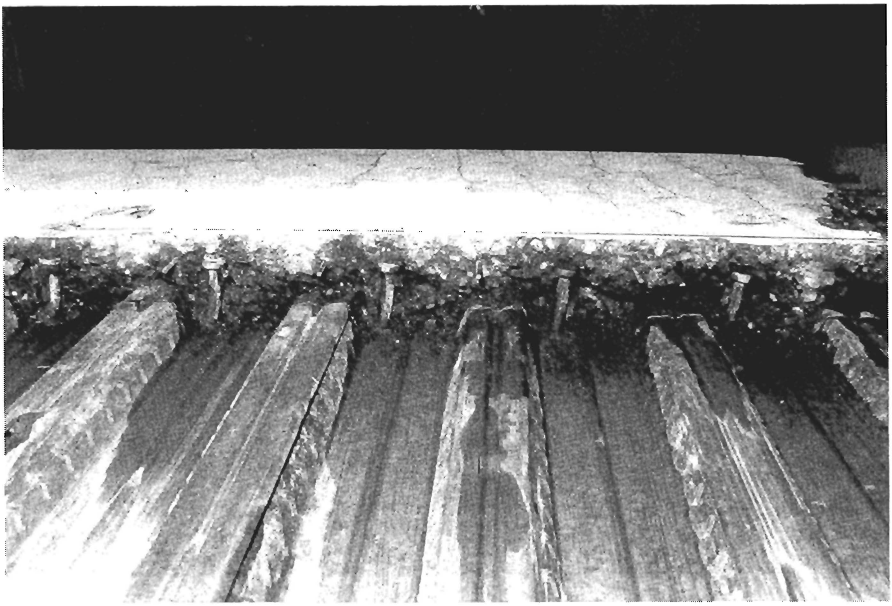


Fig.7 Failure in shear connection

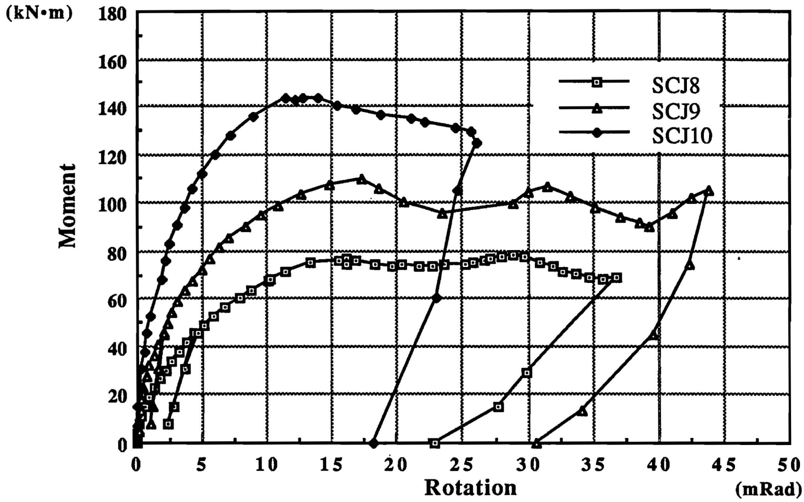


Fig.8 Influence of the locations of the partial depth end plate

Fig.9 Eventual Failure of beam by web buckling

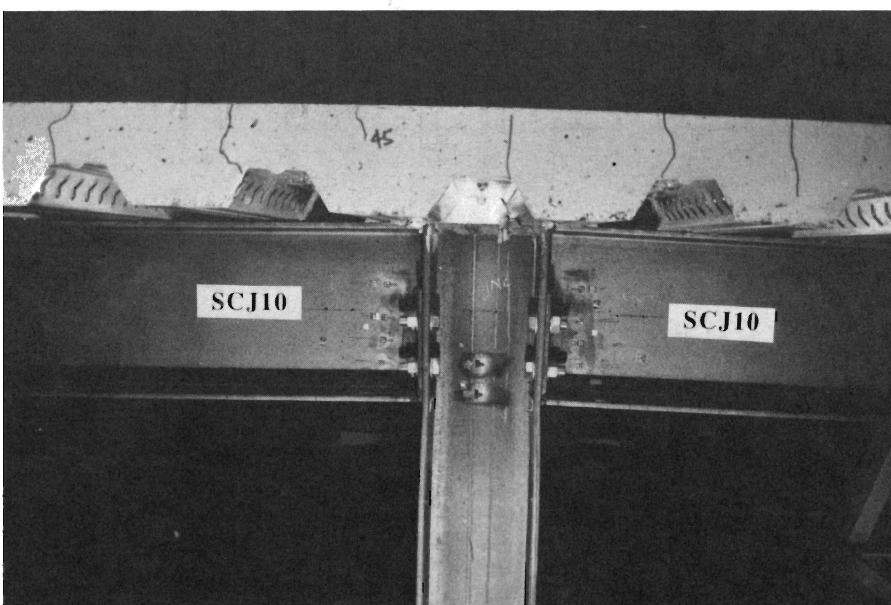


Fig.10 Eventual Failure of column by web buckling

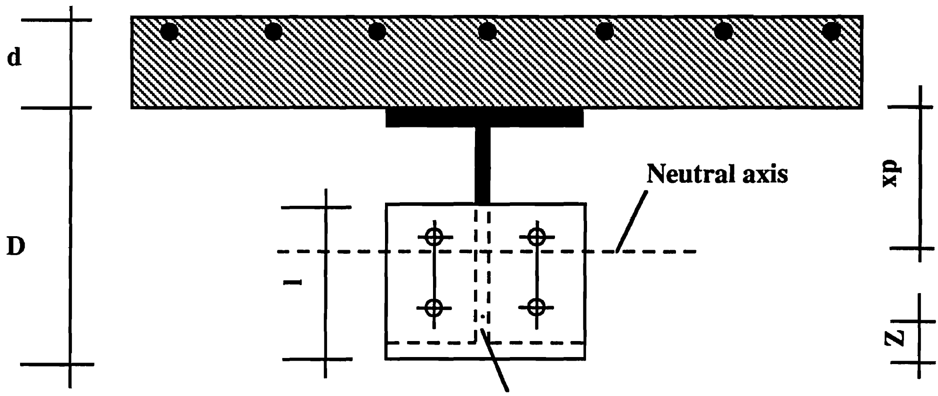


Fig. 11 Section of partial depth end plate connection