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The Local Web Buckling Strength of Stiffened Coped Steel I-Beams

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Abstract

In steel construction the flange of a steel I-beam is usually coped to allow clearance at the connection. The presence of a cope in a beam will reduce the strength of the beam in the coped region. Local web buckling at the coped region may occur when the cope length is long and/or the cope depth is large, provided that lateral-torsional buckling of the beam is prevented. Previous research recommended the provision of stiffeners at the coped region to improve the local web buckling strength of coped I-beams although no experimental evidence was provided to support the recommendation. In order to verify such recommendation, an experimental and numerical investigation of coped I-beams with stiffeners at the coped region was conducted and reported in this paper. The study showed that current recommendations did not consider the web distortion properly, i.e. local web buckling could not be prevented efficiently if only horizontal stiffeners were provided at the coped region. Both the test and the numerical results showed that the horizontal stiffeners at the cope displaced laterally due to gross web distortion. Based on the results of the parametric study of coped beams with different configurations of horizontal and vertical stiffeners, it was found that for cope depth to beam depth ratio (d_c/D) ≥ 0.3, both horizontal and vertical stiffeners are required in order to prevent local web buckling at the cope region. A preliminary recommendation for the design of coped beams with both horizontal and vertical stiffeners was proposed according to the key findings of the investigation.

Keywords: steel I-beams, copes, local buckling, stiffeners, strengthening, tests, finite element analysis

1. Introduction

In steel construction, when beams are connected to girders at the same elevation, beam flanges must be coped to provide sufficient clearance for proper attachments to supports as shown in Fig. 1. The cope can be at the top, the bottom, or at both flanges. When a beam is coped, the lateral-torsional buckling capacity of the beam, the shear capacity and the local web buckling capacity at the coped regions can be affected (Cheng, 1993). In addition, if the beam is not properly braced, the coped beam can fail in lateral-torsional buckling mode with much reduced strength (Cheng and Yura, 1988; Cheng and Snell, 1991, and Lam *et al*., 2000). However, if lateral bracing is provided at the vicinity of the copes or the cope length is sufficiently long, shear tearing or local web buckling at the coped region may occur. Investigation on the block shear tearing behavior at copes was conducted by Birkemoe *et al*. (1978) and Ricles *et al*. (1980). Cheng and Yura (1986) carried out both experimental and analytical studies on the local web buckling of coped I-beams. The effects of cope length and cope depth on the local web buckling capacities of coped beams were investigated. Design formulas were proposed for the local web buckling strengths of coped I-beams while improvement to the design formulas was provided by Yam, *et al*. (2003). In Cheng and Yura's original study (1984) on the local web buckling of coped beams, possible strengthening of coped region was studied. They recommended using stiffeners at the coped region to improve the local web buckling strength of coped I-beams. Three reinforcing details as shown in Fig. 2 are recommended. Reinforcing details of type (a) and (b) are recommended for all rolled sections or sections with a beam depth to web thickness ratio less than 60. Detail of type (c) is recommended for thin web members with a web slenderness ratio greater

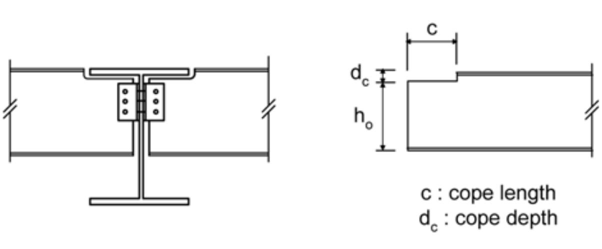

Figure 1. Typical top flange coped I-beam.

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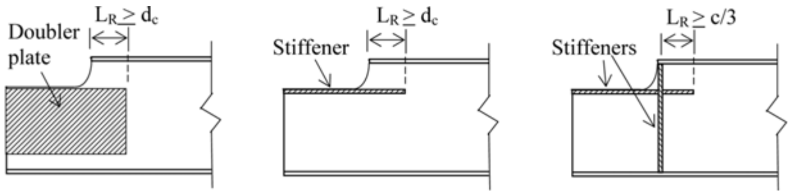


Figure 2. Web reinforcement details for coped beams.

than 60. However, no experimental evidence was provided so far to validate the recommendation. Therefore, in order to verify the strengthening details recommended by Cheng and Yura (1984), an experimental study of coped I-beams with horizontal stiffeners of type (b) at the cope was conducted and reported here. Three full-scale tests of top flange coped steel I-beams with d_c/D equal to 0.3 were conducted, and it should be noted that while horizontal stiffener were provided at the cope of two specimens, no horizontal stiffener was provided to the other. Finite element analyses of all the test specimens are also presented in this paper.

2. Test Program

Three tests were conducted to study the local web buckling (LWB) behavior of top flange coped I-beams with horizontal stiffeners. Welded beams with cross section dimensions closely similar to UB 406 × 140 × 39 (Steelwork 1990) were used as the test specimens. These UB sections are equivalent to the U.S. section W16 × 26. The nominal cope dimensions of the beams together with the measured dimensions (in bracket) are shown in Table 1. A single end plate was welded perpendicular onto the web of the coped beam end to be connected to the reaction wall. The sizes of all the end plates were carefully designed to provide sufficient shear strengths and at the same time minimized the in-plane rotational stiffness in order to simulate a simply supported boundary condition for the coped beam end. It was proposed by Cheng *et al.* (1984) that in order to prevent local web buckling of coped beams, horizontal stiffeners with extension of stiffener length, L_R, (Fig. 2) of not less than d_c ($L_R \geq d_c$) should be provided at the cope. Hence, two specimens, 406d03sa and 406d03sb with two types of stiffeners

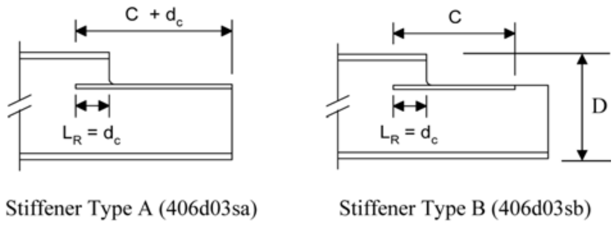


Figure 3. Coped beams with horizontal stiffeners.

(Details A and B) with a minimum extension of L_R equal to d_c were investigated, as shown in Fig. 3. The length of stiffener type A was equal to the cope length plus the cope depth whereas the length of stiffener type B was equal to the cope length only and with a clearance equal to d_c at the beam end, which was 25% shorter than stiffener type A. It should be noted that the stiffeners were welded on both sides of the web. The thickness of the stiffeners was approximately equal to the thickness of the flange while the width of the stiffeners in one side was equal to half of the width of the flange. In addition to the two specimens with horizontal stiffeners, one specimen (406d03) without stiffeners at the cope was also tested. Therefore, the local web capacity of the coped region with and without stiffeners could be compared. Tension coupons were prepared from all the test beams in the longitudinal direction and tested according to ASTM standard (A370-97a). The average tensile yield strength of the web was 343 MPa and the average elastic modulus of the web was 216,600 MPa.

The test setup is shown schematically in Fig. 4. Load was applied to the beam vertically upward. The load position was chosen not only to produce failure in the coped region but also to minimize the effect of the load itself on the stress distribution in the coped region. The

Table 1. Dimensions of test specimens

Specimen	d_c (mm)	D (mm)	B (mm)	t_w (mm)	t_f (mm)	c (mm)	d_c/D
406d03	119.4 (117)	398 (398)	142 (143)	6.0 (6.0)	8.0 (8.0)	342.9 (343)	0.3 (0.3)
406d03sa	119.4 (119)	398 (396)	142 (143)	6.0 (6.0)	8.0 (8.0)	342.9 (343)	0.3 (0.3)
406d03sb	119.4 (120)	398 (396)	142 (143)	6.0 (6.0)	8.0 (8.0)	342.9 (334)	0.3 (0.3)
Model 1	119.4	398	142	6.0	8.0	342.9	0.3 (0.3)
Model 2	119.4	398	142	6.0	8.0	497.5	0.3 (0.3)

Values in bracket indicate measured dimensions
d_c = coped depth, D = beam depth, B = flange width, t_w = web thickness, t_f = flange thickness
c = cope length

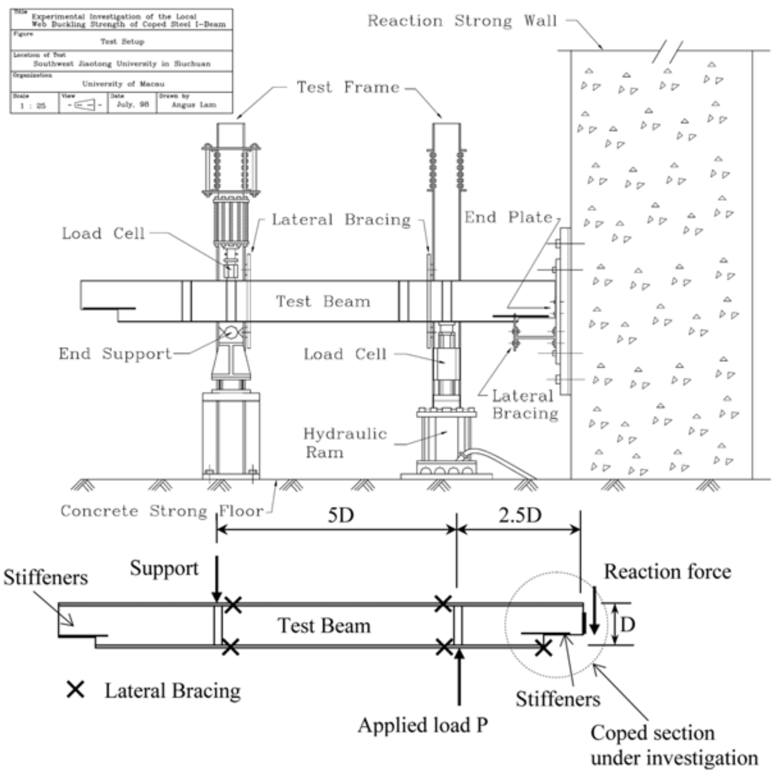


Figure 4. Test setup.

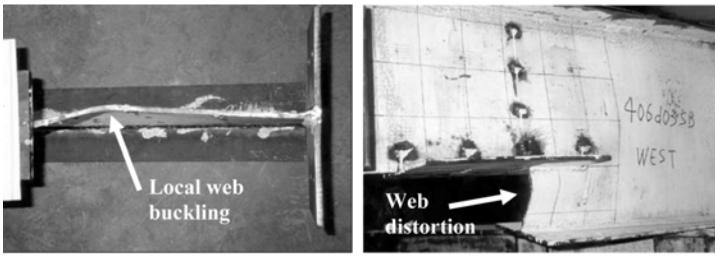
distance from the load position to the end plate was 995 mm, i.e. 2.5D approximately. Lateral bracings were provided at the load position, the pin support and the end of the cope on the flanges in order to prevent lateral movement. Since local web buckling was a local behavior, the other end of each specimen was used for another test as a new specimen once the end that was connected to the strong wall failed.

Load was applied quasi-statically and was controlled by an electro-hydraulic servo controller. At the far end support, load cell was used to measure the reaction force. The magnitude of the applied load and the far end reaction force were recorded and transferred to a data acquisition system. To measure the in-plane deflections, dial gauges were set up at the load position and at the end of the cope. Another set of dial gauges was set up on the web to measure the lateral deflection of the web. In addition, longitudinal strain gauges were mounted onto the web at the cope to measure the strain distribution of the web at the cope as well as to detect any buckling of the web.

The test procedure was generally the same for all the tests. The loading procedure was divided into two stages. In the first stage, load was applied by controlling the magnitude of the loading (i.e. load control). After the applied load reached approximately 50% of the expected maximum load, load was applied by controlling the magnitude of the vertical deflection at the loaded point (i.e. stroke control). At this stage, the vertical deflection at the loaded point was applied in increments. Load was applied until the post-buckling curve was obtained.

3. Test Results

The failure mode of the specimen without stiffeners at the cope was identified to be local web buckling at the cope region as shown in Fig. 5. The failure mode of the specimens with horizontal stiffeners was identified to be web distortion occurring at the web region between the top flange (the beam was loaded upside down) and the horizontal stiffener which exhibited significant lateral displacement (Fig. 5). The ultimate loads of the specimens (P_{Test}) are summarized in Table 2. The load versus in-plane vertical deflection curves (Fig. 6) show that linear behavior exists before buckling. This probably indicates that the specimens do not experience significant yielding prior to instability failure. The load deflection behaviors of these specimens were similar in that no sudden drop of loading was observed and the post buckling behavior was generally stable. It can be seen from the figure that the applied load decreased gradually when the vertical deflection of the loaded point increased during unloading. The test results showed that the ultimate loads of the specimens with horizontal stiffeners were larger than that of the specimen without stiffeners by about 50%. Meanwhile, the ultimate load of the specimen with stiffener type B was almost the same as that of the specimen with stiffener type A even though the length of type A stiffener was 25% longer than that of type B stiffener. The load versus out-of-plane displacement for specimen 406d03sa is shown in Fig. 7. The figure shows that a significant out-of-plane displacement occurred near



Specimen 406d03 Specimen 406d03sb

Figure 5. Buckled mode shapes of specimens 406d03 and 406d03sb at coped region.

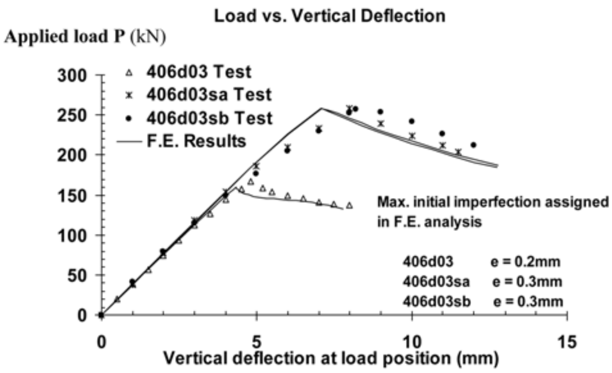


Figure 6. Comparison on load vs. vertical deflection curves between numerical and test results.

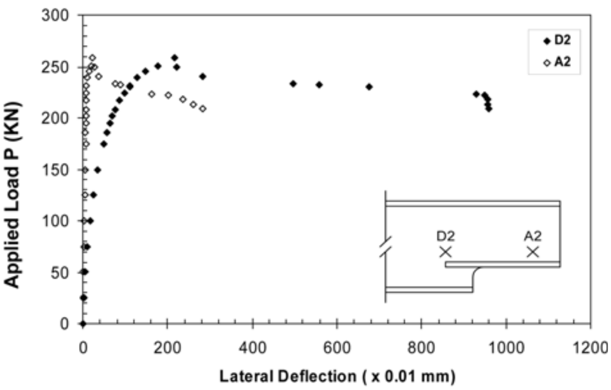


Figure 7. Test results of load versus out-of-plane deflection of points A2 and D2 of 406d03sa.

the inside end of the stiffener, indicating significant web distortion near the cope end.

The strain profile for specimen 406d03sa near the cope region at an applied load of 100 kN is shown in Fig. 8. As shown in the figure, the maximum strain occurred near the end of the stiffener (section D). The neutral axis of both the original I-section and the coped section (with stiffeners) were also plotted on the same figure for illustration purpose. The strain readings at sections A and B showed that the location of zero strains occurred very close to the theoretical neutral axis of the coped section. This illustrated that the stiffeners were able to act effectively as the bottom flange of the beam section.

4. Finite Element Analysis of Test Specimens

The test specimens were analyzed by the finite element method using the finite element program, ABAQUS (2004). Four nodes shell elements (S4R) with six degrees of freedom at each node were used to model the test beams. Typical model of the test beam is shown in Fig. 9. Tri-linear material stress-strain curve according to the tensile coupon test results was used in the material models. The load-deflection analysis included both the material and geometric nonlinearities. To consider geometric non-linear effect, initial imperfection which was based on the first mode of the local web buckling at the cope was used in the non-linear analysis. The maximum initial imperfection e used in this analysis was 0.3 mm. This initial imperfection was less than the limit of the maximum tolerance of a web plate specified in Section 7 of BS5950: Part 2 (2001). Comparison of the evaluated buckling loads obtained from the finite element analyses and the test are shown in Table 2. The test-to-predicted load ratios (P_{Test} /P_{FEM}) are found to vary from 1.00 to 1.05. The load versus in-plane deflection curves of the specimens obtained from the finite element analysis were compared with the test results as shown in Fig. 6.

In general, it is found that the analytical load (loading and unloading) versus deflection curves compare well with the test results. For the specimens with horizontal stiffeners, it was shown from the test results that even though horizontal stiffeners with $L_R = d_c$ were provided at the cope, yielding of the web or the stiffeners did not govern the design. Typical finite element results of the deformed shape and the von Mises stress contour plot of specimen 406d03sb are shown in Fig. 10(a) and (b). The non-linear finite element results showed that the horizontal stiffeners at the coped regions displaced side way due to web distortion. Same behavior was observed in the test program as well. In addition, a relatively high stress level was observed at the coped location underneath the horizontal stiffener.

5. Effects of Horizontal Stiffeners

When it was necessary to prevent local web buckling of coped steel I-beams, one of the methods proposed by

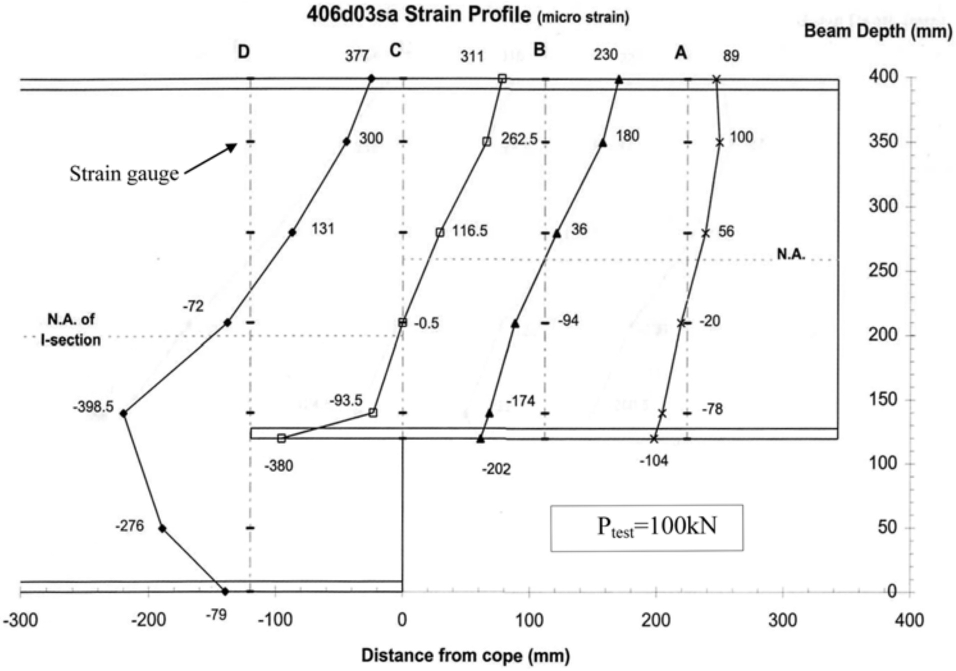


Figure 8. Test results of strain profile for specimen 406d03sa near the cope.

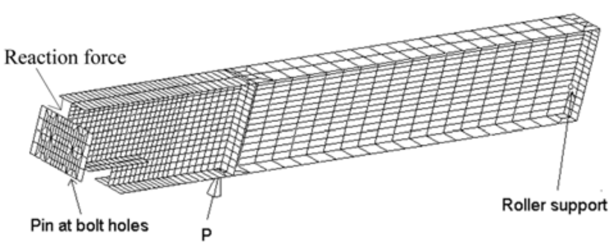


Figure 9. Finite element model of a stiffened coped I-beam.

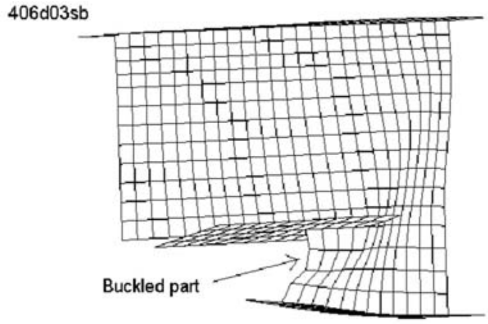
Cheng *et al*. (1984) was to provide horizontal stiffeners at the cope. However, it was shown from the test results that the horizontal stiffeners at the cope might displace laterally due to web distortion when the cope depth was too large. Therefore, in the analytical study, two different types of stiffeners were examined. The first type of stiffeners only included horizontal stiffeners while the second type of stiffeners included both horizontal and vertical stiffeners (Fig. 11). The use of the vertical stiffeners was believed to be able to prevent the side way movement of the web and the horizontal stiffeners. It was shown previously in the test program that the provision of stiffener type B, which was 25% shorter than the full length stiffener (stiffener type A) did not decrease the ultimate load. Therefore, in the first study, stiffener type B was used as the basic model. Two models of stiffener type B with different cope lengths, c = 342.9 mm (c/D = 0.86) and c = 497.5 mm (c/D = 1.25) were studied, of which the cope depth d_c was equal to D/3. The dimensions of the analytical models, which were similar to that of specimen 406d03sb, are shown in Table 1. The yield strength and the modulus of elasticity of the steel for the model were assumed to be 350 MPa and 205,000 MPa, respectively.

The length of the horizontal stiffeners was extended (by extending L_R) in order to increase the local web buckling capacity of the coped section. Typical finite element model is shown in Fig. 12. The distance between the point load and the cope end is 2.5D where D is the beam depth. Lateral bracings were provided at the load position, the pin support and the end of the cope on the top and the bottom flanges. Elastic buckling analysis was carried out first in order to obtain the first buckling mode shape of the coped I-beam. Then, non-linear analysis that incorporated the first buckling mode shape with a maximum initial imperfection equal to 0.15 mm (= 2.5% of t_w) as the initial geometry of the cope I-beam, was carried out. The results are shown in Table 3. For the model with c/D =

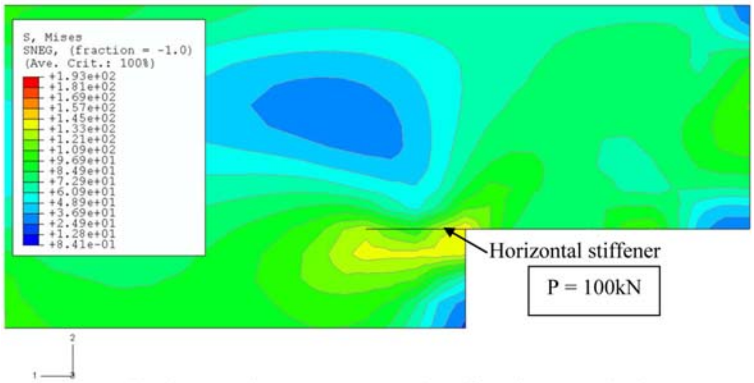
Table 2. Comparison between test results and finite element results

Specimen	P_{Test} (kN)	R_{Test} (kN)	P_{FEM} (kN)	R_{FEM} (kN)	P_{Test} / P_{FEM}
406d03	167.9	111.9	159.6	106.4	1.05
406d03sa	258.1	172.1	258.2	172.1	1.00
406d03sb	259.8	173.2	258.3	172.2	1.01

P_{FEM}-ultimate load from the finite element analysis

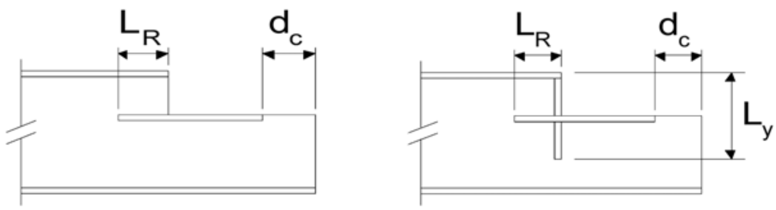


(a) Predicted buckled mode shape of specimen 406d03sb



(b) The von Mises stress contour plot of specimen 406d03sb

Figure 10. Finite element results of specimen 406d03sb.



Horizontal stiffeners Horizontal and vertical stiffeners

Figure 11. Two different types of stiffeners at the cope.

0.86, the extension of horizontal stiffeners (L_R) was assigned to vary from d_c to $5.5d_c$ (119.4 to 656.7mm). It can be seen from Table 3 that when L_R was equal to $5.5d_c$, the increase in the ultimate reaction force R_{FEM}, was about 28%, when compared with that of the specimen with $L_R = d_c$. For the model with $c/D = 1.25$, it can be seen from Table 3 that when $L_R = 4d_c$, the increase in the ultimate reaction force R_{FEM}, was about 18%, when compared with that of the specimen with $L_R = d_c$. In fact, for both cases, the ultimate reaction forces R_{FEM}, with the longest horizontal stiffeners were still significantly lower than their corresponding yield loads (first yield) at the junction of the coped and the un-coped sections. Hence, it is concluded that increasing the length of horizontal stiffeners might not increase the local web buckling capacity of coped beam efficiently. This result contradicted with the recommendation proposed by Cheng *et al*. (1984) which stated that yielding would control the capacity of the coped section if extensions of horizontal stiffeners with a length of not less than the depth of cope (d_c) were provided at the cope. The reason for this disagreement is that Cheng *et al*. (1984) used in their study a beam section with a larger d_c/D ratio of 0.5, as opposed to 0.3 in the present study. Hence, the yield moment capacity of the stiffened coped section was quite small. Therefore, the local buckling capacity of stiffened coped sections could be easily larger than its corresponding yield moment capacities and hence, the horizontal stiffeners were effective in strengthening the coped sections with large cope depths.

6. Effects of Horizontal and Vertical Stiffeners

Since it was found that increasing the length of horizontal stiffeners did not increase the local web buckling capacities of coped beams efficiently, it was decided to provide

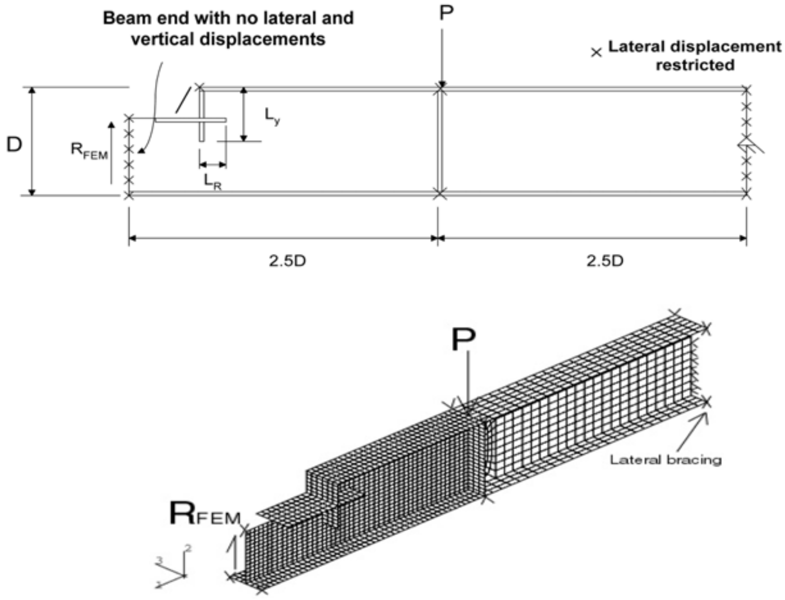


Figure 12. Finite element model of coped I-beam with stiffeners.

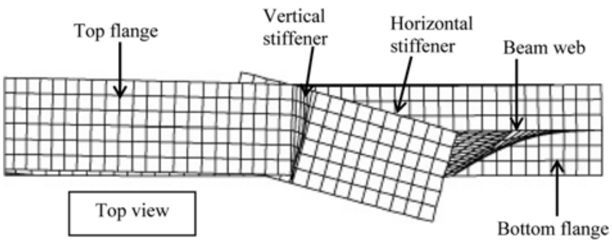


Figure 13. Buckled mode shape of coped I-beam with both horizontal and vertical stiffeners.

vertical stiffeners together with horizontal stiffeners at the cope based on the observed failure mode of the test specimens. For the model with horizontal and vertical stiffeners, a minimum extension of horizontal stiffeners, i.e. $L_R = d_c$, was used (same as the type B stiffener) and the length of vertical stiffeners was assigned to vary from zero to 238.8 mm (0 to $2d_c$). The finite element results are shown in Table 4. The general failure mode of the coped beams stiffened by both horizontal and vertical stiffeners was still sideway displacement of the horizontal stiffeners as shown in Fig. 13. However, the horizontal stiffeners were anchored by the vertical stiffeners, and therefore, rotation of the horizontal stiffeners about the longitudinal axial of the vertical stiffeners occurred as illustrated in Fig. 13.

For the model with c/D = 0.86, it can be seen that when vertical stiffeners with a length equal to the cope depth ($L_y = d_c$) was used together with the horizontal stiffeners

Table 3. Effects of providing horizontal stiffeners

c/D = 0.86			c/D = 1.25		
L_R (mm)	L_R / d_c	R_{FEM} (kN)	L_R (mm)	L_R / d_c	R_{FEM}(kN)
119.4	1	173.0	119.4	1	161.0
358.2	3	189.4	238.8	2	162.0
417.9	3.5	194.6	358.2	3	170.8
537.3	4.5	206.2	477.6	4	189.6
656.7	5.5	219.5			

Table 4. Effects of providing horizontal and vertical stiffeners (L_R = d_c)

c/D = 0.86			c/D = 1.25		
L_y (mm)	L_y / d_c	R_{FEM} (kN)	L_y (mm)	L_y / d_c	R_{FEM} (kN)
0	0	173.0	0	0	161.0
119.4	1	217.0	119.4	1	172.0
238.8	2	226.0	238.8	2	178.0
			382.0	3.2	183.0

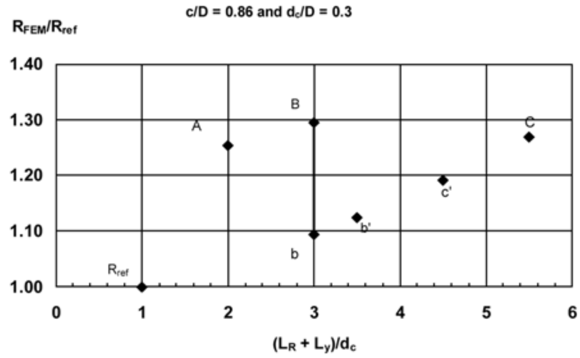
Table 5. Effects of providing horizontal and vertical stiffeners in specimen with c/D =1.25 and L_R = d_c, $2d_c$, $3d_c$ and $4d_c$

	R_{FEM} (kN)			
L_R / d_c	L_y / d_c = 0	L_y / d_c = 1	L_y / d_c = 2	L_y / d_c = 3.2
1	161.0	172.0	178.0	183.0
2	162.0	195.9	201.4	204.6
3	170.8	207.0	216.8	222.0
4	189.6	223.3	226.0	226.0

of a minimum extension ($L_R = d_c$), the ultimate load of the coped section was increased by 25%, when compared with the one with horizontal stiffeners ($L_R = d_c$) only. On the other hand, the ultimate load of the beam with both horizontal and vertical stiffeners ($L_y = d_c$ and $L_R = d_c$) was 15% higher than that of the beam with only horizontal stiffeners of a length equal to $3d_c$. When the length of the vertical stiffeners was increased to $2d_c$ (= 238.8 mm), the corresponding reaction was increased to 226 kN. This reaction force was about 31% larger than that of the model with only horizontal stiffeners of $L_R = d_c$. Although this reaction force was only 3% larger than the reaction force of the model with horizontal stiffener $L_R = 5.5d_c$, about 45% less of the stiffener material was used in the latter model, and of course, this would also reduce the length of welding.

For the model with a longer cope length (c/D = 1.25), the results of the ultimate reaction force with horizontal stiffeners ($L_R = d_c$) and vertical stiffeners ($L_y = d_c$ to $3.2d_c$) are shown in Table 4. It can be seen that when the vertical stiffeners were provided for approximately the whole depth of the beam ($L_y = 3.2d_c$), the ultimate reaction force was 183 KN which was about 14% larger than that of the specimen with horizontal stiffeners only ($L_R = d_c$). In addition, for longer cope length, increasing the length of vertical stiffeners did not increase the ultimate capacity of the coped section significantly when $L_R = d_c$ was provided. Meanwhile, this reaction force was still lower than the reaction force corresponding to the yield moment capacity and the shear yielding of the web at the coped section. Hence, it was decided to conduct further study on the effects of varying the lengths of both the horizontal and the vertical stiffeners in order to achieve the plastic or at lease the first yielding moment capacity of the coped section, or the shear yielding resistance of the web.

The local web buckling capacity of the beam with c/D = 1.25 and d_c/D = 0.3 was studied with various combinations of the length of horizontal and vertical stiffeners. The value of L_R/d_c was varied from 1 to 4 while that of L_y/d_c was varied from zero to 3.2. Non-linear finite element analysis was carried out, and the results are shown in Table 5. By using the ultimate reaction force of the model with only horizontal stiffeners as the reference value ($L_R/d_c = 1$ and $L_y/d_c = 0$, R_{ref}= 161 kN), the ratios of the ultimate reaction force to the reference value versus L_y/d_c with various L_R/d_c ratios are shown in Fig. 10. The figure illustrates that with a fixed value of L_R, the ultimate

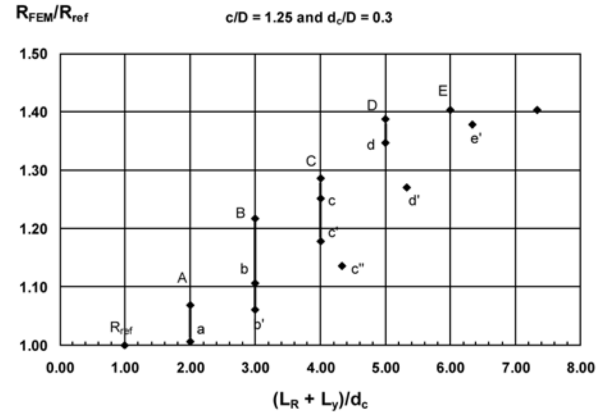


	R_{ref}	A	B	C	b	b'	c'
L_R/d_c	1	1	1	5.5	3	3.5	4.5
L_y/d_c	0	1	2	0	0	0	0

Figure 14. Effects of total length of stiffeners ($L_R + L_y$) on the local web capacity of coped section (c/D = 0.86 and d_c/D = 0.3).

reaction force increased as the length of L_y increased. Meanwhile, it is also noted that the rate of increase in the ultimate reaction force was different for different values of L_R. For $L_y = 3.2d_c$, the local web buckling capacity of the cope section with $L_R = 3d_c$ was about 10% larger than that of the specimen with $L_R = 2d_c$. However, when L_R was larger than $3d_c$, the increase in the local web buckling capacity of the coped section became insignificant. The increase in the ultimate load capacity was about 1.4% when L_R was increased from $3d_c$ to $4d_c$ with $L_y = 3.2d_c$. On the other hand, it can also be seen that providing vertical stiffeners could increase the local web buckling capacity of coped section significantly, especially when L_y was increased from zero to d_c and $L_R/d_c \geq 2$.

In order to obtain a good combination of the lengths of the horizontal and the vertical stiffeners for the best improvement in the local web buckling capacity of the coped section, the ratio of the ultimate reaction force to the reference value (R_{FEM}/R_{ref}) is plotted against the ratio $(L_R + L_y)/d_c$ as shown in Figs. 14 and 15. Both figures show that when the total length of the stiffeners, i.e. the length of horizontal stiffeners plus the length of vertical stiffeners, was maintained the same, the provision of both vertical and horizontal stiffeners simultaneously was more efficient than extending the length of horizontal stiffeners only. Obvious increase in the local web buckling capacity of the coped section was obtained when a minimum length of vertical stiffeners ($L_y = d_c$) was provided (point



	R_{ref}	A	B	C	D	E	F
L_R/d_c	1	1	2	3	4	4	4
L_y/d_c	0	1	1	1	1	2	3.2

	a	b	b'	c	c'	c''	d	d'	e'
L_R/d_c	2	1	3	2	4	1	3	2	3
L_y/d_c	0	2	0	2	0	3.2	2	3.2	3.2

Figure 15. Effects of the total length of stiffeners (LR + Ly) on the local web capacity of coped section (c/D = 1.25 and dc/D = 0.3).

A in Fig. 14 and points A, B, C and D in Fig. 15). On the other hand, Figure 15 shows that extending the vertical stiffeners to the whole depth of the beam did not seem to have significant effect in improving the local web buckling capacity of the coped sections (points c'', d' and e' in Fig. 15). However, it should be noted that the improved local web buckling strength of the coped section was still lower than the reaction forces corresponding to the yield moment capacity and the shear yielding resistance of the web at the cope.

7. Summary and Conclusions

The local web buckling behavior of coped I-beam with stiffeners in the coped region was investigated both experimentally and numerically. In the experimental program, three full-scale tests of the local web buckling strengths of coped steel I-beams were carried out. Two of them were used to study the effects of horizontal stiffeners while the other one was designed to investigate the local web buckling strength of the coped sections without stiffeners for comparison. It was shown from the test results that web distortion occurred at the coped section even when horizontal stiffeners were provided. This indicated that the failure mode of the test specimens with stiffeners was stability failure instead of material failure. This result contradicted with the recommendation proposed by Cheng *et al*. (1984) which stated that yielding would control the capacity of the coped section if extensions of horizontal stiffeners with a length of not less than the depth of cope (d_c) were provided at the cope. The reason for this disagreement may lie in the fact that the cope depth (d_c) of the beam section tested by Cheng *et al*. (1984) was large ($d_c/D = 0.5$) but the one used in this study was relatively smaller ($d_c/D = 0.3$). On the other hand, the test results illustrated that reducing the length of horizontal stiffeners (up to 25% reduction of the length) but keeping the minimal extension length equal to d_c did not decrease the local web buckling capacity of the coped section.

A numerical study on coped beams with stiffeners at the coped section was also carried out with the finite element program, ABAQUS (2004). Non-linear finite element analyses, in which the material non-linearity, geometric non-linearity and initial imperfection were taken into account, were conducted to study the effects of providing horizontal stiffeners only and providing both horizontal and vertical stiffeners on the strengths of stiffened coped I-beams. The study was conducted with a coped beam model (W16 × 26) with dc/D equal to 0.3 and c/D equal to 0.86 and 1.25. Based on the limited test data and the numerical results, the following conclusions on the effects of stiffeners are drawn:

For coped beams with horizontal stiffeners only, extending the length of the horizontal stiffeners did not increase the local web capacity efficiently.

Obvious increase in the local web buckling capacity of the coped section was obtained when a minimum length of vertical stiffeners ($L_y = d_c$) was provided together with horizontal stiffeners.

Extending the vertical stiffeners to the whole depth of the beam did not have significant effect in improving the local web buckling capacity of the coped section.

The local web buckling strength of the coped section stiffened by both the horizontal and the vertical stiffeners was still lower than the reaction force corresponding to the yield moment capacity and the shear yielding resistance of the web at the cope.

When the cope length to beam depth ratio is smaller than 1 ($c/D < 1$), both the extension length of the horizontal stiffeners (L_R) and the length of the vertical stiffeners (L_y) should not be less than d_c for effective improvement on the local web buckling strengths of coped beams.

When the cope length to beam depth ratio is larger than 1 ($c/D > 1$), both the extension length of the horizontal stiffeners (L_R) and the length of the vertical stiffeners (L_y) should not be less than $2d_c$ for effective improvement on the local web buckling strengths of coped beams.

The study presented in this paper only provided some preliminary conclusions and recommendations. In order to develop a complete set of design guidelines for the local web buckling strength of stiffened coped I-beams, further experimental and analytical studies are required.

List of symbols

c	cope length of steel I-beams
d_c	cope depth of steel I-beams
D	total depth of steel I-beam sections
e	maximum initial imperfection of beam web in finite element analysis
h_0	depth of coped section of steel I-beams

L_R extension length of horizontal stiffeners at the coped region
L_y total length of vertical stiffeners at the coped region
P concentrated load at the loading point of beam specimens
P_{FEM} ultimate loads of the specimens in the FE analysis
P_{Test} ultimate loads of the specimens in the test
R ultimate reaction forces at the connection of the coped end of beams
R_{FEM} ultimate reaction in the FE analysis
R_{ref} test ultimate reaction of the specimen without any stiffener
t_f thickness of the flange of the I-beams
t_w thickness of the web of the I-beams

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