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Lateral Buckling of Welded Beams and Girders in HT 80 Steel

Déversement de poutres soudées en acier HT 80

Kippen von geschweissten Balken aus HT 80 Stahl

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1. INTRODUCTION

Lateral instability of compression flange has been one of the important limit state criteria for the determination of the ultimate bending strength of flexural members when the compression flange is not restrained enough against lateral deflection. The author has shown the results of the investigation on the ultimate strength of beams with lateral bracing at the intermediate points[1)2)] and without any lateral supports in between[3),4),5)] from theory and test.

This paper briefly describes the lateral buckling test[3)] on welded beams and girders with steels of SM 50 and quenched and tempered HT 80 (nominal yield stresses $\sigma_y = 3200\text{kg/cm}^2$ and 7000kg/cm^2, respectively), and the initial imperfections such as welding residual stresses and initial deformations are discussed with the test results. The comparisons are also made between theory and test.

2. THE TEST SPECIMEN

No lateral bracings are provided except at the both ends of the specimen where the loading beams with heavy box cross section are connected using high strength bolts. The end condition of the specimen is thus clamped laterally and torsionally at the both ends. The test setup is shown in Fig. 1.

The detailed nominal dimension of the beam-type specimens are given in Table 1(a). The beam types are of 25cm or 30cm in beam height with the span length ranging from 2.5m to 4.5m. Total number of beam specimens is thrity-six of which

SM 50 : twenty-one including nine annealed beams, and
HT 80 : fifteen including three annealed beams.

Table 1(b) shows for the plate girders of 80cm or 100cm in height with the span length ranging from 2.8m to 4.0m. Number of sub-panels in web is two for G-D and the others have three sub-panels.

The specimens are tested under uniform bending and under moment gradient with the end moment ratio of 0.5.

3. INITIAL IMPERFECTIONS

After the test specimen is set in the position for testing and just prior to loading, the initial imperfections are measured in flanges and web plates. Table 2 summarizes the maximum lateral deflections of the compression flange δ_u and tension flange δ_ℓ at the span center. Effective length of the specimens against lateral buckling becomes $L_e = L/2$ (L=length of the specimen) under uniform bending since the clamped end condition is met at the connections of the specimens and the loading beams.

4. WELDING AND ANNEALING DATA

Welding data for HT 80 steel specimens are as follows ;

For beams : Manual welding with 5mmϕ electrode (YAWATA L80). 180 - 230 amp. current and preheating at 120°c.

For girders : Manual welding with 4mmϕ electrode (KAWASAKI KS116, B-1). 165 - 175 amp. current and preheating at 120°c.

Annealing conditions are set for relieving the welding residual stress as shown in Fig. 2.

5. THE TEST RESULTS

a) Tension Coupon Test

Test coupons with 20cm gage length for SM 50 and with 5cm gage length for HT 80 are cut out from the flange and web plates. The averaged values of static yield stress are listed below.

Beams { SM 50 : 3428 kg/cm^2
HT 80 : 7841 "

Girders { SM 50 : 3810 kg/cm^2 (8mm) and 3236 kg/cm^2 (10mm)
HT 80 : 7850 " (10mm) and 6610 " (6mm)

b) Lateral Buckling Test

Figs 3a and 3b show the load-deflection curves of a beam and a girder in HT 80, respectively, in which β = angle of rotation of the compression flange, u_1 and u_2 = **lateral deflection** of lower and upper flanges and v = vertical deflection at the span center. At the early stage of loading, the lateral and torsional deformations of the girder flanges are observed together with the buckled patterns in each web panel. Lateral deflection of the compression flange at failure is influenced considerably by the direction of the buckled deformation of the web in bending.

A summary of test results with the reference loads is listed in Table 3.

Fig. 4 shows a presentation of the test points plotted on the σ_{cr} - $(r_x/r_y)(L_e/d)$ axes[3], in which r_x and r_y are the radii of gyration of cross section about x and y axes, respectively and d = the beam height. The test points for HT 80 indicate less scatter and the reduction of lateral buckling strength becomes less for HT 80 specimens compared with SM 50 specimens in the inelastic range.

Fig. 5 shows another presentation of the test points plotted for beam-type specimens on the σ_{cr}/σ_y - $(L_e/r_y)/(L_e/r_y)_{elastic}$ axes. Non-dimensionalized slenderness ratio λ is taken in the abscissa whereas $(L_e/r_y)_{elastic}$ is the slenderness ratio at which the elastic lateral buckling stress reaches to the yield point, that is, $\sigma_u = \sigma_y$.
The maximum experimental moments M_u are non-dimensionalized by the yield moment M_y. In order to clarify the effect of residual stress distributions on the lateral buckling strength of steel beams, the comparisons are made between the annealed beams and the as-weld beams having the same sizes. Twelve pairs of specimens are tested and by taking the strength ratios $M_{annealed}/M_{as\text{-}weld}$ for each pair, it is obtained that the average strength gains are of 11% for annealed beams compared to the as-weld beams for SM 50 and 6% for HT 80 steel. It may be concluded that the welding residual stress distributions may reduce the lateral buckling strength for about 11% average for SM 50 and 6% average for HT 80 against the beams without residual stresses. Experimental findings may prove that the residual stress effect upon the lateral buckling strength becomes less with the higher yield strength steel.

6. COMPARISONS OF THEORY AND TEST

a) Inelastic Lateral Buckling Strength

Inelastic lateral buckling strength under uniform moment is determined for the arbitrary residual stress patterns using the numerical iteration technique for computing the cross sectional properties.[5) 7)] In Table 4 the calculated ideal critical elastic moment M_E and inelastic moment M_{cr} for the two different residual stress patterns (A) and (B) are given for girder-type specimens.
Fig. 6 shows the theoretical results using the residual stress patterns 1 and 2 ($\sigma_{rc} = 0.3\sigma_y$ inset in Fig. 6) are compared with the test points for the as-weld and annealed A-type beams.[3)]

b) Ultimate Bending Strength

Since the inelastic lateral buckling resistance is furnished mostly by the compression flange, the lateral buckling strength becomes very close to that of a column whose effective cross section is composed of the compression flange and one-sixth of the web[6)]. In this analysis the ultimate bending strength of members with initial imperfections such as residual stresses and lateral deflection of the compression flange is determined by the beam-column concept using the numerical integration technique[5)]. The residual stress patterns used for this analysis is shown in Table 4 as patterns (C) and (D).

In Fig. 5 the ultimate bending strength curves obtained by the beam-column concept are given for the D-type and C-type beams with the initial lateral deflection $u_o = L_e/1000$ and with and without residual stress pattern (C). Ultimate bending strength curves for the residual stress patterns (C) with, $\sigma_{rc} = 0.3\sigma_y$ and initial lateral deflection $u_o = L_e/1000$ may explain the lower bound estimate against the plotted test results. In table 4 the numerical results by the beam-column approach are also given for the plate girders G-A$\sim$G-G.

Table 1 (a) Dimensions of test beams

Test Beams	Steel	d (mm)	b (mm)	w (mm)	t (mm)	L (mm)	End Moment Ratios	Remarks
A-1-0	SM50A	250	100	6	8	3 000	1.0	annealed
A-1-1		250	100	6	8	3 000	1.0	as-weld
A-1-2		250	100	6	8	3 000	0.5	as-weld
A-2-0		250	100	6	8	4 000	1.0	annealed
A-2-1		250	100	6	8	4 000	1.0	as-weld
A-2-2		250	100	6	8	4 000	0.5	as-weld
A-3-0		250	100	6	8	4 500	1.0	annealed
A-3-1		250	100	6	8	4 500	1.0	as-weld
A-3-2		250	100	6	8	4 500	0.5	as-weld
B-1-0	SM50A	250	120	6	8	3 000	1.0	annealed
B-1-1		250	120	6	8	3 000	1.0	as-weld
B-2-0		250	120	6	8	4 000	1.0	annealed
B-2-1		250	120	6	8	4 000	1.0	as-weld
B-3-0		250	120	6	8	4 500	1.0	annealed
B-3-1		250	120	6	8	4 500	1.0	as-weld
C-1-0	SM50A	300	100	6	8	3 000	1.0	annealed
C-1-1		300	100	6	8	3 000	1.0	as-weld
C-2-0		300	100	6	8	4 000	1.0	annealed
C-2-1		300	100	6	8	4 000	1.0	as-weld
C-3-0		300	100	6	8	4 500	1.0	annealed
C-3-1		300	100	6	8	4 500	1.0	as-weld
D-1-0	HT80	250	100	7	10	2 500	1.0	annealed
D-1-1		250	100	7	10	2 500	1.0	as-weld
D-1-2		250	100	7	10	2 500	0.5	as-weld
D-2-0		250	100	7	10	3 000	1.0	annealed
D-2-1		250	100	7	10	3 000	1.0	as-weld
D-2-2		250	100	7	10	3 000	0.5	as-weld
D-3-0		250	100	7	10	3 500	1.0	annealed
D-3-1		250	100	7	10	3 500	1.0	as-weld
D-3-2		250	100	7	10	3 500	0.5	as-weld
E-1-0	HT80	250	120	7	10	2 500	1.0	as-weld
E-2-0		250	120	7	10	3 000	1.0	as-weld
E-3-0		250	120	7	10	3 500	1.0	as-weld
F-1-0	HT80	300	100	7	10	2 500	1.0	as-weld
F-2-0		300	100	7	10	3 000	1.0	as-weld
F-3-0		300	100	7	10	3 500	1.0	as-weld

d = beam height, b = flange width, t = flange thickness, w = web thickness, L = beam length

Table 1 (b) Dimensions of test girders

Test Girders	Steel	d (mm)	b (mm)	w (mm)	t (mm)	L (mm)	End Moment Ratios
G-A	SM50A	1 000	130	6	10	4 100	1.0
G-B	SM50A	1 000	120	6	8	4 100	1.0
G-C	HT80	800	110	6	10	3 300	1.0
G-D	HT80	800	110	6	10	2 800	1.0
G-E	HT80	800	130	6	10	3 300	1.0
G-F	HT80	800	130	6	10	2 800	1.0
G-G	Flange HT80 Web SM50A	800	110	6	10	3 300	1.0

Table 2 Initial imperfections of flanges

Test Beams and Girders	Upper flange δ_u (mm)	Lower flange δ_l (mm)	δ_u/L	δ_l/L
A-1-0	0	0	0	0
A-1-1	1.5	0	1/1 960	0
A-3-1	4.5	0	1/1 000	0
B-1-0	3	8	1/ 990	1/ 370
B-2-0	0	2	0	1/1 985
B-3-1	0	6	0	1/ 745
C-1-0	1.5	1.5	1/1 970	1/1 970
C-1-1	4.8	6.8	1/ 620	1/ 435
D-2-0	0	0	0	0
D-2-2	0	0	0	0
D-3-1	2.5	0	1/1 400	0
D-3-2	1	0	1/3 470	0
E-2-0	1	0	1/2 960	0
E-3-0	2.5	0	1/1 390	0
G-A	1	1.5	1/4 100	1/2 720
G-B	1	5	1/4 100	1/ 820
G-C	1	2	1/3 300	1/1 650
G-D	3	3	1/ 940	1/ 940
G-E	0	0	0	0
G-F	2	0	1/1 400	0
G-G	0	1	0	1/3 300

Table 3 Summary of reference and experimental loads

Seecimens	M_y (t·m)	M_p (t·m)	M_{max} (t·m)	σ_{max} (kg/cm²)	M_{max}/M_y
A-1-0	8.18	9.45	7.69	3 218	0.94
A-1-1			7.00	2 929	0.86
A-1-2			9.70	4 069	1.19
A-2-0	8.18	9.45	7.94	3 322	0.97
A-2-1			6.61	2 766	0.81
A-2-2			8.42	3 523	1.03
A-3-0	8.18	9.45	6.33	2 649	0.77
A-3-1			5.75	2 406	0.70
A-3-2			8.54	3 573	1.04
B-1-0	9.47	10.78	9.40	3 406	0.99
B-1-1			8.00	2 899	0.84
B-2-0	8.47	10.78	8.60	3 116	0.91
B-2-1			8.76	3 174	0.93
B-3-0	9.47	10.78	9.28	3 362	0.98
B-3-1			7.10	2 572	0.75
C-1-0	10.41	12.16	7.58	2 493	0.73
C-1-1			9.20	3 026	0.88
C-2-0	10.41	12.16	8.04	2 645	0.77
C-2-1			6.88	2 263	0.66
C-3-0	10.41	12.16	7.48	2 461	0.72
C-3-1			6.89	2 266	0.66
D-1-0	22.53	26.08	21.16	7 373	0.94
D-1-1			20.89	7 279	0.93
D-1-2			27.90	9 721	1.24
D-2-0	22.53	26.08	20.68	7 206	0.92
D-2-1			18.54	6 460	0.32
D-2-2			23.10	8 049	1.03
D-3-0	22.53	26.08	19.21	6 693	0.85
D-3-1			16.24	5 659	0.72
D-3-2			21.84	7 610	0.97
E-1-0	26.14	29.84	26.46	7 946	1.01
E-2-0	26.14	29.84	24.42	7 333	0.93
E-3-0	26.14	29.84	22.79	6 844	0.87
F-1-0	28.68	33.50	26.32	7 191	0.92
F-2-0	28.68	33.50	22.42	6 126	0.78
F-3-0	28.68	33.50	20.78	5 678	0.73
G-A	71.68	106.32	43.6	1 968	0.61
G-B	72.31	93.74	41.0	2 160	0.57
G-C	113.90	128.81	64.4	4 438	0.57
G-D	113.90	128.81	83.4	5 748	0.73
G-E	126.15	140.23	94.8	5 899	0.75
G-F	126.15	140.23	105.1	6 540	0.83
G-G	113.90	128.81	65.5	4 514	0.58

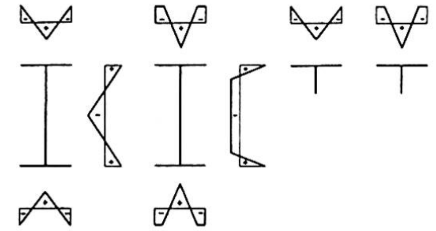
M_y = Calculated Yield Moment
M_p = Calculated Plastic Moment

Table 4 Comparison of theory and test for girders

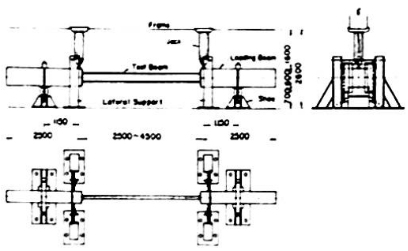
Test Girders	Experimental	Theoretical									
		Lateral Buckling Strength					Ultimate Bending Strength				
			Residual Stress Patterns				Residual Stress Patterns				
			(A)		(B)		(C)		(D)		
	$M_{max}^{(tm)}$	$M_E^{(tm)}$	$M_{cr}^{(tm)}$	M_{cr}/M_{max}	$M_{cr}^{(tm)}$	M_{cr}/M_{max}	$M_{u_1}^{(tm)}$	M_{u_1}/M_{max}	$M_{u_2}^{(tm)}$	M_{u_2}/M_{max}
G-A	43.6	87.55	56.63	1.30	49.82	1.14	48.75* 55.27**	1.12 1.27	44.38 52.29	1.02 1.20
G-B	41.0	55.52	51.32	1.25	46.99	1.15	41.09 47.30	1.00 1.15	38.17 44.46	0.93 1.08
G-C	64.4	69.53	68.75	1.07	68.25	1.06	53.27 58.39	0.83 0.91	51.20 57.37	0.80 0.89
G-D	83.4	96.06	83.60	1.00	75.87	0.91	54.29 78.96	0.65 0.95	52.45 74.99	0.63 0.90
G-E	94.8	114.16	93.83	0.990	81.39	0.86	— 90.54	— 0.96	— 85.43	— 0.90
G-F	105.1	157.99	99.31	0.95	82.40	0.78	76.79 99.24	0.73 0.94	74.93 95.66	0.71 0.91
G-G	65.5	67.60	67.10	1.02	66.62	1.02	— 48.65	— 0.74	— 47.72	— 0.73

* Upper line is for δ_u indicated in Table 2
** Lower line is for infinitesimal small u_o as $u_o = L_e/10{,}000$

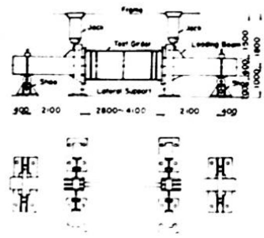
Assummed Residual Stress Patterns



	(A)	(B)	(C)	(D)
σ_{rc}/σ_y =	0.3	0.3	0.3	0.3
σ_{rt}/σ_y =	0.3	1.0	0.3	1.0

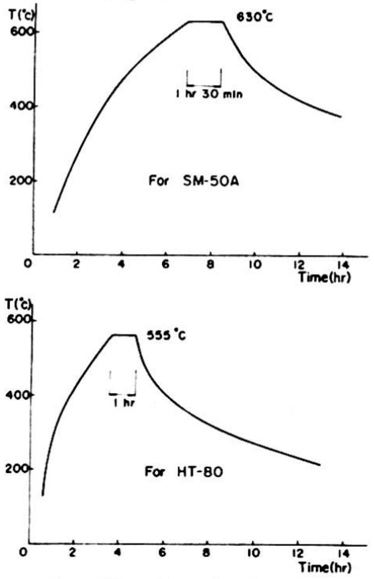


Test setup for beams



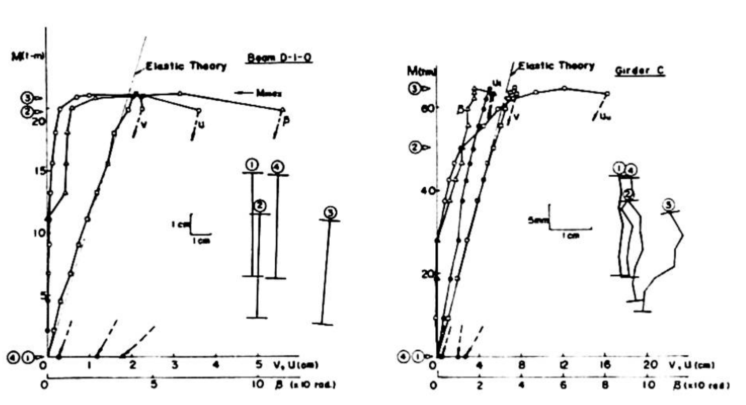
Test setup for girders

Fig. 1



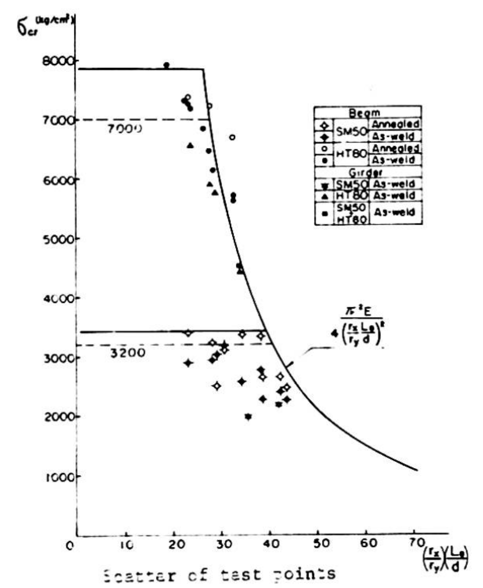
Annealing data for beams

Fig. 2



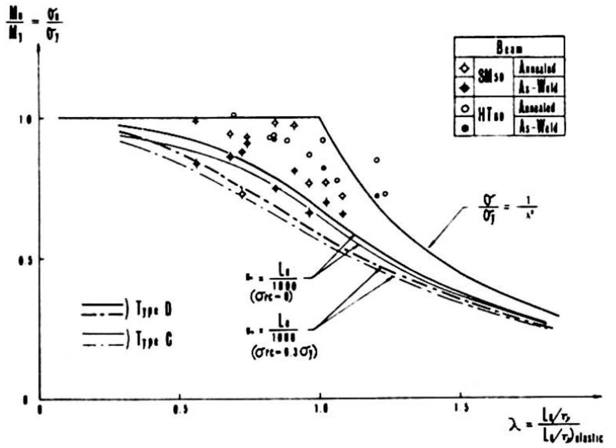
Load-deflection curves

Fig. 3



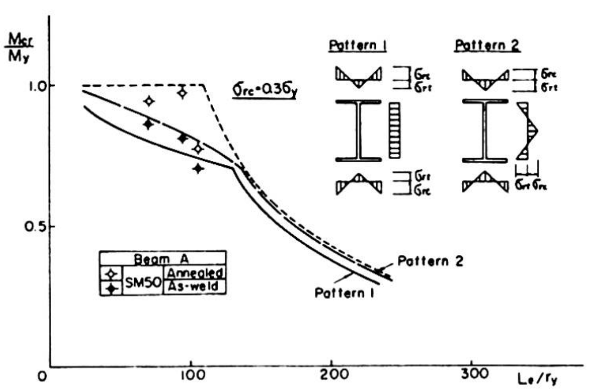
Scatter of test points

Fig. 4



Comparison of the results by beam-column approach and test for beams

Fig. 5



Comparison of the results by lateral buckling theory and test for beam A

Fig. 6

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SUMMARY

This report presents the results of an extensive experimental investigation into the behaviour and strength of flexural members with high strength steels which are failed by lateral buckling. A total thirty-six beam-type and seven girder-type members is tested under uniform moment and moment gradient. Test results of as-weld and annealed specimes are compared with the inelastic lateral buckling theory with the specified residual stress patterns and also with the beam-column approach to ultimate bending strength using the effective compressive section with initial imperfections.

RESUME

Ce rapport présente les résultats d'essais détaillés sur le comportement de la résistance d'éléments en acier à haute résistance soumis à la flexion, et qui ont été détruits par déversement. Les essais de spécimens soudés et recuits ont été comparés selon la théorie du déversement, en fonction des contraintes résiduelles, ainsi que selon l'approche "poutres-colonnes" pour le moment fléchissant de rupture, considérant la section effectivement comprimée avec des imperfections initiales.

ZUSAMMENFASSUNG

Dieser Bericht enthält die Resultate einer ausführlichen experimentellen Untersuchung über das Verhalten und die Tragfähigkeit von Biegeträgern aus hochfesten Stählen, die durch Kippen zerstört wurden. Eine Gesamtheit von 36 Trägertypen und 7 Balkenträgern wurden unter konstanten, linear verlaufenden Biegemomenten getestet. Testresultate von geschweissten und spannungsarm geglühten Proben wurden mit der unelastischen Kipptheorie und den bezeichnenden Eigenspannungsverteilungen und ebenso mit einer Näherungsuntersuchung als exzentrischem Knicken verglichen, indem die effektiv zusammendrückenden Abschnitte mit anfänglichen Imperfektionen verwendet werden.