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Cyclic behaviour of plate-to-beam welded connections

Bachir NASSER[1], Ahmed MAHMOUDI[2], Ali BOUTALEB[3], Salah Eddine BOUDRAA[4],
[1]*Laboratoire LM2SC. Département de Génie Civil USTO Oran-ALGERIA*
[2]*Laboratoire LM2SC. Département de Génie Civil USTO Oran-ALGERIA*
[3]*ENSET Département de Génie Civil Oran-ALGERIA*
e-mail: nasser54dz@yahoo.fr
[4]*ENSET Département de Génie Civil Oran-ALGERIA*

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Abstract

Modern codes contain application rules based essentially on monotonic tests that allow the design of welded connections between a profile with H section and a plate or a beam flange without transverse stiffeners in the H section. The validity of such designs has been tested in cyclic plastic conditions on 5 specimens. The results indicate that the design procedure in codes is on the safe side in cyclic conditions.

Key Words: Plate-beam, steel, strength, welds, cyclic.

Introduction

A contribution to the study of the behaviour of welded plate-beam steel connections under cyclic loading is presented. It deals with the problem of flange flexibility when the beam is an H section without transverse stiffeners. The test has been applied on 5 specimens. Four parameters were tested: the web slenderness, the steel yield stress, the (b_{eff}/b) ratio, and the weld quality;b is the width of the plate through which the external load is applied; b_{eff} defines the effective width of the flange of the H section). The test results were compared with the theoretical predictions provided by Eurocode3 (2004) which contains design rules for such connections. The degree of accuracy and the limitations of the design procedure based on static loading (Plumier, 1994; Ballio et al., 1990; C.T.I.C.M, 1981) were also examined.

The collapse mode, which occurred in both and H steel beams under the action of a double local load applied symmetrically to the flanges, presented buckles over the whole height of the beam web just before the collapse of the flanges. This collapse state is characteristics of symmetrical beam-column connections where the flanges of the beam are bent under the local tension/compression applied to the H or I section (Graham et al., 1960; A.I.S.C, 1969; Hang et al., 1973; Sherbourne, 1975; Jaspart, 1991; Aribert et al., 1988, 1990).

Experimental Program

The 4 parameters considered in the tested specimens were purposely selected to emphasize the connection of 2 collapse modes in terms of instability.

The first collapse mode corresponds to the ultimate capacity of the beam web to sustain a compressive load, taking into account the possibility for the load to spread through the thickness of the flange and the rounded connection zone that exist between flange and web of rolled sections; this mode defines the ultimate limit strength of the connection. A plastic deformation of the web is associated with this first mode.

The second collapse mode is consistent with elastic or elastoplastic instability of the web; it can occur before attaining the ultimate limit strength of the web computed without considering instability.

The first 4 test specimens labelled as C1, C2, C3, and C4 were made of mild steel beams (S235, nominal yield stressf_y= 235 MPa); and in the fifth test, labelled as C5, the grade of the steel beam was higher (S460, nominal yield stress f_y= 460 MPa).

Description of the tests

In each test, a double punching/pulling load was applied to steel beams by welded plate as shown in Figure 1.

The compression (alternatively tension) load applied through 2 rigid steel plates simulates the action of a beam flange in compression (alternatively tension) in an actual beam-column connection. The plates are fillet welded to the beam flanges. The dimension “a” of the weld throat is defined in compliance with Eurocode3 (2004).

Tensile tests have been carried to determine the mechanical properties of the beams, plates, and welds.

The measured geometrical characteristics together with the strength characteristics of the specimen tested are given in Table 1.

Table 2 presents the geometrical and mechanical properties of the welded plates used to introduce the compression/tension load and the mechanical properties of weld metal. These properties have been established by tension tests realised on specimens machined out of the weld seams. It should be noted that the use of “rutile” type of welds is easier.

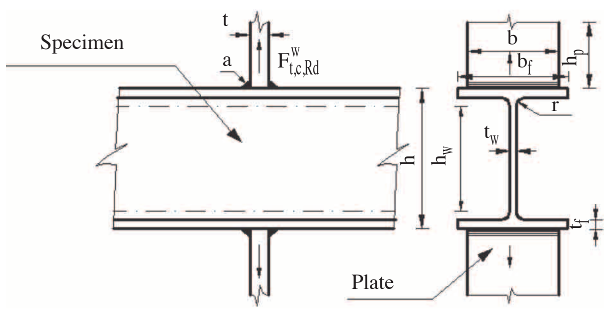

Figure 1. Geometrical characteristics of the steel beams used and details of the tested connection.

The characteristics of “rutile” and “basic” weld metals are respectively:

- ESAB OK 46-22 (ISO E 43 3 R22); R_e= 430 MPa; R_u= 510 MPa; A% = 28 ;
- SAFER NF 510 (ISO 2560 E51 5B 120 26 (H))
  R_e= 465 MPa,, R_u= 545 MPa, A% = 28.

Test set-up

Figure 2 shows sketches of the test set-up. A high precision differential hydraulic jack (SHENCK RD60 of 1000 kN) was used. Three hinges prevent the development of secondary moments.

Table 1. Measured geometrical and material properties of the tested specimens.

Test	Plate					Weld metal				
	Height h_p mm	Width b mm	Thickness t mm	f_y MPa	f_u MPa	type	a mm	R_e MPa	R_u MPa	A%
C1	130	159.9	9.9	301.0	431	Rutile	6	315	434	28.0
C2	130	159.8	9.9	292.8	429	Basic	6	409	461	28.2
C3	130	100.0	9.9	292.8	429	Basic	6	409	461	28.2
C4	130	159.8	9.9	292.8	429	Basic	6	409	461	28.2
C5	130	165.0	9.9	402.0	588	Basic	8	409	461	28.2

Table 2. Measured geometrical and material properties of the plate and the weld metal.

Test	beam	h mm	b_f mm	t_w mm	t_f mm	r mm	f_{yf} MPa	f_{yw} MPa	f_{uf} MPa	f_{uw} MPa
C1	HEA160	153.3	160.3	6.6	9.4	14.9	274.5	305.2	430.0	451.0
C2	HEA160	153.8	160.5	6.6	8.8	14.9	303.8	340.0	434.0	454.0
C3	HEA160	154.2	160.2	6.5	9.0	14.9	305.8	340.0	434.0	454.0
C4	HEA160	153.8	161.6	6.3	8.8	14.3	360.0	400.0	483.0	534.0
C5	IPEA360	357.5	166.1	6.6	11.5	15.6	463.0	568.0	524.0	635.0

To compensate for a possible misalignment of the plates, which is likely to involve an overall rotation of the beam when the compression effort is increased, two height adjustable support systems were placed at each end of the beam (Figure 2). A force transducer was placed at each support.

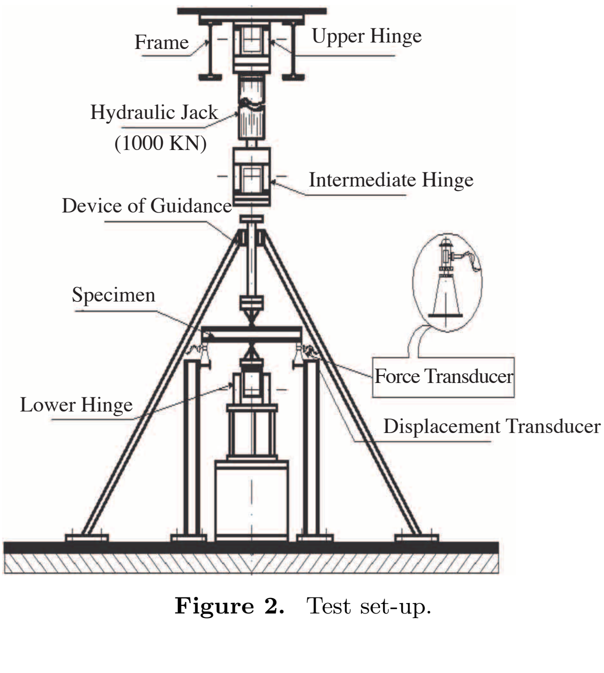

Figure 2. Test set-up.

The data acquisition and the recording of measurements were carried out using a Hottinger Baldwin Messtechink system, UPH3200 type.

Figure 3 shows the measured displacements in a schematic way. The vertical and relative displacements between the plates captured by transducers (1) and (2) detect the overall information on the behaviour of the connection in the region of the applied load. The transverse displacements of the beam web were captured by transducers located at right angles to the web plane.

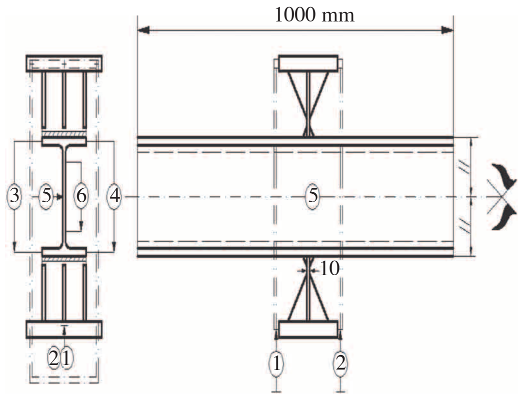

Figure 3. Displacement measurements.

Loading procedure

In each test, the load was applied with 20 kN increments until collapse. All specimens were tested under symmetrical alternate controlled loading from F_i^- (compressive loading) to F_i^+ (tensile loading) with $|\mathrm{F}_i^-| = |\mathrm{F}_i^+|$,$F_i$ being the maximum force reached at each loading stage. Several cycles were carried out at each level F_i.

Presentation and analysis of the results

The tests results are presented in terms of collapse mode, resistance, and effects of the weld type and displacements. The results were compared with design values provided in Eurocode3 (2004) and with results of the Graham model (1960).

Modes of collapse

After each test, buckles appeared over the whole height of the web; a collapse by failure of the welds then followed, except for specimen C1.

Figures 4 and 5 illustrate respectively the failure aspect of the weld after the test on specimen C1 and the collapse mode by local compression with a well identified buckling aspect after the test on specimen C5.

Strengths

Table 3 shows the measured values of the collapse loads and the design resistance according to Eurocode3 (2004) and to the model by Graham et al (1960). In detail, the comparison includes the plastic strength in tension of beam flange $F^{f}_{t,Rd}$, the plastic tensile strength of the plate (without stiffeners) $F^{eff}_{t,Rd}$, the plastic tension and compression strengths of the beam web $F^{W}_{t,c,Rd}$, the strength of the beam web considering the effect of local buckling $F_{b,Rd}$ and the experimental collapse load $F_u^{\exp}$. All the values were computed on the basis of the real material characteristics.

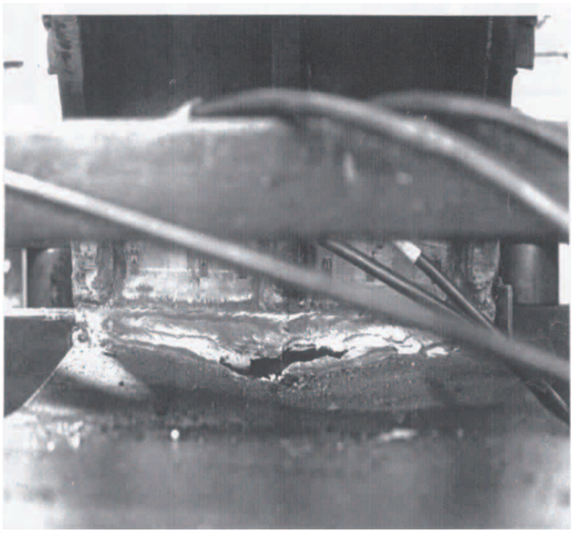

Figure 4. Failure of the weld bead (test C1).

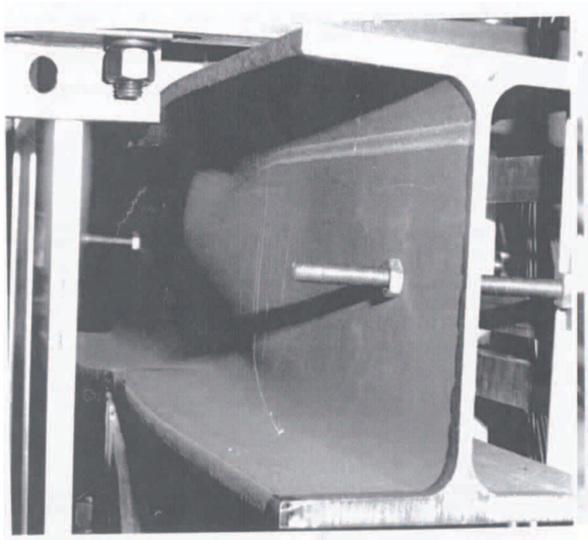

Figure 5. Buckling with local compression (test C5).

The test results indicate that, if welds of appropriate quality are applied, like in C2, C3, and C4 tests, designs taking into account the recommended values of the effective width b_{eff} are on the safe side.

The results in Table 3 also shows that the amplitudes of the experimental collapse loads $F_u^{\exp}$ are well beyond the computed values $F^{eff}_{t,Rd}$, even under demanding cyclic conditions.

Observations of collapse modes are as follows. In specimens C2, C3, and C4, which are made of S235 steel, the failure is due to a local compression followed by the crushing of the beam web. The values obtained by the model of Graham (1960) are more realistic and accurate than those predicted by the model of Eurocode3 (2004), because the latter includes a contribution of the weld beads that appears rather virtual. The $F_u^{\exp}/F^{w}_{t,c,Rd}$ ratios takes the values 1.21, 1.11, 1.18, and 1.20, respectively.

Specimen C5, which is made of S460 steel, fails by local buckling of the web, with a

$F_u^{\exp}/F_{b,Rd}$ ratio equal to 1.66.

Finally, failure takes place in local compression, under alternate cyclic loads; The usual formulas of static resistance are still on the safe side for situations of accidental nature (partial energy dissipation in a connection under seismic loading or inappropriate response of a connection under quasi-static loading at the serviceability limit state).

The average of $F_u^{\exp}/F_u^{th}$ ratios is equal to 1.32 (F_u^{th} being the minimum of $F^{w}_{b,Rd}$,$F^{w}_{t,c,Rd}$). This should be sufficient to estimate that the computed values are on the safe side. This is confirmed by the test results obtained by Nasser (2007).

Effects of the weld quality

Comparing the results of tests on specimens C1 and C2, in which only the quality of the welds is varied, one can observe that the type of welds has an impact on the resistance in alternate cyclic loading. Specimen C1 was made with normal quality "rutile" weld; specimen C2 was made with "basic" welds. Specimen C1 failed in the weld bead for a tensile load of 290 kN. Specimen C2 failed in compression upon the application of 350 kN load. Based on this observation, the remaining tests were limited to specimens welded with the "basic" quality.

Table 3. Computed resistance and experimental failure loads.

Test	$F^{f}_{t,Rd}$	$F^{eff}_{t,Rd}$	$F^{w}_{t,c,Rd}$	$F^{w}_{t,c,Rd}$	$F_{b,Rd}$	$F^{\exp}_{u}$
	E.C.3[a]	E.C.3[a]	E.C.3[a]	Eq[b]	E.C.3[a]	
	kN	kN	kN	kN	kN	kN
C1	278.5	304.5	298.9	264.7	263.2	290
C2	271.3	284.1	326.2	288.1	285.1	350
C3	278.6	287.9	323.5	286.0	281.0	340
C4	296.3	279.8	358.8	316.0	302.8	380
C5	579.0	470.1	629.9	545.0	270.8	450

[a] Model of Eurocode3 (2004)
[b] Model of Graham et al. (1960)

Displacements

Load-displacement curves F-δ were recorded in all tests. The diagrams of specimen C3 are presented in Figures 6 to 8. In Figure 6, the displacement represents the measured values of the whole unit plate-beam. In Figure 7, the displacement indicates the measured values at the plate level, between the edges of the beam flanges. Figure 8 shows the evolution under increasing load of the web transverse displacement measured in the centre of the web, in terms of connection.

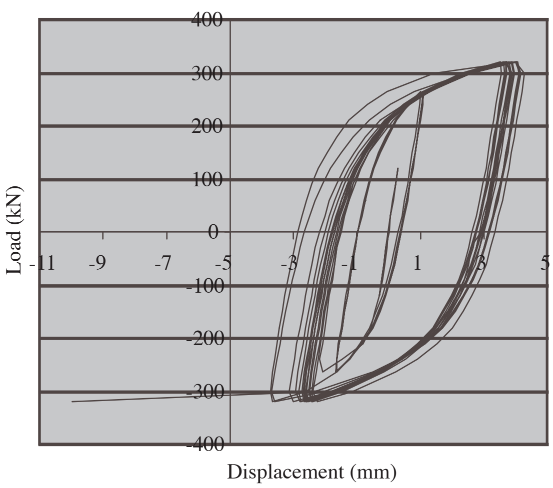

Figure 6. Load-displacement relationship for the whole unit plate-beam of the specimen C3.

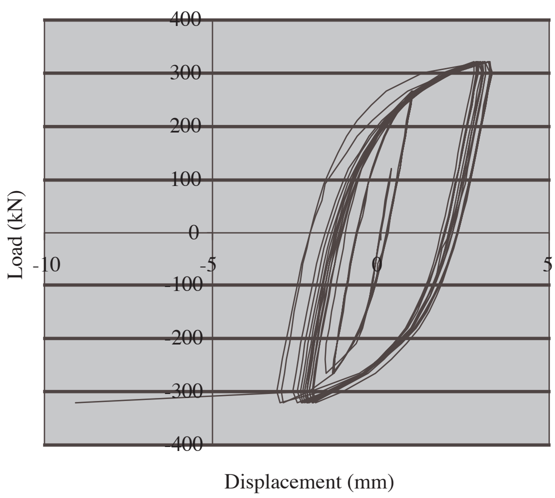

Figure 7. Load-displacement relationship measured at the flange ends of the specimen C3.

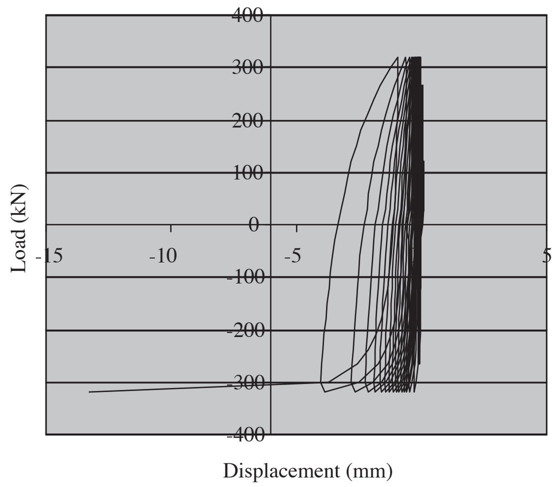

Figure 8. Transverse load-displacement relationship measured in the centre of the beam of the specimen C3.

From the typical curves obtained by the test results carried out in this survey, one can observe that:

- For an amplitude of the alternate cyclic loading equal to the computed strength intensity, for which failure occurred ($F^f_{t,Rd}$ for the test on specimen Cl, $F^w_{c,Rd}$ for the test on specimen C2, C3, and C4, and $F_{b,Rd}$ for the test on specimen C5), see Table 3, very stable cycles can be observed: they progressively overlap with no hardening or softening aspects occurring.

- Apart from specimen C1 for which a sudden failure occurred in the weld bead due to a reduced strength, the remaining specimens sustained the whole external applied loads. However, during the tests, a cyclic phenomenon was observed only for cycles with amplitudes very close to the collapse load.

- As far as the cyclic behaviour was concerned, one can notice that the slopes measured at the beginning of the entire cycles remain practically equal to the slope that corresponds to the initial monotonous curve in the elastic regime. Generally, one can say that the repeated cycles in the same range of loads (F_i^+, F_i^-) are almost identical. Indeed, no fall of strength up to the cycle preceding the failure was observed. Furthermore, the total ductility is relatively small since it is only of the order of 2 for the cycles up to the ultimate plastic failure load of the web. The ductility is approximately 5 for the last cycle before failure. The stiffness decrease does not exceed 20% before failure.

Conclusion

The work reported in this paper was carried out to study the various phenomena relative to the problem of local compression for the welded plate-beam connections without stiffeners, under alternate cyclic loading. Experimental results obtained via 5 tests have been analysed and lead to the following conclusions:

- The weld quality used is of relevant significance for the connection tensile capacity. This parameter should be considered in the design procedures of the actual codes in order to avoid early failure in the welds;

- Static based design procedures provided by codes to evaluate the web local strength in compression are likely to be on the safe side even in the case of demanding cyclic loading;

- For relatively thinner beams, where a collapse mechanism by warping due to local compression prevails, a more obvious resistance drop can be observed; in that case, the use of a static based design procedures is not likely to give enough safety with respect to warping;
- Although the failure under tensile load was purposely isolated using an appropriate weld quality (specimen C1), the strengths predicted according to the design procedures contained in the actual codes are generally lower than the measured ones. This confirms that the static based design formulae used in such procedures remain safe in the case of alternate cyclic loadings;
- The cyclic loading was applied in different ways; in all cases, the beam sustained the entire alternate cyclic load up to a loading stage close to the ultimate limit state.

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