The Dogbone: Back to the Future

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The weakening of specific sections of a structure in order to change them into reliable energy dissipative zones in case of an earthquake is an idea which was developed in principle in the 1980s by the author of the present paper. During a research project sponsored by the Luxembourg based steel producer ARBED and the European Union, the idea was tested and validated as concerns its contribution to ductility. Because weakening a structure, even locally, might have negative influences, other analyses were also performed during the same research and later on by David (1990).

At that time, ARBED filed a patent on the reduced beam section (RBS) or "dogbone" concept, but, after the Northridge earthquake, waived any licensing fees and claims. This certainly encourages the development of this technique. Research efforts are now being made in many places in the United States to define the best applicability conditions for dogbones, in particular their geometry. In a recent paper in this Journal, Iwankiw and Carter (1996) presented test results based on this idea, whose time has come. In order to further its applications, existing European data, some of which were published by the Commission of the European Community (1992) in the report on the research mentioned above, some published in conference proceedings by Plumier (1990), some unpublished, are presented here.

The general purpose of the ARBED-EU research (1992) was to define an economical composite steel concrete constructing system, which would at the same time assure the fire and earthquake resistance of the structure. The fire resistance aspect had been covered in previous research and the work carried out was essentially concerned with the cyclic behaviour of beam-column connections.

There were 3 series of tests:

- test series 1 consisted of 20 tests on various types of connections of pure steel and composite 1 beam-1 column specimens (facade columns).
- test series 2 consisted of 20 tests of pure steel and composite 2 beams-1 column specimens (interior columns)
- test series 3 consisted of 10 tests on complete composite plane frames (three columns, two bays).

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Tests series 1 and 2 were performed at Politecnico di Milano (Italy). Series 3 tests were performed at the T.H. Darmstadt (Germany) and University of Liege (Belgium).

At the time, reduced beam sections were thought to be interesting for several reasons:

- the overstrength design rule, applied to the connection of the beam to the column, brings welds of exaggerated size when fillet welds are used
- the conclusion is the same for bolted connections, for the connecting elements (end plate, angles), as well as for the number and size of bolts
- in case of a wrong steel delivery on site (yield strength of steel above upper limits defined in Eurocode 8), reduced beam sections offered an easy alternative to the rejection of the delivery
- reduced beam sections also appeared as an economical and easy solution in which the inertia of the beam is unchanged where necessary for deflection requirements (midspan). Reduction is carried out to create a controlled and safe dissipative zone in a very localized zone, avoiding the development of the plastic bending moment in the connection itself.

GEOMETRY AND SURFACE ASPECT OF THE REDUCED SECTION

The specimens tested in the ARBED-EU research were all made using HE300B sections for the columns (equivalent to W12×12×79) and HE260A sections for the beams (equivalent to W10×10×49). The connections considered in specimens with reduced beam sections are those of Figure 1.

The geometry of the reduced sections tested in series 1 and 2 of the research are defined in Figure 2. The reasons for these choices are explained in another paragraph. The reduction of flange width is accomplished by oxygen cutting, followed by a disc machining of the surface, in order to avoid stress concentrations and early cracks. All the reduced sections of series 1 and 2 behaved well and displayed a high ductility. See Table 1. Results concerning J1, K1, J5 and K3 specimens have not been published other than in the research report. In series 3, another geometry of the reduced section was used in order to solve a problem of overstrength of the steel delivered to the Liege Laboratory: 404 Mpa (58,6 ksi), instead of an estimated 300 Mpa (43,5 ksi) for a 235 Mpa (34 ksi) characteristic yield strength. Because concrete had already been infilled between the flanges, the oxygen cuts could not be cleaned by disc

Table 1.										
Specimen Type— Symbol	Steel or Composite	M_y kNm	(*) (kip-ft)	θ_y rad	$M_{0.025}$ kNm	(**) (kip-ft)	M_u kNm	(***) (kip-ft)	θ_u rad	Number of inelastic cycles
FS - E1	St	250	(184)	0.009	295	(217)	395	(291)	0.083	18
RBS - F1	St	230	(169)	0.009	280	(206)	285	(210)	0.098	24
RBS/FS		0,92			0,95		0,72			
FS - E3	Comp. 1	380	(280)	0.011	430	(317)	400	(295)	0.085	21
RBS - F2	Comp. 1	325	(239)	0.010	380	(280)	335	(247)	0.086	24
RBS/FS		0,86			0,88		0,83			
FS - J1	St	360	(265)	0.009	420	(309)	590	(435)	0.100	18
RBS - K1	St	320	(236)	0.012	460	(337)	525	(387)	0.100	15
RBS/FS		0,89			1,09		0,89			
FS - J5	Comp. 2	660	(486)	0.011	740	(545)	840	(619)	0.045	6
RBS - K3	Comp. 2	560	(413)	0.010	660	(486)	830	(612)	0.098	12
RBS/FS		0,85			0,89		0,99			
LG	Comp. 2	560	(413)	0.025	585	(431)	630	(464)	0.047	5

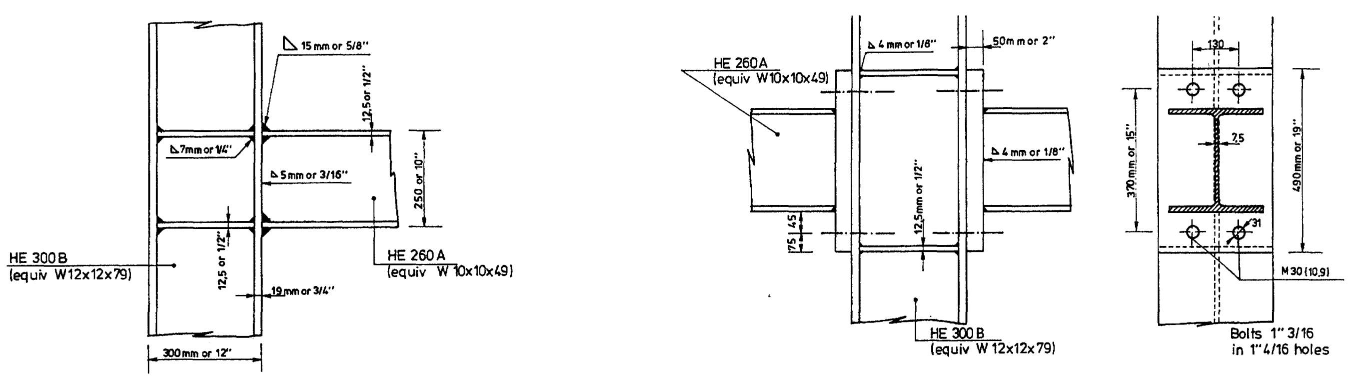
* M_y is the experimental value of the plastic moment *Pl.*
** $M_{0.025}$ is the experimental value of the bending moment corresponding to a 0.025 radian rotation in the beam, between sections A and B.
*** M_u is the bending moment corresponding to the maximum rotation θ_u between section A and B reached during the test and allowing three complete cycles without failure or a loss in resistance over 30 percent of the resistance of the first cycle of same displacement.

machining. A relatively early failure was observed in that case: the ductility was still high (4), but the number of inelastic cycles was lower (5).

These results may be explained by the surface aspect in the reduced section and also by a more severe geometrical shape.

DEFINITION OF THE TEST PROGRAM

The testing procedure used during the tests is the one developed in 1986 by the European Convention for Constructional Steelwork (ECCS 1986). The ATC-24 procedure (1995) is very similar to the ECCS procedure. The loading history can be summarized as follows:



Specimens E1,F1,E3,F2 (fillet welds)

Specimens J1,J5,K1,K3 (butt welds)

Fig. 1. Details of connections.

- during the first loading cycles, a yield displacement e_y is defined
- then 3 cycles are performed between equal displacements $+2e_y$ and $-2e_y$
- then 3 cycles are performed from $+4e_y$ to $-4e_y$
- and so on at $\pm 6e_y$, $+8e_y$, ...
- the test is stopped when a loss in resistance of over 30 percent between two cycles at the same displacement is observed.

The research program involved tests on similar specimens, some with RBS and others without RBS, that is, full section FS.

Some specimens were only steel, others had concrete encased between the flanges, others were composite steel sections with a slab. The various types of specimens are mentioned in Table 1 by the symbols St, Comp. 1 and Comp. 2 respectively.

The test specimens are described with these symbols in Table 1.

All these results are expressed in terms of bending moments applied to the beam in a situation involving a gradient of the bending moment in the beam—Figure 3.

All the characteristics bending moments discussed hereafter are computed as $M = F.l$ where l is the distance between the load axis and the column axis. See Figure 3.

The steel of the tested sections was a Fe 360 steel (characteristic yield stress 235 Mpa or 34 ksi).

For the tested specimens, the real steel mechanical properties were, on the mean:

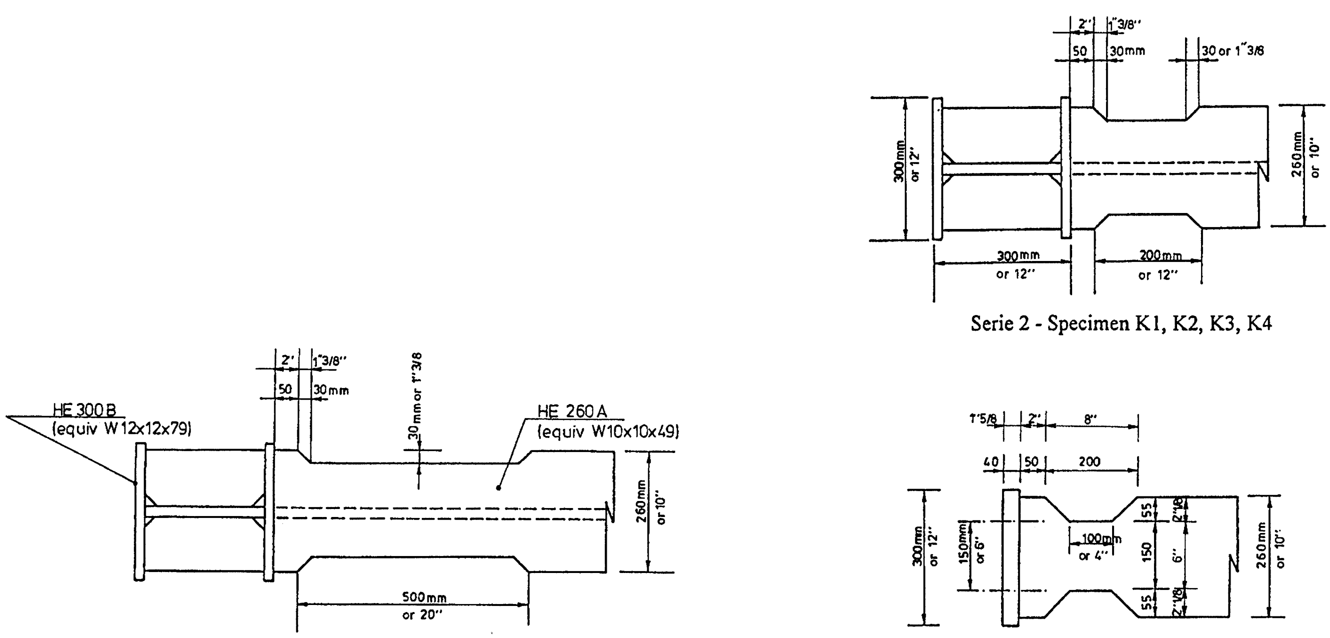
- yield stress: 300 Mpa (43,5 ksi)
- ultimate stress : 420 Mpa (60,9 ksi)
- elongation at failure measured on a 100 mm. (4 in.) basis: 30 percent

RESULTS OBTAINED

In Table 1, comparing the results of all full section FS to each associated reduced beam section RBS, it can be seen that a 23 percent reduction of the width of the flanges (0,23 = 60/260) brings a reduction of M_y and $M_{0,025}$ of about 10 to 15 percent only. A real plastic hinge is developed, in spite of the polygonal aspect of the reduction of section. Ductility in rotation θ_u/θ_y is greater than 8.3. Ductility expressed in terms of rotations is always greater than 3,2 times the 0,025 radian rotation which is sometimes considered as the upper limit of θ required in practice. The total number of inelastic cycles is always greater with design involving RBS than without.

Only specimen LG, for reasons explained above, gave a relatively low number of inelastic cycles.

The results of specimen K3 which is a composite section with a slab show that the dogbone idea works also for com-



Serie 2 - Specimen K1, K2, K3, K4

Serie 1 - Specimen F1 et F 2

Serie 3 - Specimen LG tested in LIEGE.

Fig. 2. Geometry of the tested specimens.

posite sections, in spite of the greater elongation involved in the bottom flange in that case; the ultimate rotation θ_u and the number of inelastic cycles are similar to those obtained for specimen K1, which is similar to K3 for the steel sections involved, but without any composite character.

Diagrams were recorded during the tests of the specimens in order to set forward the various contributions to flexibility.

Two diagrams recorded during the test on specimen K3 are presented at Figure 4. The upper diagram presents the rotation resulting from the total deformation of the connection zone. This deformation results from a series of terms: the shear deformation of the panel zone of the column, the deformation of the connection itself (end plates, bolts) and the deformation of the beam in bending and shear. The lower diagram gives the rotation resulting from the shear deformation of the panel zone. From these diagrams and the observations during the test of specimen K3, it has been concluded that the dogbones realized in the beams had been able to concentrate the plastic deformation in the beam and to provide a great ductility. A similar specimen, but without dogbones (see J5 in Table 1 and 2) had been failing in the bolts after a far less ductile life.

Looking back after 5 years on all the research work done (Schleich, 1992) and having in mind the effects of Northridge January 1995 earthquake, it appears that, besides of the dogbone influence, another essential result has to be stressed: no early or brittle failure was observed inside or in the vicinity of welds of the tested specimens—See Table 2. All failures took place after large local plastic strains in the steel sections

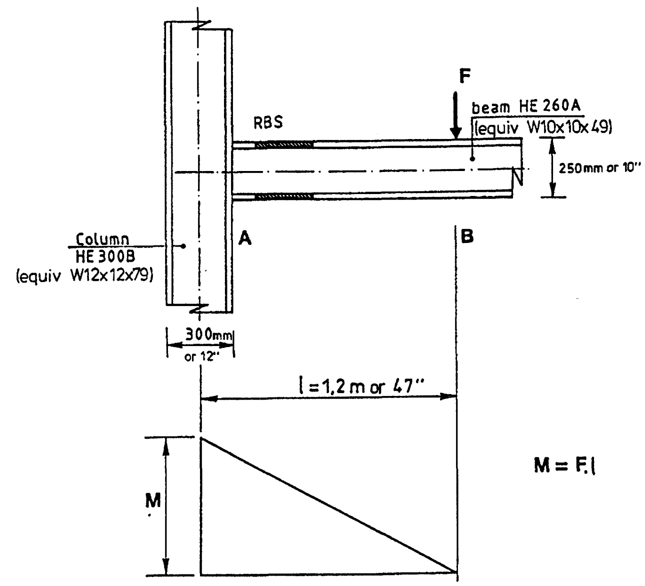


Fig. 3. Definition of M.

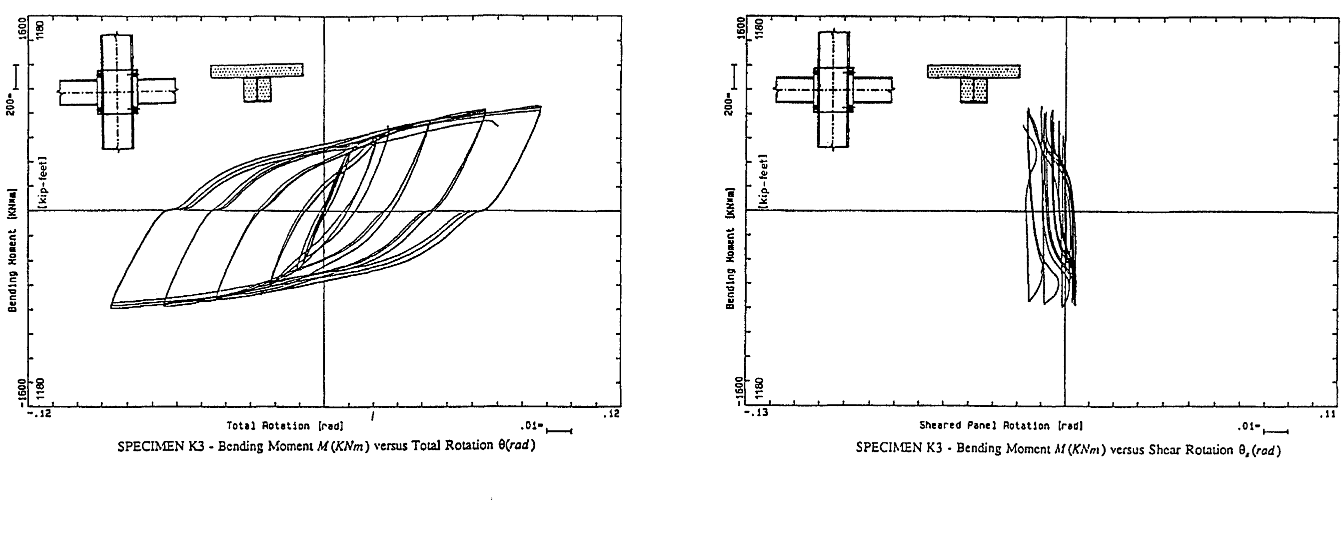


Fig. 4. Effectiveness of dogbone in a composite beam.

Table 2.			
Specimen Type— Symbol	$\frac{\theta_u}{\theta_y}$	Number of inelastic cycles	Failure
FS-E1	9,2	18	crack in beam near weld
RBS-F1	10,8	24	buckling in dogbone zone
FS-E3	7,7	21	crack in beam near weld
RBS-F2	8,6	24	buckling in dogbone zone
FS-J1	11,1	18	no failure—shear deformation of column panel zone exceeding capacity of test set up
RBS-K1	8,3	15	no failure—bending deformation of beam and shear deformation of column panel zone exceeding capacity of test set up
FS-J5	4,1	6	bolt failure
RBS-K3	9,8	12	bolt failure

had been realized. This is valid for fillet welds as well as for butt welds and is the result of the following facts:

- the parent material belonged to weldability classe C (absorbed energy 35 J/cm^2 at O°C, that is 25,8 lbf-ft at 32°F)
- the size of fillet welds was important
- the butt welds connecting a flange to an end plate were made from both side of the flange
- the material involved was not prone to lamellar tearing
- the axial stresses in columns were as low as 14 Mpa (2 ksi).

INFLUENCE OF THE REDUCTION IN FLANGE WIDTH ON THE FIRE RESISTANCE OF BEAMS

At the time of the ARBED-EU research, it was felt that the RBS idea was unusual enough to require further thinking on what might be the negative influences of the technique. Given the fire resistance context of the research, a concern was the potential reduction in load carrying capacity of structures involving RBS when fire conditions were realized.

This problem was explicitly studied by means of numerical modelling.

The conclusion is that the decrease in resistance at high temperatures is smaller than the decrease in mechanical resistance at normal temperatures, which is negligible (see next paragraph), so that this aspect of the design does not impair the worth of reduced beam sections.

INFLUENCE OF REDUCED SECTIONS ON THE DESIGN OF THE STRUCTURE

The existence of RBS in a structure increases its flexibility. This increase is relatively low since only a short length of the beams are involved, but it was feared that, in the end, bigger sections would be necessary and that the total weight of steel would be increased.

A study on the impact of the use of reduced beam sections on the design of a structure was carried out as the final paper of a student in Civil Engineering at the University of Liege (David, 1990). It consisted of the complete static and seismic design of two 5-story frame structures, with 3 bays in each direction. One frame was classical, the other frame included RBS with a 20 percent reduction in the width of the flanges. This study concluded that:

- The use of reduced beam section does not make the use of bigger sections of beams necessary;
- the cost of the reduction of the beam width ("shaving" of

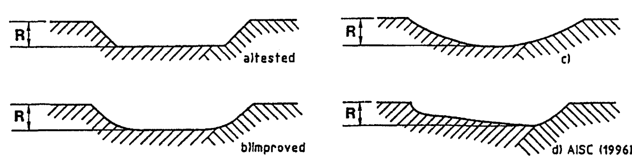


Fig. 5. Different possible shapes of reductions in beam sections.

section) is balanced by the reduction of the cost of the connections.

GEOMETRY OF THE REDUCTION OF THE BEAM SECTION

The shape of the reduction in beam section and the technique used to realize it should be defined considering the following facts:

- a plastic hinge has a length which is of the order of the beam height
- the technique has to deal with two types of stress concentration: local ones, like the scratches made by flame cutting, and global ones, resulting from the global shape of the reduced section
- local stress concentrations are of a higher order of magnitude when compared to global stress concentrations. Referring to one classic book in this field (Peterson, 1953), it can be found that the stress concentration factor K_t can be as high as 10 when the radius of the local notch is 1 percent of the width of the bar in tension, while k_t is only 3 for the shape of Figure 2, if the radius of the circle connecting the two straight cuts is 3 percent of the width of the flange; this means a 0,25 inch radius, which is easy to realize.

These facts induced the definition of the polygonal cuts sketched at Figure 2. Tests have shown that such RBS are a satisfactory solution, provided that a good surface aspect of the cuts is realized after oxygen cutting.

Since all the tests gave satisfactory results, no parametrical study has been made at the time in view of optimizing the shape of these reductions in section. Smoother connecting lines between the straight lines, as sketched in Figure 5b, would reduce the risk of a wrong carrying out of the intersection zone of straight cuts and are thus advisable for a constant quality of RBS.

An even smoother solution can be a fully curved cut. See Figure 5c. It has been tested in the United States. However, the reduction in plastic resistance is then not constant over the length of the reduced beam section. Then, on the mean, it brings less reduction of the plastic resistance of the reduced zone and subsequently a higher demand on the neighbouring zones than in design 5a or 5b. In fact, design 5c seems essentially influenced by a worry about the global shape of the RBS, which we have explained to be a secondary problem, at the expense of the efficiency of RBS.

More recently, a design in which the plastic resistance is adjusted to the shape of the bending moment diagram has been tested by AISC (Iwankiw, Carter, 1996). Its advantage is in principle a simultaneous development of yielding over the whole length of the RBS. However, this is only true when the bending moment diagram is well defined. If static loading of variable amplitude is possible, then the bending moment diagram can have various aspects, and an RBS shape with variable width may be no better adapted than an RBS shape with a constant width.

CONCLUSIONS

The experiments and analyses made in Europe in the period 1989–1990 on the reduced beam section technique have set forward the following facts:

- reduced sections of the type sketched in the two upper drawings of Figure 2 provide safe and ductile dissipative zones at the end of the beams.
- the presence of these "fuse" zones brings smaller demands on neighbouring zones; this has particularly been observed in some tests where the plastic (bending) mechanism took place in the reduced section of the beams when a RBS was used, while a plastic (shear) mechanism took place in the column when a full section of the beam was tested.
- given the shape of the diagram of bending moments in the beams of a frame structure submitted to seismic action, the reduction in the available plastic moment of the beam is less than the reduction in geometrical dimensions.
- parametrical studies have shown that the local reduction in flange width does not significantly reduce the stiffness of the beam, so that no change in cross section is required to comply with deflection requirements for beams under service (static) loads or to achieve fire resistance.
- RBS of the type given in the two upper drawings of Figure 2 gave satisfactory results; other shapes of reduction of section deserve further studies; however, for reasons explained here, they should not bring tremendous improvements on the basic design sketched at Figure 2.
- satisfactory results have been obtained, as well on steel sections as on composite sections (steel beam + slab).

Obviously, the quality of the results obtained is related to the elongation properties of the beam material.

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