Experimental analysis of cold-formed steel beam-to-column joints

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ABSTRACT: Aseismic design requires selection of energy dissipation mechanisms which combine stable cyclic performance with possibility of actually controlling the key behavioural parameters. When beam-to-column joints are comprised in the mechanism, it has been shown that benefits may be achieved in terms of increasing the cost/effectiveness ratio for the framework. Recent tests on semi-rigid joints indicated that use of joint action in seismic zones is feasible, and possibly advantageous, also in frames with cold formed members. The paper intends to illustrate the main features of these tests, to report the results of a first evaluation of the experimental data, and to link joint cyclic responses and aseismic design requirements for semi-rigid frames.

1 INTRODUCTION

A key parameter to measure the seismic efficiency of steel frames is the ability to dissipate energy by undergoing large plastic deformations without formation of "brittle" mechanisms. This capability of the whole system basically depends on the stability of the hysteretic behaviour of individual components, such as members and joints (Castiglioni and Di Palma 1989, Popov et al. 1986); is then function not only of the ductility of the material but also of the importance of overall and local buckling as well as of the low cycle fatigue phenomenon (Ballio and Calado 1986). Satisfactory aseismic design requires that all behavioural factors are considered and "utilized" in order to achieve an "optimal" costeffectiveness ratio.

The above mentioned design criteria, which characterise the so-called ductility approach to earthquake engineering, hamper the use of steel frameworks with cold-formed members, which by their own nature are designed optimally when reference is made to, and allowance is taken for, local buckling.

This certainly applies if the traditional models of "simple" or "rigid" frames are adopted in design. In the first case, beam-tocolumn joints, designed to resist only shear actions, are realized with simple details and the energy dissipation capability is concentrated in the bracing cantilever systems, which in most instances do not possess a satisfactory seismic response. On the other hand, the "rigid" frame ductility is substantially limited by the buckling behaviour of cold formed members (sections of which are generally semi-compact or even slender). A possible philosophy to improve the cost-effectiveness of cold formed steel structures in seismic areas is represented by the use of systems where joints and bracing cantilevers are associated as the key dissipation elements. Suitable design criteria should enable to balance the contribution of the two systems by taking into account their strength, stiffness, ductility and hysteretic behaviour. The lack of experimental data as well as of simple prediction methods enabling approximation of joint response prevents nowadays this criterion from being used in current practice. Reliable joint models require to be based on an extensive experimental background. Therefore the first phase of a research study presently in progress at the Universities of Trento and Ancona, aimed at checking the feasibility of using semi-rigid cold formed frames in seismic areas, was devoted to the experimental analysis of joints. Seven different types of beam-to-column cold-formed steel joints were subject to cyclic loading. Evaluation of the experimental data has been carried out in order to permit determination of the key response characteristics. Besides, numerical dynamic analyses were conducted aimed at a first appraisal of joint action and of joint behavioural requirements in aseismic cold formed semi-rigid frames. Both the experimental and numerical results seem to confirm the suitability of these systems for use in earthquake prone regions.

2 THE EXPERIMENTAL ANALYSIS

In order to have an insight in the cyclic behaviour of cold-formed semi-rigid joints of practical interest, the four specimens of four-ways internal joints shown in figure 1 were tested (Bernuzzi & De Martino 1991). The beams are, in all specimens, pairs of cold-formed channel sections, though different as to sensitivity to local buckling: i.e., in specimens PF1/A, PF2 and PF3 they are slender (class 4), according to Eurocode 3 (1991) classification, in specimen PF1/B they are compact (class 2). Columns are also made of cold-formed shapes in specimens PF1/A and PF1/B (where sections are slender (class 4) and plastic (class 1) respectively), while hot-rolled profiles are used in the two other specimens (PF2 and PF3). Bolted connections were selected on the basis of fabrication and erection considerations; they employ angles,
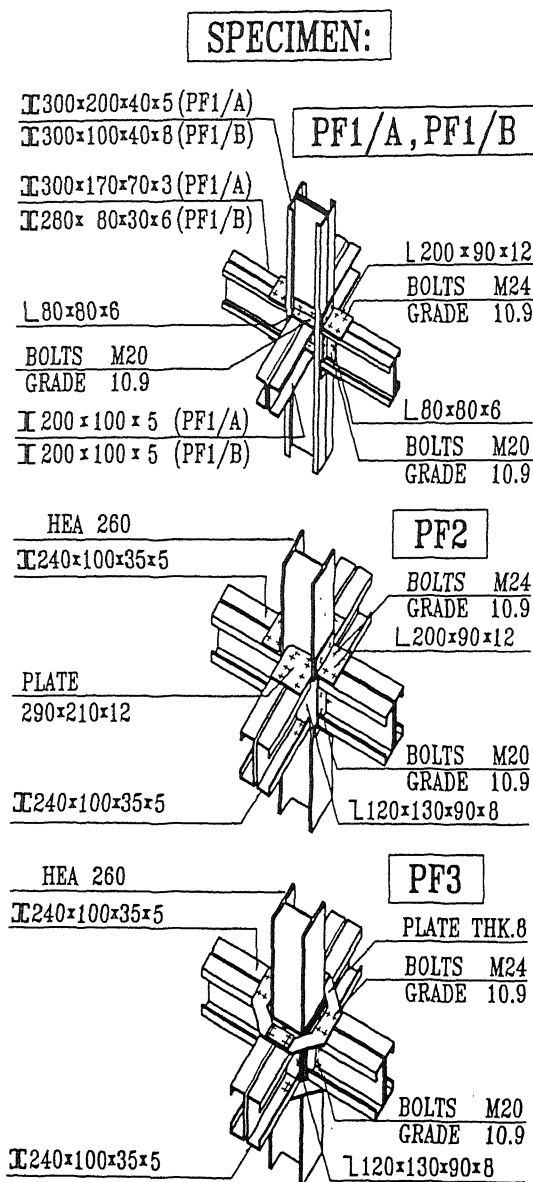
SPECIMEN:

I300x200x40x5 (PF1/A)
I300x100x40x8 (PF1/B)

PF1/A, PF1/B

I300x170x70x3 (PF1/A)
I280x 80x30x6 (PF1/B)

L200x90x12
BOLTS   M24
GRADE   10.9

L80x80x6

BOLTS   M20
GRADE   10.9

L80x80x6

I200x100x 5 (PF1/A)
I200x100x 5 (PF1/B)

BOLTS   M20
GRADE   10.9

HEA 260

PF2

I240x100x35x5

BOLTS   M24
GRADE   10.9
L200x90x12

PLATE
290x210x12

BOLTS   M20
GRADE   10.9
L120x130x90x8

I240x100x35x5

HEA 260

PF3

I240x100x35x5

PLATE THK.8

BOLTS   M24
GRADE   10.9

BOLTS   M20
GRADE   10.9
L120x130x90x8

I240x100x35x5

Figure 1

"Z" shaped cold formed elements and different
types of plates in order to cover a rather
wide range of stiffness and strength.
Loads were applied at the free ends of the
cruciform specimens (Figure 2) by means of
servo-controlled hydraulic jacks, which
enabled the test to be conducted under dis-
placement control. The adopted procedure is
the ECCS "short procedure" (1986), and the
assumed controlled parameter was the dis-
placement at the ends of the cantilever
beams. If the specimen did not collapse at
the maximum level of displacement achievable
by the jacks   (+/-100mm), a low cycle fa-
tigue test was performed cycling at this
maximum level until collapse was attained.
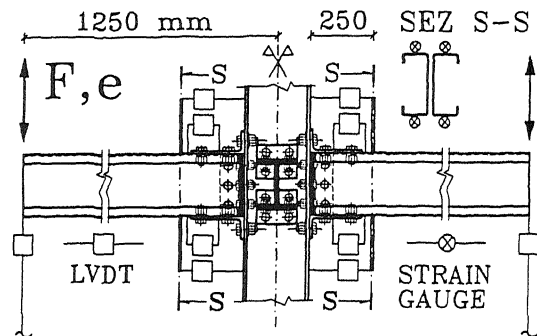Specimens were fabricated as in normal shop

Figure 2

practice (tolerances hence were the usual
ones for constructional steelwork). They were
tested along both the strong (X) and the weak
(Y) axis of the column. Fairly similar re-
sponses were observed for the left and left
connection of each specimen; in the follow-
ing, reference is hence made to the right
joints. The measuring setup (fig. 2) was
designed so as to enable evaluation of both
the global response of the nodal zone and the
local behaviour of the connection and its
components.

3 THE CYCLIC BEHAVIOUR

The responses observed in the tests are here
briefly presented, and discussed, with par-
ticular reference to features of interest for
seismic design. Table 1 reports values of the
most significant stiffness, strength and
ductility parameters: M is the joint moment
(defined as the moment at the column face),
ϕ is the joint rotation, e is the displace-
ment at the free end of the beam, subscripts
y and p refer to yielding and full plasticity
respectively, while subscripts max and u
refer to the maximum resistance achieved and
to the collapse condition.
With reference to the main behavioural fea-
tures specific to the different joints, it
should be mentioned that:
- In joint PF1/A-X (Fig. 3) main sources of
inelastic deformation were column flange
bending (due to flange thinness) and bolt
slip, while connection angles experienced
fairly modest yielding. Hysteretic behaviour
is satisfactory and ductility is high
($|e_u/e_y|$ = 18 and ϕ_{max}/ϕ_y > 20). Collapse was
achieved by fracturing of the column flange.
This made testing the specimen in the weak
plane of the column infeasible;
- Specimen PF1/B-X, with the same form of
connection as PF1/A but thicker members,
showed a more balanced set of deformation
sources: column bending is less important,
while bending of angle cleats is predominant,
though deformation of beam flanges and webs
due to bolt bearing plays also a significant
role as soon as the number of inelastic
cycles increases. The noticeable pinching of
the hysteretic loops is associated with the
plastic deformations observed in the various
connection components, which delay contact to
occur between such elements and member walls.
Collapse was due to fracture of the bottom
flange cleat, at a joint rotation ϕ = 83.3
mrad corresponding to a partial ductility
ratio of 23.8.

TABLE 1

TEST	ϕ_y^+ mrad	$\dfrac{M_y^+}{\phi_y^+}$	$\dfrac{M_{max}^+}{M_p}$	$\dfrac{\phi_{max}^+}{\phi_y^+}$	ϕ_y^- mrad	$\dfrac{M_y^-}{\phi_y^-}$	$\dfrac{M_{max}^-}{M_p}$	$\dfrac{\phi_{max}^-}{\phi_y^-}$	$\dfrac{|e_u|}{|e_y|}$
PF1/A-X	4.0	8.9	0.43	20.3	4.0	8.1	0.39	20.9	18
PF1/B-X	3.5	16.1	0.79	23.8	3.3	17.0	0.80	25.3	18
PF1/B-Y	1.5	10.7	0.53	44.7	1.7	9.4	0.52	37.4	32
PF2-X	2.3	13.1	0.86	35.8	3.1	13.5	1.06	29.0	28
PF2-Y	2.2	19.8	0.70	26.5	2.0	18.2	0.70	25.3	40
PF3-X	4.5	11.1	0.95	20.5	4.5	11.1	1.01	19.3	18
PF3-Y	3.2	11.4	0.74	27.8	3.3	9.4	0.70	26.8	22
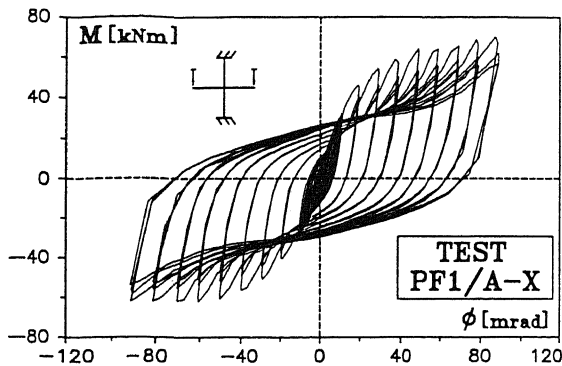
Figure 3

The general response of specimen PF1/B-Y is similar, though the greater pinching and the presence of "softening-hardening" branches at high inelastic displacements, as a consequence of the interaction between the "shear" and rotational deformations of the connection in the absence of any direct attachment between the beam web and the column. Progressive fracturing of several cleats was observed during the last series of "fatigue" cycles at a ratio e_u/e_y =32.

− Flange cleat response characterizes joint PF2-X behaviour in both the elastic and inelastic phases. Besides, bolt slip and bearing deformation affect substantially the shape of the hysteretic loop (Fig. 4). Partial ductility is as high as 28 with reference to the deflection e, and collapse once again is associated with the fracture of a connection component: the top flange cleat.

The Z shaped web connecting element also fractures causing collapse of joint PF2-Y, at the 10th cycle of the "fatigue" test for e_u/e_y = 40.

− The behaviour of the continuity plates of specimens PF3 is fairly complex: basically they are subject to axial forces and biaxial bending. As a consequence, the effectiveness of these elements was lower than expected (in particular in the Y-plane) in all stages of the response, including collapse. Partial ductility ratios in terms of rotation were > 19 and > 26 respectively for the X and Y joints.

The following general remarks on the behavioural characteristics can be made:
   (1) the whole set of responses is generally

satisfactory in terms of stiffness, strength and deformation capacity. It is important to note that no local buckling occurred, in spite of the large inelastic deformations achieved, and of the type of member cross-section. Besides, results indicate that full strength joints may be devised, with a satisfactory cyclic response.
   (2) Y plane joints showed a complex overall response with non negligible vertical displacements, due to a shear transfer mechanism with limited efficiency. An interaction between the shear and rotational behaviour appears to be the cause of certain features of the response as the lack of symmetry and the presence of softening-hardening inelastic branches.
   (3) The sequence of loading may have effects on the onset of slip causing asymmetry of the hysteretic loops.
   (4) The influence of column member type was not significant (i.e. cold-formed vs. hot-rolled columns), but for specimen PF1/A-X with very thin column shapes. However, the absence of axial load applied to the column prevents from drawing any conclusion on this factor.
   (5) As to "elastic" stiffness and ultimate strength, specimen PF2, with cleats and Z shaped elements, behaved slightly better than specimen PF3 with flange continuity plates. Pinching is however more important.

A comparison among all the tested joints is only possible in non dimensional terms. Figures 5 and 6 use the normalized moment and rotation recommended by EC3. All joints are semi-rigid, though the X plane joints lie in the upper part of the semi-rigid region and the Y plane joints lie in the lower part. Initial stiffness if very high for all joints
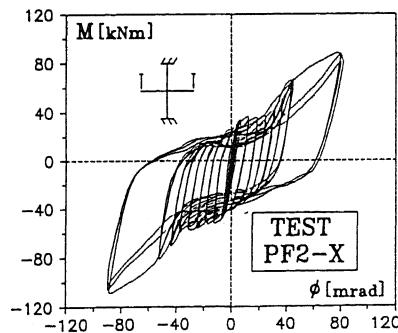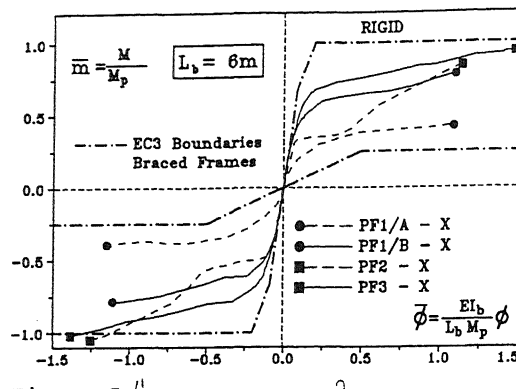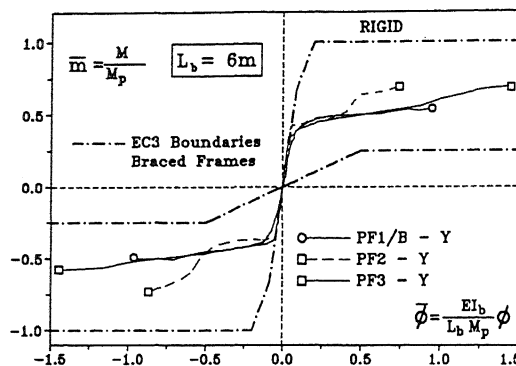

Figure 4

Figure 5

Figure 6

but PF1/A-X, suggesting a fairly high level
of elastic continuity .

4 ASSESSMENT OF EXPERIMENTAL RESPONSES

The joint behaviour described in the previous
section immediately appears as being general-
ly satisfactory. The stiffness, strength and
ductility parameters were already considered
and commented. A more meaningful assessment

of the efficiency of the cyclic behaviour may
be attempted based on the response parameters
defined by the ECCS Recommendations; in
particular reference can be made to the
resistance and rigidity drop as well as to
the dissipated energy. Similar trends were
observed for these parameters, when deter-
mined for the positive and negative emicyc-
les, also in the presence of remarkable
asymmetry in the response; in the following
reference will hence be made to the positive
emicycles only, but for the cumulative ener-
gy.
In figure 7 the resistance drop (defined as
the ratio between the value of the maximum
moments achieved in the third and first cycle
of the set of three cycles at the same dis-
placement value) is plotted as a function of
the partial ductility ϕ/ϕ_y. This ratio is in
most cases rather constant and higher than
0.90, indicating that deterioration of the
response occurring for cycling at the same
level of inelastic deformation is moderate.
In joint PF1/A-X the thinness of the members
increases the bearing deformation, subse-
quently delaying the full effectiveness of
the joint for moment reversal. A similar
comparison within the same set of cycles,
made with reference to the absorbed energy,
confirms that deterioration is modest.

Joint rigidity steadily decreases for all
specimens up to partial ductility values (in
terms of rotation) close to 10. At this value
X plane joints possess between 25 (PF1/B-X)
and 70% (PF1/A-X) of the elastic stiffness
F_y/e_y, while stiffness of Y plane joints
ranges from 25 (PF1/B-Y) to 60% (PF2-Y) of
the elastic one. An additional moderate de-
crease occurs when ϕ/ϕ_y further increases.
Stiffness deterioration also depends on the
number of cycles at the same partial duc-
tility ratio.
Seismic efficiency is mainly related to the
capability of absorbing energy; figures 8-10
refer to this capability: figure 8 shows the
cumulative absorbed energy, figure 9 the
average energy dissipated per cycle in each
set of equal ductility cycles and figure 10
the cumulative energy normalized with respect
to the energy absorbed by an equivalent
ideally elastic-plastic joint. All energies
are plotted with respect to the rotational
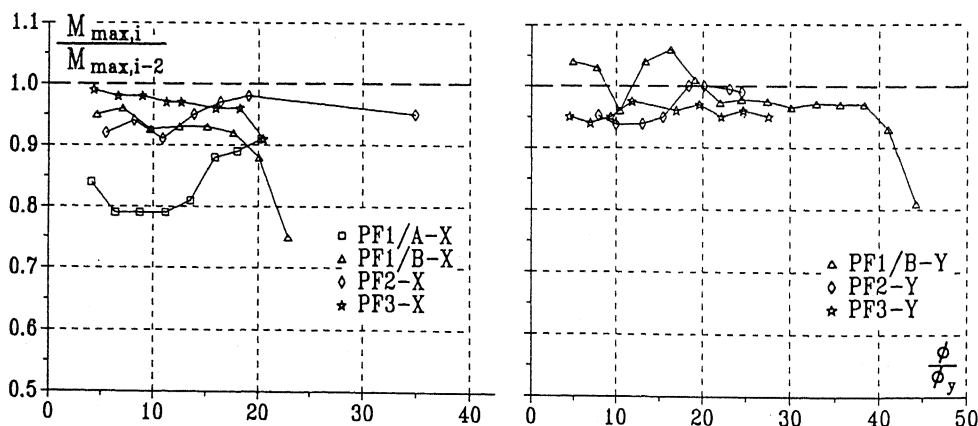partial ductility.
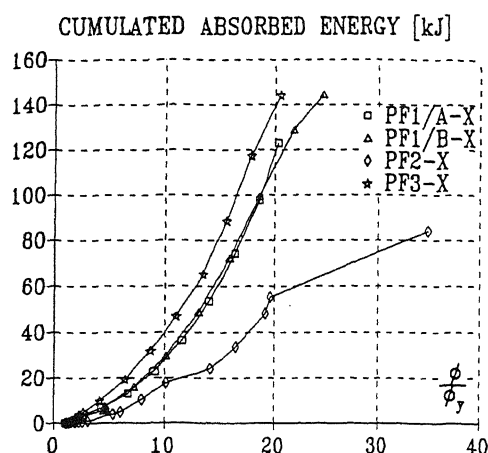The trend of cumulative absorbed energy

Figure 7

CUMULATED ABSORBED ENERGY [kJ]

Figure 8
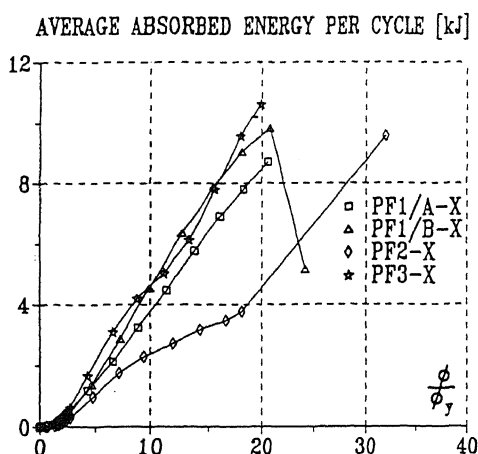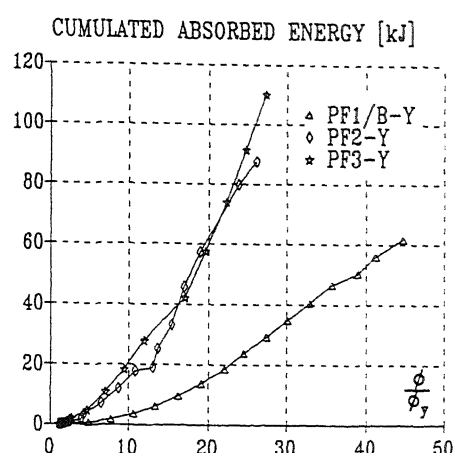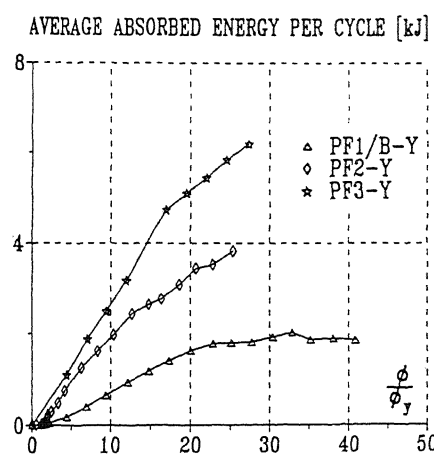
AVERAGE ABSORBED ENERGY PER CYCLE [kJ]

Figure 9

curves is fairly similar for all joints (joint PF2-X was tested with a final significant increase in the applied displacement, see figure 4). Inelastic deformation due to bolt bearing and bending of connection components affects more remarkably the overall response of joints PF2-X and PF1/B-Y, which show a significant pinching of the loop. This is clearly pointed out by the energy curves in figures 8 and 9. The absorption capacity seems satisfactory with reference to both the total amount and the way this has been built up (energy dissipation capacity is raising with joint rotation (Fig. 9) practically up to attainment of collapse). Deterioration of joint behaviour (basically in terms of stiffness and pinching) is increasingly balanced by the "hardening" of the response, as it appears by the normalized cumulative energy curves plotted in figure 10. The negative influence of the interaction between shear and rotational deformations causes a reduction of the effectiveness of the response of Y plane joints in the range of the higher ductility ratios. If the two specimens are considered for which a direct comparison is possible, specimen PF3 behaves better in

terms of energy absorption than PF2 in both planes, despite the opposite indication provided by the envelopes of the M - φ curves. This is certainly due the lower pinching in the response of PF3.

5 CONCLUDING REMARKS

The feasibility of concentrating seismic energy dissipation in beam-to-column joints, also in presence of cold formed members, may be regarded as the main outcome of the tests, on which this paper reports. A satisfactory cyclic performance was observed even in the presence of connected elements with slender cross section. In particular the deformation capacity was fairly high, and collapse was associated to the fracture of joint elements, whilst local buckling did not occur. Furthermore, it should be noted that all joints tested are semi-rigid and that connection detailing is rather simple.
Joint performance, however, should not be considerd per se, but in relation with the design requirements of frame seismic response. A first appraisal of semi-rigid joint
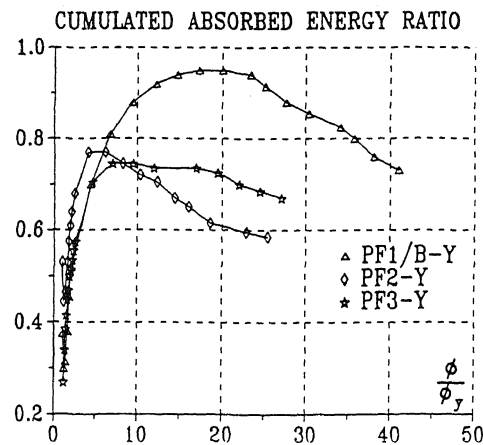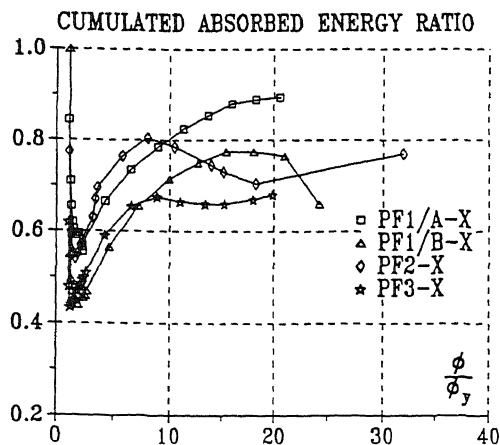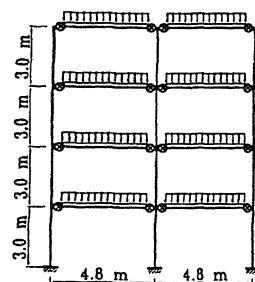
CUMULATED ABSORBED ENERGY RATIO

PF1/A-X
PF1/B-X
PF2-X
PF3-X

$\frac{\phi}{\phi_y}$

CUMULATED ABSORBED ENERGY RATIO

PF1/B-Y
PF2-Y
PF3-Y

$\frac{\phi}{\phi_y}$

Figure 10

⊗ SEMI-RIGID JOINT

Beams   ][280x80x30x6

Columns   HE 300 B

Figure 11

action in cold formed frames was obtained via a limited series of nonlinear analyses of the frame in figure 11. The ground acceleration of the El Centro 1979 earthquake (scaled by a factor 1.3) and of the 1976 Friuli one (recorded at Tolmezzo) were selected as seismic excitation. Both earthquakes are demanding in the frequency range of interest for semi-rigid steel frames. The joints considered were the PF1/B-X and the PF3-X. A first evaluation of the numerical results indicates that general responses are satisfactory, although joints stiffness is an important factor to reduce unfavourable second order effects. The inelastic joint cycles were limited in number and amplitude (maximum rotation attained by joints was 15 mrad, corresponding to $\phi/\phi_y = 4$), in spite of a significant contribution to energy dissipation. No definitive conclusion can however be drawn at present. A wider numerical study will be carried on in the subsequent phases of the research, also aimed at developing a set of data to be used as background in a reviewing process of the present ECCS testing recommendations, which appear to be too severe. Braced frames will also be included in the study and the proposed design philosophy checked, which combines the energy absorption capabilities of joints and bracings

in order to achieve a satisfactory frame performance. A complementary extension of the experimental study will be carried on to investigate the influence of factors such as the column axial load (which should play an important role for cold formed columns) and the cross sectional characteristics. This will permit development of cyclic behavioural models of joints suitable for design analysis.

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