Influence of Concrete on the Cyclic Behaviour of Composite Connections

Influence du béton sur le comportement d'assemblages mixtes sous action cyclique

Betoneinfluss auf das Verhalten Verbindungen von Verbundbauten

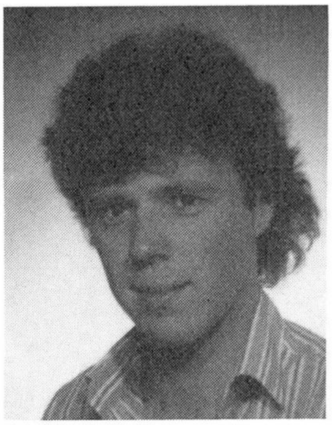
Giulio BALLIO
Professor
Politecnico
Milan, Italy

André PLUMIER
Dr. Eng.
Université de Liège
Liège, Belgium

Bruno THUNUS
Res. Engineer
Université de Liège
Liège, Belgium

Giulio Ballio, born 1940, is full Professor of Steel Structures at the Politecnico of Milan. He was chairman of ECCS TC 13-Eartquakes and now he chair ECCS-TC 1 Safety and Loadings. His professional activity deals with the design of both concrete and steel structures.

André Plumier, born in 1947, obtained his engineering degree in 1970 at the University of Liège. He was in charge of research on fatigue, stability and welding residual stresses. He obtained his Ph.D in 1980 and worked in the earthquake engineering field.



Bruno Thunus, born in 1965, obtained his engineering degree in 1987, at the University of Liège. He worked as a research engineer at the University since then.

SUMMARY

A recent research programme on prefabricated composite steel concrete structures under seismic loading provides an opportunity to investigate several aspects of the influence of concrete on the mechanical behaviour of the beam-column connection zone.

RÉSUMÉ

Une recherche récente sur des structures préfabriquées mixtes acier-béton sous action sismique permet de mettre en évidence plusieurs aspects de l'influence du béton sur le comportement des zones d'assemblage poutre-poteau.

ZUSAMMENFASSUNG

Anlässlich eines Forschungsprojektes über Fertigteile in der Verbundbauweise welche durch Erdbeben beansprucht werden, wird die Wirkung des Betons auf das Verhalten von Rahmenknoten erläutert.

1. INTRODUCTION.

Most of the research work of the last decades in the field of composite steel concrete structure is bearing on their static behaviour and their fire resistance. In the field of seismic resistance, quite a lot of work was done in Japan and in North America. However, these results are most of the time quite specific ; gaps in knowledge remain, to which corresponds a lack in precision for design rules on composite steel concrete structure in seismic codes.

To fill some of these gaps and define a prefabricated fire resistant constructional system which would also be earthquake resistant, a research on seismic resistance of composite structures was started in 1987, under ARBED and European Community sponsorship. It involves work from several Universities in Europe : Milano, Darmstadt, Aachen, Wüppertal and Liège.

The research programme includes as a first step the definition of some typical connections to be tested and tests on interior (Serie 1) and exterior (Serie 2) column and interpretation of tests. A second step is devoted to the definition of design criteria, the development of numerical models for connections, allowing a good evaluation of the overall behaviour of a complete structure.

2. TEST SERIE 1 - EXTERIOR COLUMNS.

2.1. Definition of the tested elements.

The contraints of the choice between various technical solutions are :
- all sections in the real structure are H sections with concrete between the flanges
- fire resistance must exist ; however fire and earthquake resistance must not necessarily be realised together.

A choice of 6 basic connection designs was made, taking into account various parameters : rigidity, ductility, possibility of 1 column - 4 beams connection, the easyness of assembly, the quantity of concreting and welding, the workshop cost and transportation cost. Six design, from A to F on Figure 1, were finally selected. In order to evaluate the influence of the composite aspect of these solutions, each design is realised in various combinations : concrete between the flanges only or with slab, reference case without any concrete. Eighteen specimen (see Table 1) are finally prepared for testing.

2.2. Design of specimens.

All the test specimens have the same exterior dimensions. Sections are HEA 260 for beams and HEB 300 for columns.
The material characteristics are :
- steel grade Fe 360 (fy = 235 N/mm²) for beams and columns
- steel grade Fe 510 (fy = 355 N/mm²) for connecting parts
- yield strength of the rebars : 500 N/mm².
- concrete strength : 25 N/mm²
- bolt strength 10.9 (fu = 1000 N/mm²).

The joints are designed according to Eurocodes 3,4 and 8. Eurocode 8 requires to take into account the real yield strength value (+/- 10 %), the calculations are realised with fy = 290 N/mm instead of 235 N/mm² for beams and columns. This value is confirmed by tensile tests on steel.

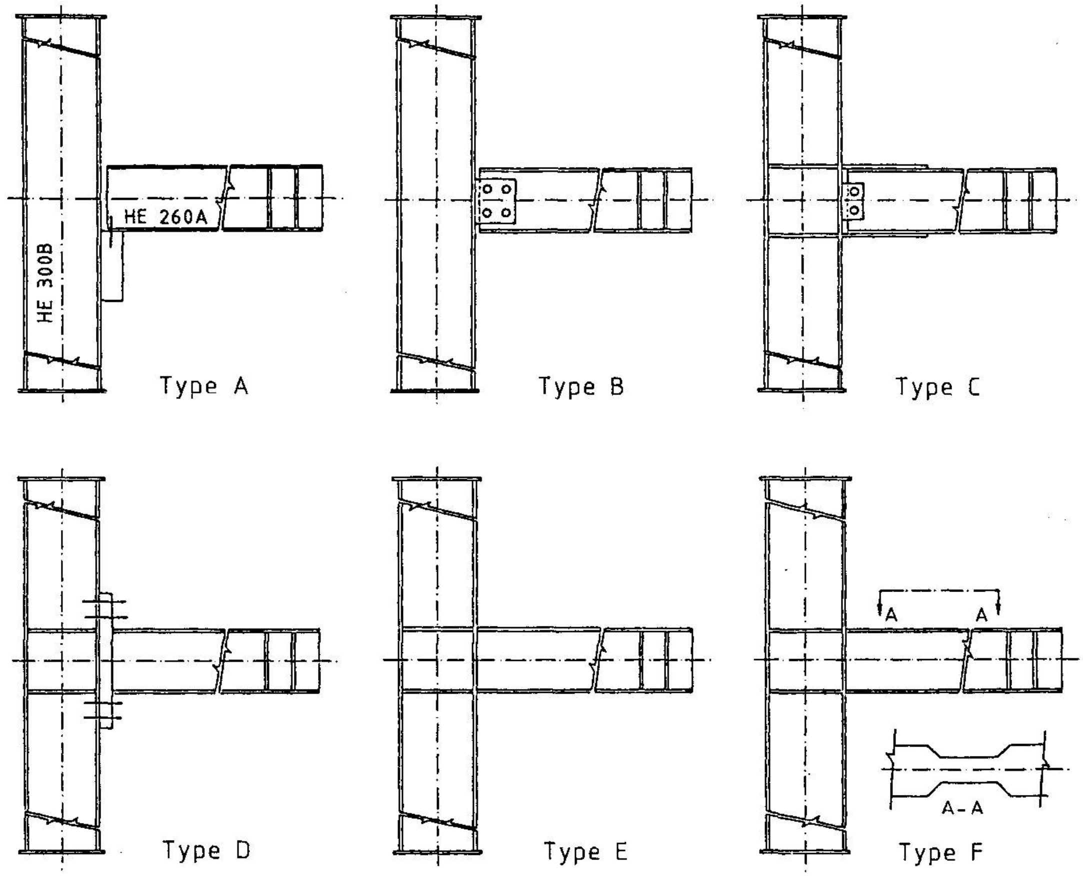


Figure 1.

	bare steel elements	composite column steel beam	composite elements without sleeb	composite elements with slab
full-rigid	C1, D1, E1	E2	C2,D2,D4,D5,E3	C3, D3
semi-rigid	D6, F1		D7, D8, F2	
hinged				A1, B1

Table 1.

In such specimens, several phenomenons can correspond to ultimate behaviour :
a) plastic hinge (bending) in the beam
b) plastic deformation of the panel zone of the column (shear)
c) plastic hinge (bending) in the column
d) plastic deformation in the connecting plate
e) weld failure, bolt failure.

When yield strengthers and mechanisms are known, it is up to the designer to choose the end of its specimens. The lines followed for the design were :
c) is avoided by the choice of the sections
a) and b) are computed at same resistance level for bare steel elements and that design is kept when concrete is put in : $R_{dy\ panel} = R_{dy\ beam}$.

d) and e) are obviated by compliance to Eurocode 8 criterion on connections $R_{d\ connections} \geq 1{,}2\ R_{y\ beam}$.

2.3. Testing and analysis considerations.

The deformations to be observed during the tests can be subdivided into several terms. The three terms Θ_h, Θ and Θ_s - see figure 2, characterize the various components of the total deformation $\Theta_t = \Theta_h + \Theta_c + \Theta_s$

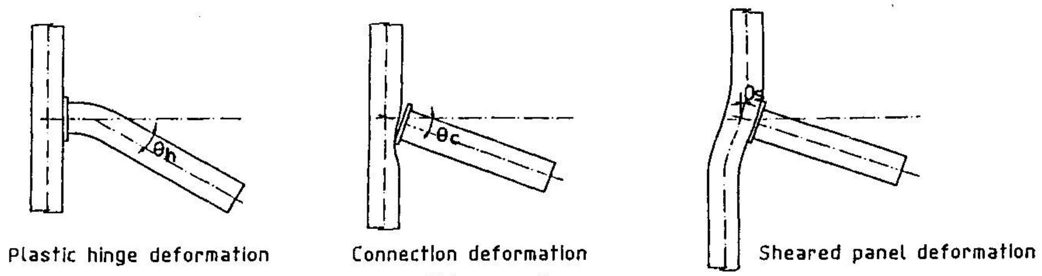
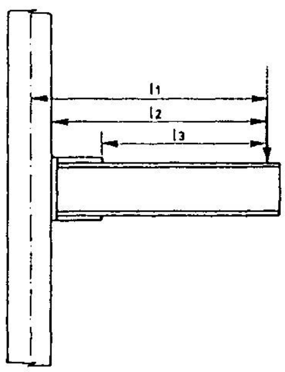


Figure 2.

However Θ_c is difficult and useless to measure for rigid connections and a unique parameter called "beam plastic rotation" $\Theta_b = \Theta_h + C_c$, was prefered.



The significant mechanical parameter to interpret the test is the bending moment $M = P.\ell$. However, a question arise when choosing between ℓ_1, ℓ_2, ℓ_3 (figure 3). This question cannot be solved in one unique fully satisfactory way for various reasons, but finally ℓ_1 is adopted because it is the most commun data and it gives safe results.

Figure 3.

The tests of Serie 1 and Serie 2 are realised at Politecnico di Milano, in a test set up described in [12]. They are static cyclic test, with steps of increased displacements, according to ECCS procedure [13]. The results are presented in classical M - Θ hysteresis curves and also in "interpretation functions" of the ECCS procedure. Examples of these presentations are given at Figure 4a) and 4b).

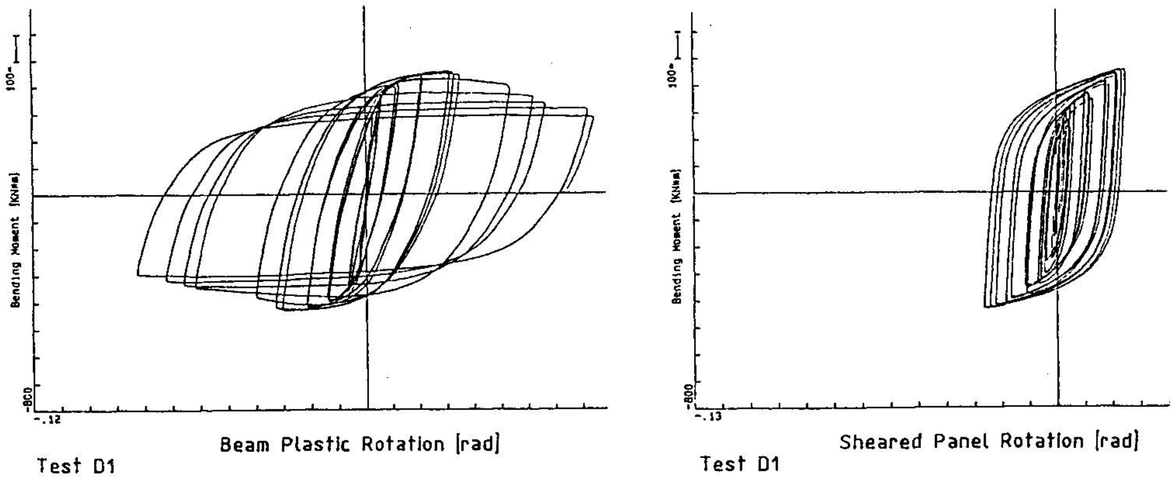
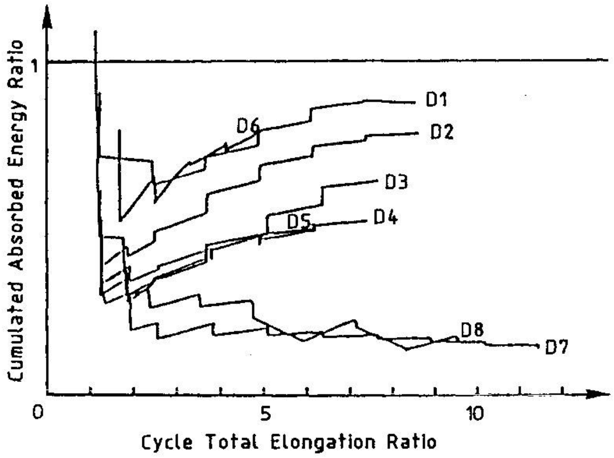


Figure 4 a).

M - Θ curves for test D1 (2 examples).



Comparison through interpretation function of energy according to ECCS procedure. Example of serie D of test specimen

Figure 4.b

Test results presentation.

2.5. Some conclusions on the results obtained in Serie 1.

The results of Serie 1 allow some observations and conclusions, which are valid in the design context defined in 2.2.

- Concreting between the flanges only makes the yielding mechanism move from the panel zone of the column to the beam.
- The presence a slab (and the corresponding increase in beam inestia) moves this mechanism back to the panel zone.
- Concreting between the flanges brings an increase in initial rigidity of about 25 to 30 %. The increase grows to 55 % with the slab.
- The bending resistance of the various tested elements is characterized on Figure 5 . My is the plastic bending moment in the ECCS testing procedure meaning [2]. M_{max} is the maximum bending moment observed during the test. M_u corresponds to failure. M_{pb} is the plastic resistant moment of the bare steel beam. (M_y is very close to M_{pb} for bare steel elements).

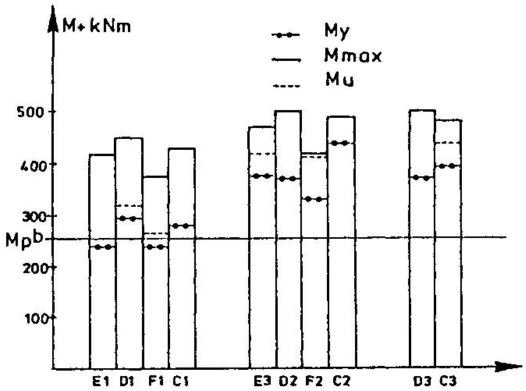


Figure 5.

- Elements concreted between the flanges are 30 to 50 % higher in design resistance M_y than bare steel elements.
- On the contrary M_{max} are quite similar for both kind of elements.
- The C connections are bad in composite structures because they concentrates all yielding in a narrow area near the welds, bringing premature weld failures
- Ductility levels are the greatest for bare steel elements, but composite elements have ductility levels higher than 6.

3. TEST SERIE 2.

3.1. Definition of the tested elements.

The basis data for the choice of the elements of test Serie 2 interior columns are the same as for serie 1. It is however decided to test more on the possible "locking" of the panel zone by the concrete by designing more specimens in which the weak resistance area is the panel zone and, at the opposite, to test some specimens with strong reinforcement of the panel zone.

The results obtained in Serie 1 also influence the choice :
- thin abutting plates are abandonned (bolt bending gives premature failure).
- fillet welds are replaced by butt welds, because the design of fillet welds according to Eurocode 3 and 8 gives enormous welds.
- the panel zone size in increased in type D connections, because it appears that the real panel zone heigth is that of the abutting plates.
- The fully welded connection, quite unpractical on site, is modified into a bolted connection for the web.

But for those small changes, Serie 2 elements corespond to Serie 1.

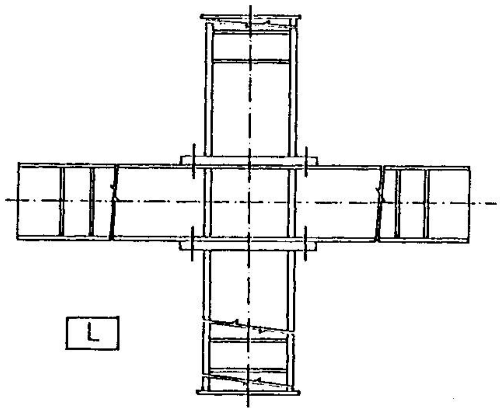


Figure 6.

Connection L, Figure 6, is a fully new design in which the beam is continuous and the column interrupted.

3.2. Some conclusions on the results obtained in Serie 2.

- The results obtained in Serie 1 are confirmed : concrete increases the rigidity and the design resistance M_y of the bare steel element to the level of resistance this bare steel element has after steel hardening has taken place.
- Many specimens "fail" by a panel zone mechanism, but it is so ductile that the test is stopped because of the displacement range of the test set up (500 mm), without any drop in resistance.
- The panel zone resistance under cyclic loading seems significantly higher than foreseen by the Eurocode 3 or U.B.C.
- The increase in heigth of the panel zone tested in element J (equivalent D) gives the expected increase in resistance.
- With strong reinforcement of the panel zone, the yield mechanism moves to the beam. This bring a higher design resistance M_y, but also a less ductile failure : the ductility level drops from 9 to 4 ; those failures take place in the connecting elements, welds, bolts or bolt anchorage area.

4. CONCLUSIONS.

The conclusions given above set forward positive influences of the use of composite steel concrete structure, in comparison to bare steel structures : higher design resistances, higher rigidities, and same level of ductility. These value of resistance and rigiditiy seems reliable because the negative influence of the crushing of the concrete existing in the large displacement field is there replaced by the positive influence of strain hardening.

Another general conclusion of these test is on the importance for the designer to keep the "be ductile" idea in mind. The tests show how often an increase in resistance is correlated with a drop in ductility. It seems for instance that the yielding of the panel zone should not be avoided, since it gives a ductile dissipative zone, with, in case of composite H sections of the type studied in our research, a resistance higher than foreseen.

BIBLIOGRAPHIE.

1. Ballio, Calado, Steel bent sections under cyclic loads. Costruzioni Metalliche n°1 - 1986.
2. Recommanded testing procedure for assessing the behaviour of structural steel under cyclic loads, ECCS, Publication n°45, 1986.