European Development of Seismic Design Guidelines for Composite Steel Concrete Structures

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ABSTRACT

Until recently, many aspects of earthquake resistant design of composite steel concrete structures were unclear. Recently, an important research effort presented in companion papers was made in Europe. It involved experimental activity, numerical modelling and the analysis of existing literature references.

Design guidelines have been developed, which are now introduced in Eurocode 8. Some aspects of the problems met and the options taken are presented here.

INTRODUCTION

In Europe, an important and recent research effort, focused on seismic design of moment frame structures with rigid connections, has been made. Two main design options exist for such frames:

- either the composite behaviour of beams at their connections to the columns is neglected; then, the design can be based on the properties of the beams steel sections only; however, this requires a proper disconnection of the slab from the steel sections, because the capacity design of columns requires that the plastic moments of beams are not underestimated
- or the stiffness and plastic resistance of the composite beams are considered in the design; then, design data must be provided, which define the effective widths of beams, the sections and lay out of re-bars in the slab, the proportions of sections, the behaviour factors, requirements on shear connectors, etc…

This has been achieved and introduced in a new Section of Eurocode 8, which is briefly presented here. This new Section and its research background are presented in detail in the ICONS Report (Plumier 2001).

DESIGN CONCEPTS

Earthquake resistant composite buildings can be designed to one of the following concepts:

Concept a	Dissipative structural behaviour with composite dissipative zones.
Concept b	Dissipative structural behaviour with steel dissipative zones.
Concept c	Non-dissipative structural behaviour.

Two structural ductility classes, I (Intermediate) and S (Special) are defined. They correspond to an increased ability of the structure to dissipate energy through plastic mechanisms. A structure belonging to a given ductility class has to meet specific requirements on connections, steel sections and detailings. An available local plastic rotations, which is 25mrad for ductility class I and 35mrad for ductility class S, is required. Ordinary non seismic design (symbol O) is allowed in low seismicity regions.

DESIGN CONCEPT	BEHAVIOUR FACTOR q	DUCTILITY CLASS
c Non dissipative structure	$1\leq q\leq 1,5$	O for Ordinary
a or b Dissipative structure	$1,5<q< 4$	I for Intermediate
a or b Dissipative structure	$q \geq 4$	S for Special

Table 1. Design concepts, behaviour factors and structures ductility classes

MATERIALS

The use of concrete class lower than C20/25 or higher than C40/50 is not allowed. Reinforcing steel, bars and welded meshes, considered in the plastic resistance of dissipative zones have to satisfy requirements on the ratio f_u / f_y and elongation. Except for closed stirrups or cross ties, only ribbed bars are allowed as reinforcing steel. Welded meshes not complying with the ductility requirements may be used in dissipative zones. In that case, ductile reinforcements duplicating the mesh must be placed.

The problem behind this is that in moment frames submitted to earthquakes, a reliable negative plastic moment in the connection zone requires the presence of ductile reinforcements, while the beam plastic moment used in the capacity design of column considers all contributions of the reinforcements, non ductile welded mesh included. When duplication of non ductile reinforcement by means of ductile reinforcements is realised, the capacity design of columns comes to an over design of these columns. In practice, the more economical solution will be obtained without continuity of non ductile reinforcements in dissipative zones.

STRUCTURAL TYPES AND BEHAVIOUR FACTORS

Composite steel concrete structures are assigned to one of the following structural types according to their behaviour under seismic actions

- Moment resisting frames in which beams and columns can be either steel or composite.
- Composite concentrically braced frames, in which columns and beams can be either steel or composite, braces being structural steel.

- Composite eccentrically braced frames (see explanations further).
- Composite structural systems which behave essentially as reinforced concrete walls.
- Composite steel plate shear walls, vertical steel plates with concrete encasement

The behaviour factor q accounts for the energy dissipation capacity of the structure and takes, for what are typical composite systems, the values given in fig.1. If the designer is willing to, the structural overstrength α_u / α_1 can be evaluated. The values of q for moment frames and braced frames composite structures are similar to those for steel and reinforced concrete structures. It has been shown (Sanchez 1999) that the values of q does not differ significantly from one material type to another, as long as the available local ductility are similar and a correct evaluation of the "first yield" in the structure is made. This one should correspond to the loading for which the first plastic hinge starts plastic rotation. However, some other factors, like a reduced low cycle fatigue resistance of T sections, in comparison to symmetrical sections, and the effect of damping due to friction at steel concrete interface, could influence the proposed values of q.

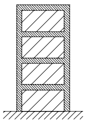
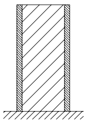
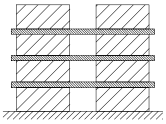
	Ductility Class	
	S	I
e) Reinforced concrete shear wall elements. $\frac{\alpha_u}{\alpha_1} \approx 1.1$ TYPE I  Steel or composite moment frame with concrete infill panels. TYPE 2  Concrete walls reinforced by encased vertical steel sections. TYPE 3  Concrete shear walls coupled by steel or composite beams.	$4\frac{\alpha_u}{\alpha_1}$	$2.5\frac{\alpha_u}{\alpha_1}$
f) Composite steel plate shear walls with RC elements. $\frac{\alpha_u}{\alpha_1} \approx 1.2$	$4\frac{\alpha_u}{\alpha_1}$	$2.5\frac{\alpha_u}{\alpha_1}$

Fig.1. Structural types and maximum associated behaviour factors

GENERAL CRITERIA FOR DISSIPATIVE STRUCTURAL BEHAVIOUR

Structures with dissipative zones are designed such that dissipative zones have adequate ductility and resistance and such that yielding, local buckling or other phenomena due to hysteretic behaviour do not affect the overall stability of the structure.

Semi-rigid and/or partial strength connections are permitted, provided that they have adequate rotation capacity consistent with global deformations, that members framing into the connections are stable and that the effect of connections deformations on global drift is taken into account. Non-dissipative parts of dissipative structures and the elements connecting dissipative to non dissipative parts of the structure shall have sufficient overstrength to allow the development of cyclic yielding of the dissipative parts.

PLASTIC RESISTANCE OF DISSIPATIVE ZONES

Two plastic resistances of dissipative zones are considered in the design of composite steel concrete structures: The lower bound plastic resistance of dissipative zones is the one considered in design checks concerning sections of dissipative elements; e.g. $M_{Sd} < M_{p\ell,Rd}$. It is computed considering the concrete and only the steel components of the section which are ductile. The upper bound plastic resistance of dissipative zones is the one considered in the capacity design of elements adjacent to the dissipative zone; it is established considering the concrete and all the steel components present in the section, including those that are not necessarily ductile, e.g. welded meshes.

DETAILING RULES FOR COMPOSITE CONNECTIONS IN DISSIPATIVE ZONES

Local design of the reinforcing bars needed in the concrete of the joint region has to be justified by models that satisfy equilibrium. In fully encased framed web panels of beam/column connections, the panel zone resistance can be computed as the sum of contributions from the concrete and steel shear panel, if the aspect ratio h_b/b_p of the panel zone satisfies definite conditions. In partially encased stiffened web panels, a similar assessment is permitted with additional conditions.

When a dissipative steel or composite beam is framing into a reinforced concrete column or into a fully encased composite column, vertical column reinforcements with design axial strength equal to the shear strength of the coupling beam are placed. These reinforcement are confined by transverse reinforcement. The presence of face bearing plates in the beam is required.

DETAILING RULES FOR MEMBERS .

Sufficient local ductility of members which dissipate energy under compression and/or bending is ensured by restricting their width-to-thickness ratios. For dissipative zones of composite members, the requirements of cross-section are those given in Table 2.

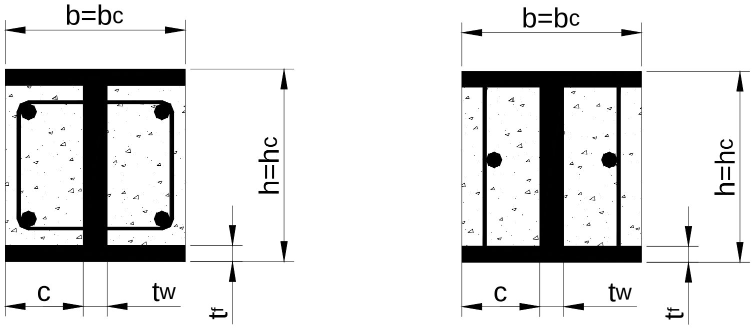
Ductility Class of Structure	S	I	O
Behaviour Factor (q)	$q \geq 4$	$1.5 < q < 4$	$1 \leq q \leq 1.5$
Partially Encased (flange outstand limits c/t)	10 ε	15 ε	21 ε
Filled Rectangular (h/t limits)	24 ε	38 ε	52 ε
Filled Circular (d/t limits)	$80\ \varepsilon^2$	$85\ \varepsilon^2$	$90\ \varepsilon^2$

$\varepsilon = (fy/235)^{0.5}$

Table 2. Relationship between behaviour factor and cross-section classification

The limits given in Table 2 are based on available investigations on the rotation capacity of composite members. They provide cyclic rotation capacity over 35 and 25 mrad for Ductility Classes S and I, respectively, under a maximum axial load representing 30% of the plastic section capacity. The slenderness limits of partially encased sections for the three ductility classes correspond to the limits of Class 1, 2 and 3 in Eurocode 4. Also, the limits given for Ductility Class O of filled sections correspond to Class 3 in Eurocode 4.

For partially-encased members, additional links welded to the inside of the flanges, as shown in Figure 7.2, may delay local buckling in the dissipative zones. The limits given in Table 2 for flange slenderness may be increased if these bars are provided at a longitudinal spacing (s_l) which is less than the flange outstand: $s_l/c < 1.0$. For $s_l/c < 0.5$, the limits given in Table 2 may be increased by up to 50%. For values of $0.5 < s_l/c < 1.0$, linear interpolation may be used.



a) hoops welded to web b) straight bars welded to flanges

Fig 2. Details of transverse reinforcements.

In practice, columns are generally not designed to be dissipative, so that design to Eurocode 4 solve the problem. However, there remain circumstances for which designing to have dissipative sections : bottom of all types of columns in moment frames at ground level, top and bottom zones at any storey level in fully encased columns ("critical zones" of reinforced concrete). For this reason, design guidance for dissipative columns is given.

Three cross-section types are considered: fully-encased, partially-encased and filled sections.

Concerning bond and friction shear resistance at the steel-concrete interface, they are not reliable in cyclic response conditions, so that in non-dissipative columns, the design values of the bond stress are only one-third of the static bond capacity. Wherever the bond stress is inadequate, it is necessary to provide shear connectors. In the design of all types of composite columns, the resistance of the steel section alone or the combined resistances of the steel section and the concrete encasement or infill may be considered in bending, but there remain strong restrictions, due to a lack of research and contradictory test results for what concerns the shear resistance. For fully encased section, it is the concrete section resistance. For partially encased section, it is the steel section resistance, except if special details are developed. For filled sections, it is either the steel or the concrete section which has to be considered as resistant.

In steel beams composite with slab, the design objective defined at present is to maintain the integrity of the concrete slab during the seismic event, while yielding takes place in the bottom part of the steel section and/or in the re-bars of the slab. This is achieved by means of the following indications.

Effective width of slab b_{eff} have to be considered in the definition of the moment of inertia and of the plastic moments of sections; they are defined at Table 3 (note: in general $b_{eff} = 2\ b_e$).

To achieve ductility in plastic hinges, the ratio x/d of the distance x between the top concrete compression fibre and the plastic neutral axis to the depth d of the composite section should comply with:

$x/d < \varepsilon_{cu} / (\varepsilon_{cu} + \varepsilon_a)$ where: ε_{cu} is the crushing strain of concrete in cyclic conditions

ε_a is the total strain in steel at Ultimate Limit State

Beams conceived to behave as composite elements in dissipative zones may be designed for partial shear connection in any sagging moment region according to Eurocode 4, provided that the minimum connection degree is not less than 0,8 and the total resistance of the shear connectors within any hogging moment region is not less than the plastic resistance of the reinforcement. When a profiled steel sheeting with ribs transverse to the supporting beams is used, a reduction by a rib shape efficiency factor k_r has to be considered. Full shear connection is required when non ductile connectors are used. For beams with slab in which dissipative zones are located, specific reinforcements of the slab called "seismic rebars" must be present in the zones of connections of the beams to the columns. They are sketched at Fig.3. Their sections As and their layout must be designed to reach ductility. This has led to the definition of an Annex on slab design in the connection zones.

b_e	Transverse beam	b_e for M_{Rd} (PLASTIC)	b_e for I (ELASTIC)
At interior column	Present or not present	For M^- : 0,1 ℓ, For M^+ : 0,075 ℓ	0,05 ℓ 0,025 ℓ
At exterior column	Present as an edge beam fixed to the column - in the plane of the columns, with connectors for full shear and specific detailing for anchorage of re-bars - exterior to the column plane, with re-bars of the hair pin type	For M^- : 0,1 ℓ, For M^+ : 0,075 ℓ	0,05 ℓ 0,025 ℓ
At exterior column	Not present or no re-bars anchored	For M^- : 0 For M^+ : $b_c/2$ or $h_c/2$	0 0,025 ℓ

Table 3. Effective width of slab.

ANALYSIS OF STRUCTURES

In moment frames, the plastic resistance of a composite section can be computed considering only the steel section, if the slab is totally disconnected from the steel frame in a circular zone around a column of diameter $2b_{eff}$, b_{eff} being the greater of the effective width of the beams connected to that column. Total disconnection means no contact between slab and any vertical side of steel element (columns, shear connectors, connecting plates, corrugated flange, omega steeldeck nailed to the flange of steel section, …).

For a dynamic elastic analysis of the structure under earthquake action, the stiffness I1 of composite sections in which the concrete is in compression is computed considering the effective concrete section and a modular ratio $n = E_a / E_c = 7$.

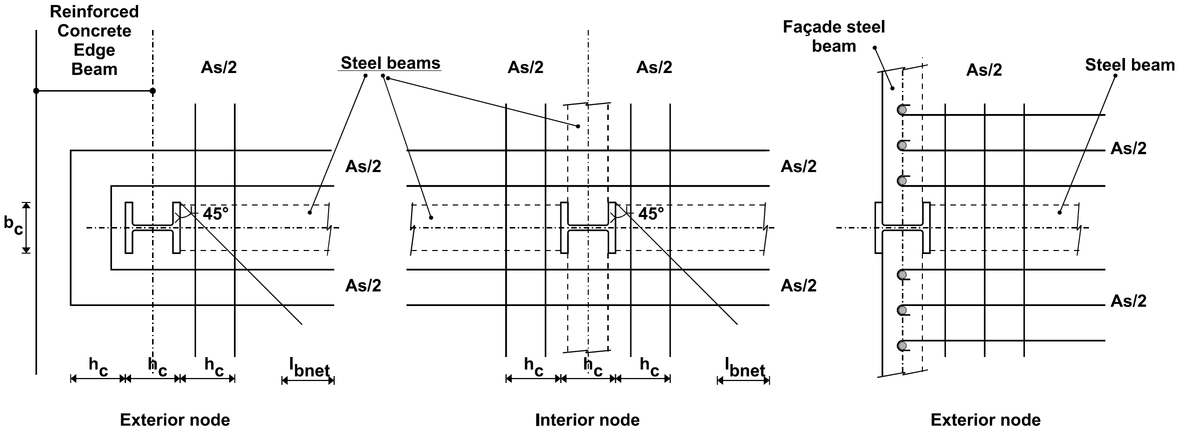


Fig. 3: Layout of “Seismic Rebars”

The stiffness I2 of composite sections in which the concrete is in tension is computed considering that the concrete is cracked and that only the steel parts of the section are active.
The structure is analysed considering the presence of concrete in compression in some zones and in tension in other zones. A unique equivalent moment of inertia can be used:
$I_{eq} = 0.6\ I_1 + 0.4\ I_2$

RULES FOR COMPOSITE ECCENTRICALLY BRACED FRAMES

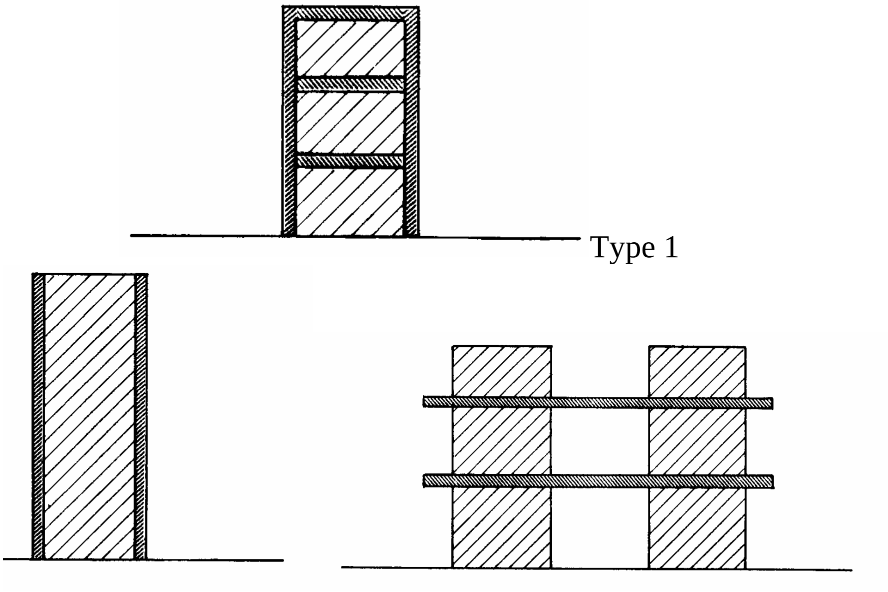
Composite frames with eccentric bracings are designed such that the dissipative behaviour occurs essentially through yielding in shear of the links. All other members should remain elastic and failure of connections should be prevented. Columns, beams and braces can be structural steel or composite. In principle, the door is open to steel only or composite design for all members. However, due to the large deformations in dissipative zones of eccentric bracings (8mrad), there are uncertainties which are not acceptable for dissipative zones: an underestimated link capacity would lead to an under-design of braces and columns and possibly to their failure. The gap in knowledge is similar for what concerns “disconnection” of the slab in that range of local rotations. Links working in bending in beam elements with slab raise an evaluation problem. Beam links made of steel section with slab yielding in shear correspond to a mastered situation, because the slab contribution in the shear resistance is negligible. Vertical steel links are also accepted. On that basis, design guidance is: links can be made of steel sections, possibly composite with slabs; they may not be encased; they must be short or intermediate with a maximum length e:

$$e < 2M_{p,\ link} / V_{p,\ link}$$

STRUCTURAL SYSTEMS MADE OF REINFORCED CONCRETE SHEAR WALLS COMPOSITE WITH STRUCTURAL STEEL ELEMENTS.

When properly designed, these systems have shear strength and stiffness comparable to those of pure reinforced concrete shear wall systems. The structural steel sections in the boundary

members however increase the flexural resistance of the wall and delay plastic flexural hinging (tall walls). Like for reinforced concrete structures, two levels of ductility and two values of the behaviour factor are defined, depending on the level of requirements in the detailing rules.
Structural systems Types 1 and 2 (Fig 4) are designed to behave as shear walls and dissipate energy in the vertical steel sections and in the vertical reinforcements. Type 3 is designed to dissipate energy in the shear walls and in the coupling beams.



Type 1

Type 2

Type 3

Fig. 4. Composite shear walls

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