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THE ECCS ACTIVITY IN THE FIELD OF RECOMMENDATIONS FOR STEEL SEISMIC RESISTANT STRUCTURES

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SUMMARY

This paper summarizes the activity recently developed by the Committee TWG 1.3 "Seismic Design" of the European Convention for Constructional Steelwork (ECCS).
The main goal of this activity is to provide unified rules for testing and designing seismic resistant steel structures.
The main principles at the base of the new proposed Recommendations are here presented and commented.

INTRODUCTION

The European Convention for Constructional Steelwork develops his wide activities by means of several Technical Committees. The earthquake subject is covered by the Committee TWG 1.3 "Seismic Design", which is devoted to carry out studies and research on the behaviour of steel structures under seismic actions.

During the last few years the developments of the research activity on seismic resistant steel structures have been mainly oriented to produce results in the following directions:

a) experimental analysis on full scale steel joints, subassemblages and bracings under monotonic and cyclic loading conditions;

b) analytical models which can be calibrated on the experimental evidence in order to be introduced in the calculation of the whole structure;

c) numerical simulation of plane or space structural schemes in both static and dynamic range in order to evaluate the effect of non-linearities (geometrical and mechanical) on the overall performance;

d) simplified method in order to find out more or less refined values of the ductility factors to be used with practical purposes in the recommendations.

The achievement of these steps allows to codify the main results in the field of research and design of seismic resistant steel structures. As soon as a satisfactory degree of knowledge is reached, the Committee produces his own Recommendations (ref. 1 and 2). It also presently acts as a consulting body for the Draft Panel which is now completing the drawing up of the Eurocode n. 8(ref.3), Chapter 3: Specific rules for steel structures.

EXPERIMENTAL AND NUMERICAL ACTIVITY

As far as points (a) and (b) are concerned, many tests have been carried out in the recent past (ref. 4,5 and 6), with the aim to reduce the existing gap of knowledge in this field, as it has been clearly emphasized from the up-to-date state-of-art (ref. 7). It was observed that the experiments on structural components under cyclic loads are carried out everywhere in several research organisms by means of particular testing procedures which usually differ each other. It doesn't allow to strictly compare the various results coming from different laboratoires.

The necessity to unify such experimental procedures seemed to be unquestionable and the Committee decided to work out a proposal for a recommended testing procedure for assessing the behaviour of structural steel elements or connections under cyclic loads (ref.1).

The main problem was to define the fundamental parameters which must be recorded during the experiment for the interpretation of the overall behaviour of the structural components. They have to characterize the degree of ductility, the amount of energy absorption and the deterioration of rigidity and strenght as far as the number of cycles increases.

The hysteretic load versus deflection loops obtained by cyclic tests can be also interpreted by an appropriate mathematical model (ref. 8 and 9), which has been used to calibrate the procedure and to give a unified interpretation of the experimental data by comparing the above-defined behavioural parameters.

The semi-rigid relation-ship of the technological joints calls the attention of its influence on the overall behaviour of framed structures.

The analysis of the behaviour of different structural typologies (point c) represents one of the subjects now in progress within the Committee. The knowledge in this field seems to be far from a satisfactory conclusion, due to the difficulties to introduce in the calculation model the actual behaviour of connections, the geometrical deviation from the theoretical scheme, the mechanical imperfection of members, the lack of regularity of the space structure. So, the research must go in this direction as far as the amount of results can be considered enough to be codified.

On the other hand, it is very pressing the need to cover the design field with provisional rules based upon semplified method (point d). In this perspective, a proposal for the assessment of the so-called q-factor has been recently formulated (ref.10) by taking into account the following aspects:

- local ductility of elements and joints in dissipative zones;
- plastic redistribution capability of the structural scheme;
- influence of the global unstable effects;
- expected collapse mechanism.

DESIGN RULES

Contents. The text of the ECCS Recommendations for Steel Structures in Seismic Zones (ref. 2) is now at its final draft. The main parts of these Recommendations are:

- part. I, "General principles and seismic actions", which gives:
  - requirements and criteria concerning reliability against collapse as well as for limiting damage and unforeseen behaviour;
  - definition of seismic actions by means of a normalized design spectrum;
  - combination rules for seismic actions and design actions.
- part. II, "Rules for structural analysis", which introduces:
  - the definition of structural regularity;
  - the account of non structural elements on the overall behaviour;
  - the suggested calculation methods (direct dynamic analysis, response spectrum modal analysis, static equivalent analysis);
  - the safety verifications from the point of view of no-collapse requirements, limitation of damage and limitation of unforeseen behaviour.
- part. III, "Rules for structural design", which represents the part more closely characterized from the point of view of steel structures and it is strictly correlated to the Eurocode n. 3; it refers to:
  - design criteria for non-dissipative and dissipative structures;
  - materials properties;
  - structural typologies;
  - behaviour factors;
  - local ductility;
  - connections;
  - diaphragms and horizontal bracings;
  - safety checks for the structural elements in the main typologies.

Dissipative structures. They have been classified according to their behaviour under seismic events, by assuming that some of their parts move out from the linear range in order to dissipate energy by means of ductile hysteretical behaviour. They are:

a) Frame structures, consisting of frames which resist the horizontal forces in an essentially bending manner. In these structures the dissipative zones are mainly located near the beam-to-column joints and energy can be dissipated by means of cyclic bending behaviour.

b) Concentric truss bracings, in which horizontal forces are mainly resisted by bars subjected to axial actions. In these structures the dissipative zones are mainly located in the tensile diagonals They are subdivided in:
   - diagonal bracings, in which the horizontal forces can be resisted by the tension diagonals only, neglecting the compression diagonals;
   - V-bracings, in which the horizontal forces can be resisted by considering both tension and compression diagonals.

   The K-bracings cannot be considered as dissipative when the diagonal intersection lies on the column, due to the cooperation of the column to the yielding mechanism.

c) Eccentric truss bracings, in which the horizontal forces are mainly absorbed by axially loaded bars, but the eccentricity of the layout allows the energy dissipation by means of a cyclic bending or shearing behaviour of the beams. They belong to the group of dissipative structures provided that the bending or shear limit strength of the beams or part of them precedes the attainment of the tension or compression limit strength of the other bars.

d) Cantilever structures, which act essentially as beam-columns in which the dissipative zones are mainly located at the base.

e) Braced frame structures, in which horizontal forces are resisted by both frames and bracings acting in the same plane.

f) Structures with reinforced concrete cores or walls, which mainly resist the horizontal forces with dissipative zones located at the base.

g) Mixed structures in steel and reinforced concrete, in which horizontal forces are resisted by both reinforced concrete structures and steel frameworks or truss bracings.

Behaviour factor. For the above-mentioned typological classes of steel dissipative structures, the corresponding q-factors values, which account for energy dissipation capacity and post-elastic resistance, are the following:

- Structures mainly resisting in bending (frame structures, eccentric truss bracings, braced frame structures):

$$q = 5\ \alpha_u/\alpha_1 \leq 8$$

- Concentric truss bracings:
  - . diagonal bracings $\quad q = 4\ \alpha_u/\alpha_1$
  - . V-bracings $\quad q = 2\ \alpha_u/\alpha_1$

- Cantilever structures:

$$q = 3\ \alpha_u/\alpha_1$$

As simplified rule, in regions of low seismicity q = 2 may be adopted without taking into account any further ductility requirement in case of highly regular buildings with frame or truss structures made of rolled sections.

The parameter α_1 and α_u have the following meaning:

α_1 is the multiplier of the horizontal seismic actions, by keeping constant the other design loads, which corresponds to the point where the structure reaches its elastic limit in one section;

α_u is the multiplier of the horizontal seismic actions, by keeping constant the other design loads, which corresponds to the point where the structure reaches the maximum load bearing capacity due to the formation of plastic hinges in the assumed dissipative zones in a sufficient number to transform the structure in to a mechanism or/and due to the presence of some element which becomes unstable.

The above values of q are valid provided that:

- appropriate "regularity" requirements are strictly respected by the building;
- appropriate detailing rules for connections assure sufficient

local ductility;
- appropriate design rules guarantee the formation of a global collapse mechanism.

Assessment of local ductility. Sufficient local ductility is assured by limiting the width-thickness ratio b/t in compressed parts of the cross-sections. In non-dissipative zones and in dissipative zones of structures calculated with $q \leq 2$, the b/t ratios should respect the limit given in the Eurocode n. 3. In dissipative zones of structures calculated with $q > 2$, three ductility classes have been proposed, depending on the chosen values of the q-factor:

class A	$5 \leq q < 6$
class B	$4 \leq q < 5$
class C	$2 < q < 4$

Class A-values are necessary where sufficient rotation capacity of plastic zones is required.

Class B-values must be reached where plastic resistance shall be attained.

Class C-values are necessary where the resistance is limited by the yielding of the extreme fibres.

For $q > 6$, the b/t ratios of class A can be used, provided $N/N_y \leq 0.1$, being N_y the squash load of the member.

Connections. In dissipative zones they shall have sufficient overstrength to allow for yielding of the connected parts. Connections made by means of butt-welds or full-penetration groove welds do not require any particular check. For fillet-weld or bolted connections the resistance of the connection must be 1.2 times the resistance of the connected member, by considering the upper value of its yield strength.

Amplification factors. They have been introduced in dissipative structures in order to avoid the formation of local collapse mechanisms.

In case of framed structures the amplification factor is used in the verification of columns, in order to guarantee that the yielding of beams precedes the yielding of columns, except at the base of the frame.

In case of concentric truss bracings the amplification factor is used in the verification of beams and columns and of diagonal member connections, in order to guarantee that the axial yielding of the tensile diagonals precedes the ultimate strength of their connections as well as the collapse of beams and columns.

In case of eccentric truss bracings the amplification factor is used in the verification of columns and diagonal members, in order to guarantee that the yielding in bending or in shearing of beams precedes the collapse of columns and diagonal members.

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