ECCS Approach for the Design of Steel Structures to Resist Earthquakes

L'approche de la CECM pour la conception de structures métalliques anti-sismiques

Lösungsvorschlag der EKS zum Entwurf von Stahltragwerken unter Erdbebenlast

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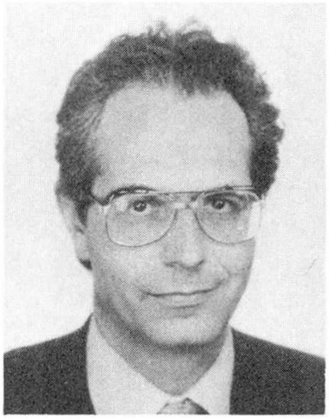

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SUMMARY
In the last three years the European Convention for Constructional Steelwork (ECCS) has been considering the problems arising in the seismic design of steel structures. In this paper the type of approach followed is highlighted, some results are reported, and future trends and research needs are discussed.

RÉSUMÉ
Pendant les derniers trois ans, la Convention Européenne de la Construction Métallique (CECM) a dédié son attention aux problèmes concernant le sécurité des constructions en acier en zone sismique. Dans cet article on présente les méthodes, utilisées et quelques résultats de ce travail. On indique les études futures et les recherches nécessaires pour compléter l'analyse du problème.

ZUSAMMENFASSUNG
Während der letzten drei Jahre hat sich die Europäische Konvention für Stahlbau (EKS) eingehend mit den Problemen des seismischen Entwurfs von Stahlkonstruktionen befasst. In diesem Vortrag wird ein möglicher Lösungsweg durchleuchtet. Es werden einige Ergebnisse aufgezeigt und auf Zukunftstendenzen und notwendige Forschungen hingewiesen.

1. SEISMIC ACTIONS AND CODES FORMAT

Two different criteria may be found in current codes in order to state seismic actions.

a - The response spectrum is correlated to a reduced value of ground acceleration [1,2,3], approximately the 10-15 % of the expected peak value during a strong earthquake. The structure must be checked at the elastic limit but large plastic deformations may occur during a seismic event. If strictly applied, this approach should lead only to the design of structures with an high level of ductility as frames with rigid joints, braced frames and eccentric bracings. It may not cover some typical european structures as truss bracings, widely adopted for low rise apartment hauses, and isolated columns as commonly used in mill buildings.

b - The response spectrum is correlated to a realistic value of ground acceleration [4,5,6]. Thus the response spectrum is transformed into a design spectrum reducing its values by a factor $q > 1$, the so called "structural behaviour factor". The factor q takes into account the elastic plastic behaviour of the structure, the ductility resources of structural elements and their joints. Such an approach may allow the use of structures with limited resources of ductility, provided that greater values of seismic actions are assumed in the design.

The Eurocode n. 8 - Common Unified Rules for Structures in Seismic Regions, recently issued by the Commission of the European Communities [7] states the design spectrum

$$C\ (T) = A\ R(T)/q \tag{1}$$

where:

C(T) is the value of the design spectrum at the period T;

A is the design value of the ground acceleration; depending on the degree of local seismic activity, suggested values of A are between 0.15 and 0.35 g.

R(T) is the value of the normalized design spectrum. It depends on the soil nature and it is stated on the basis of 5% of damping ratio as from fig. 1;

q is the behaviour factor. The Eurocode states: "This parameter takes into account the energy dissipation capacity of a ductile responce. The values of the parameter q depend on the basis of classification of structural system according to ductility levels".

With regard to the above appoach steel structures may be distinguished into two main categories:

- non dissipative structures ($q=1$) designed to withstand seismic actions, and remain in the elastic range.
- dissipative structures ($q>1$) designed in such a way that, during a seismic event, some of their parts (dissipative zones) may move out of the elastic range in order to dissipate energy by mean of a ductile hysteretical behaviour.

Non dissipative structures do not need the ductile behaviour of members and joints to be taken into accont. Dissipative zones of dissipative structures must be designed according to some limitations for joints, slenderness and b/t ratios.

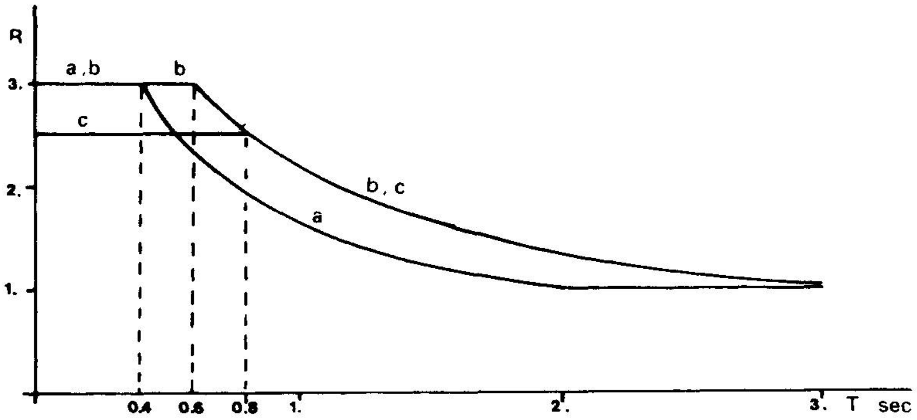

Fig.1 Design spectra as from Eurocode

Tentative values of q factors, as stated by the not jet issued Part III of Eurocode n. 8, are as follows:

- for frames, braced frames, eccentric frames, cross bracings provided that 2nd ordes effects may be disregarded $q=4$ m
- for the abore structural system but with relevant 2nd order effects $q=3$ m
- for cantilever structures $q=2$ m

where m is the ratio between the multiplier of the design loads corresponding to the attainment of the collapse and the multiplier of the design loads corresponding to the attainment of yielding in the most stressed fiber.
Of course for each structural system limitations are given. For example, frames must be designed in order to have dissipative zones in beam elements and not in columns. In eccentric bracings, dissipative zones must be considered in the girders and not in the diagonals. In truss cross bracings tension diagonal members only may be considered active in withstanding lateral forces; their slenderness is limited to $1.5\sqrt{(E/f_y)}$.

2 - THE ASSESSMENT OF BEHAVIOUR FACTOR

From formula (1) the following statement can be derived.
" A correct definition of the values of behaviour factor q"
" is foundamental for a reliable and economic design."

Thus the researchers must join their forces in order to state correct values for q factors.
From a theoretical point of vew the parameter q corresponds to the ratio between the seismic intensity (in the sense of the peak value A) which cause the collapse of the structure and the attainment of the elastic limit state . In other terms let us suppose that a structure attains its elastic limit state when subjected to a seismic event (accelerogram) with a peak value A/q. If we scale the accelerogram up to the peak value A, plastic deformations will occur but their values will not exceed the maximum ones consistent with the integrity of the structure.

Thus the following item are necessary in order to state q :
- design the structure at the elastic limit state for a given seismic action (accelerogram with peak value A_o)
- define the plastic limit deformations at critical sections
- increase the seismic action (A/A_o) and predict the elastic plastic behaviour until the limits of plastic deformations are reached for the value A_u
- define $q=A_u/A_o$

It is self evident that this procedure does not lead to practical results. In fact:
- it is applicable only to a well defined structure
- if the structure is designed and methods for predicting elastic plastic behaviour are available, it is not worth while to assess q. The structure may be checked by non linear analysis.

From a practical point of view the problem of assessing q must be semplified. One way may be as follows:
- state the q values depending on structural systems (frames, bracings, inverted pendulum, ect) together with the local demand of ductility
- provide a ductility greater than the demanded one.

In order to accomplish the first step a method independant from the definition of a limiting plastic deformation is needed. A possible approach is as follows.

Let us imagine that an engineer have to design two anologous structures in two different sites with two different codes having the same format. In the site n.1 the code n.1 states a ground acceleration A_1 and a behaviour factor q_1. Let be A_2, q_2 the values for the same quantities in site n. 2. If $A_1/q_1 = A_2/q_2$ the design spectra (1) are equal for both sites and thus the same identical structure is well suited for both sites. Let be v_d the value of the displacement at the elastic limit state of a meaningful point of the structure.

When a seismic event will occur the behaviour of the two structures will not be the same. If the assumption of the ductility factor theory are accomplished [8,9], the displacements will be $v_{A1} = q_1 v_d$; $v_{A2} = q_2 v_d$.
Thus the following statements hold:
- two structure are identical and have the same design displacement v_d if $A/A_1 = q/q_1$
- if the ductility factor theory is valid then $v_A/v_d = q$

Assume an accelerogram with a peak value A and design the structure at the elastic limit state assuming A_1 and q_1. Let v_d be the value of the displacement of a meaningful point. Increase the value A of the peak value and evaluate the maximum value v_A of the displacement. Three patterns are possible (fig.2) Pattern "a" corresponds to a behaviour in compliance with the results of the ductility factor theory. Pattern "b" shows an unsafe behaviour because everywhere $v_A > q\, v_d$ Pattern "c" presents a first safe range $(v_A < q v_d)$ followed by an unsafe one. The values of $q=v_A/v_d$ for which the ductility factor theory is accomplished may be choosen as q values for the structure and $v_A/v_d = q$ represents the ductility overall demand of the structure. The above method was used for assessing q factors for columns of mill buildings [10].

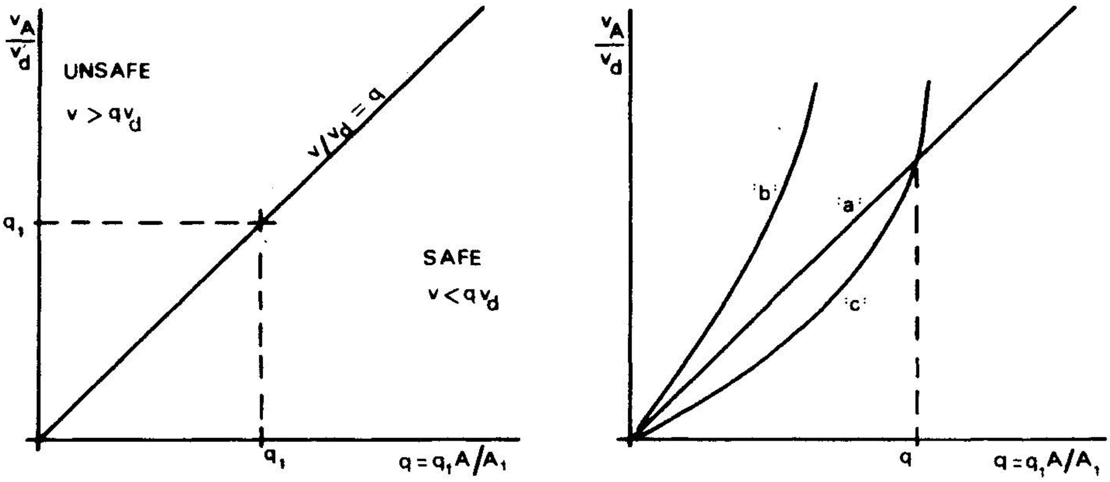

Fig. 2 Determination of the q factor

3 - TESTING PROCEDURES

Numerical models and non linear dynamic analysis are necessary for assessing q values and ductility demand. On the other hand it seems compulsory to perform experimental tests in order to:
- check the correctness of the numerical models
- control the possibility of providing a ductility not less than demanded one.

Shaking table tests are surely the closest to the reality. On the other hand they need very high investments and menagement costs. Thus dynamic tests appear more suited for giving the final proof of the reliability of a structure rather than for appointing a structural system or for comparing different structural solutions.
For the above reasons ECCS pointed its attention to static cycling tests and drafted a recommended procedure [11,12] in order to perform such tests. The major points of this proposal are:
- the choice to impose at each cycle the value of the displacement rather than that of the applied force.
- the definition of various parameters that may characterize the structural behaviour of the speciman (ductility, full ductility, rigidity, maximum load, energy).
- the care of looking at possible deteriorating behaviour imponing three cycles for each value of imposed displacement.
- the criteria for determining the end of the test.

The purpose of this procedure is to standardize the tests in order to produce results that may be compared each other and with the ones of numerical models . As an example in fig.3 are represented the experimental results of a full scale test of a cross bracing .

In fig. 4 the experimental patterns are compared with the results of the numerical simulation of the test by mean of a finite element model [13].

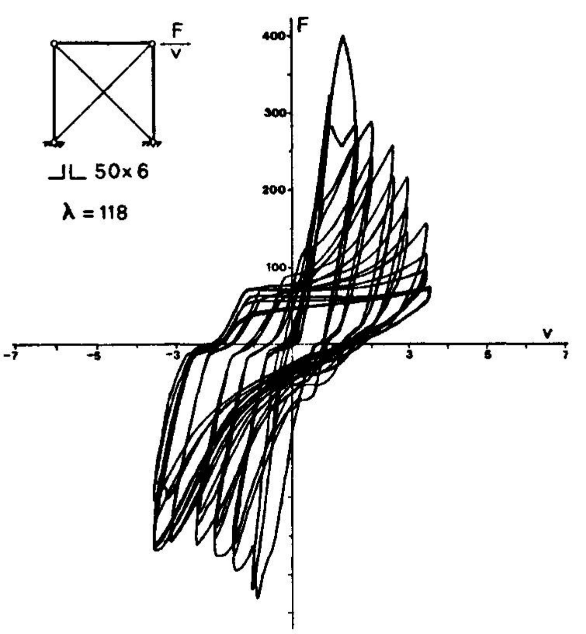

Fig. 3 Experimental test on a cross section truss bracing F (KN) - v (cm)

4 - RESEARCH NEEDS

Both numerical and experimental studies are necessary in order to prove the reliability of most common european steel structures in seismic zones.

a - It is necessary to assess the structure coefficient q for both framed and braced structures with different height (one, four and eight floors are typical for civil buildings). Similar studies for composite structures are also useful as well as for the most common shapes of industrial buildings.

b - Probabilistic studies are necessary in order to state loading combination for industrial buildings with heavy cranes in seismic zones.

c - Expansion joints, if any, must be much larger in aseismic structures. This condition may give some problems in designing long structures for industrial plants and may suggest to avoid expansion joints even if neglecting temperature effects. Non linear studies looking for a good compromise between temperature effects at serviceability limit states and seismic forces at ultimate limit states must be performed.

d - Ductility of structural elements and connections must be experimentally assessed.Width to thickness limiting ratios in order to avoid local instability for elastic and plastic design are well known, if loads are monothonically increasing. It is necessary to state limits to b/t ratios also when cycling loads may occur in order to allow or forbid the use of cold formed profiles for aseismic structures.

e -The semi-rigid joints are developping for their economic convenience. Their suitability in seismic zone is still undemostrated as well as the benefits of slipping in bolted connections. Test on models, subassemblages and full-scale structures are needed.

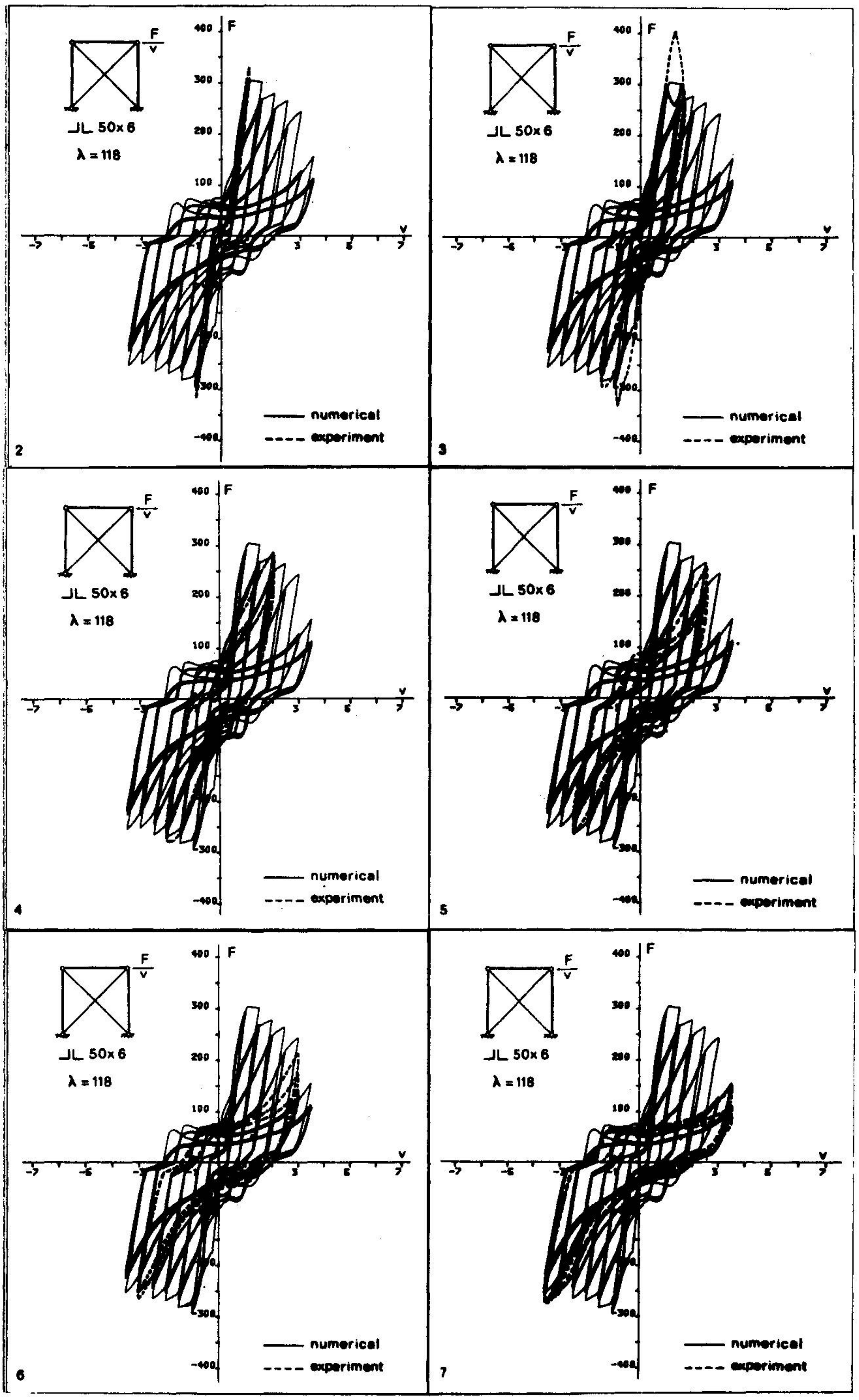

Fig. 4 - Numerical results on the bracing of Fig. 3

At present the efforts of ECCS Working Group WG 1.3, are mainly devoted to points a), d), e). They are mainly supported by researchers of Aachen (D), Liege (B), Milan (I), Napoli (I), Rennes (F), but it is hopeful that in future more and more studies will be performed, in order to make deeper the knowledge of steel structure behaviour in seismic zones.

5 - REFERENCES

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