DEVELOPMENT OF RISK ACCEPTANCE CRITERIA FOR THE DESIGN OF STEEL STRUCTURES

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INTRODUCTION

Structural design codes must deal with the safety issue either implicitly or explicitly. Under the implicit approach used in daily practice the risks relating to a specific project are not quantified, a situation that entails important drawbacks since structural safety decision-making is not based on rational criteria and is therefore subject to possible over-reaction; furthermore, current rules are unsuited to the analysis of innovative technologies and may stifle the implementation of new solutions. With the progressive acknowledgement of the consequences of these shortcomings in the existing legislation, some of the more recent codes have begun to include explicit risk analysis in structural design [1]. However, inasmuch as the regulations presently in force establish only a general framework for explicitly addressing the safety issue, such methods for analysing risk have been virtually ignored in everyday practice to date. There is therefore a need to develop simple methods, models and decision criteria geared to the practical application of risk analysis in design.

1 APPROACH FOR THE DEVELOPMENT OF PRACTICAL TOOLS

The results of risk analysis should be compared to safety requirements when deciding whether the system analyzed is acceptable. The easiest and perhaps the most logical approach is to establish acceptable risk to be at the level of inherent risk set out in existing structural standards, inasmuch as they represent general practice and are regarded to be acceptable by definition. Acceptable risks therefore depend on the degree of reliability implicitly required by such standards, which in turn depends on the level of uncertainty associated with standardized rules.

In the present context, the difficulty lies in the want – in codes of practice – of any explicitly established degree of uncertainty associated with the standardized rules in force for structural design. Moreover, these rules have not been fully calibrated with consistent criteria. Indeed, the reliability of structural members strictly designed to the rules of current standards shows a wide scatter and strongly depends on the relevant design situation, the constituent materials used, the relevant failure mechanism, as well as on the combination of other parameters. Therefore, the target reliability levels defined by modern standards such as the structural Eurocodes [1] are notional and their relation to the level of reliability implicit in such codes is likewise unknown.

In view of the above, the following issues must be addressed to develop tools for the practical application of risk analysis methods in structural engineering, e.g. in the design of steel structures for buildings:

- Determination of the state of uncertainty associated with the rules set out in the existing standards on structural design (section 2).
- Deduction of the level of reliability implicitly required in such standards (section 3).
- Development of mathematical models to estimate the consequences of failure (section 4).
- Determination of the acceptable level of risk associated with structures (section 5).

2 UNCERTAINTIES ASSOCIATED WITH DESIGN RULES

Probabilistic models must be deduced for the structural design variables representing the state of uncertainty associated with the rules laid down in a consistent set of codes addressing basis of design, actions on structures and resistance of structures. Generally speaking, the requirements that should be met by such models are as follows:

- Representation of the physical properties of the respective variable.
- Consistency with probabilistic models laid down in the JCSS Probabilistic Model Code [2].
- Representation of the state of uncertainty associated with the existing design codes.
- Applicability to practical situations, representing relevant uncertainties by means of random variables.

The procedure used for deducing probabilistic models representing the state of uncertainty associated with the design rules for steel structures laid down in the Spanish codes on basis of design [3], actions on structures [4] and resistance of steel structures [5] can be deduced from [6]. Generally speaking, the rules contained in the aforementioned three standards are compatible with the respective structural Eurocodes. For the present purpose one of the main differences can be found in the partial factors to be used for the resistance of cross-sections and members: a value of 1.05 has been adopted in [5] whereas Eurocode 3 [7] recommends a numerical value of 1.00 for buildings.

3 LEVEL OF RELIABILITY ACCORDING TO CURRENT CODES

3.1 Procedure

The procedure adopted to determine the level of reliability implicitly required by a consistent set of codes for structural design consists in the following steps:

1. Definition of the scope of the survey: structural systems to be studied, types of failure to be addressed.
2. Selection of a representative set of structural members.
3. Identification of the most representative failure mechanisms for the members chosen. Formulation of the respective limit state functions.
4. Identification of the probabilistic models for the variables involved in the limit state functions.
5. Determination of the level of reliability implicitly present in the codes analyzed on the basis of both the limit state functions identified (step 3) and the stochastic models determined (step 4).

Determination of the level of reliability implicit in the rules analyzed (step 5), in turn, involves three further steps:

- Design of each of the structural members selected (step 2) in accordance with the consistent set of codes for which the implicit level of reliability is to be determined. Inasmuch as conservative design has a significant effect on the level of reliability, it should be performed strictly so the structural members comply exactly with the structural safety requirement laid down in the codes used.
- Determination of the probability of failure for each of the representative failure mechanisms (step 3), for all the strictly designed members. The probabilistic models used for these intents and purposes are the models for the variables representing the state of uncertainty associated with the rules laid down in the codes analyzed (step 4 and section 2).
- Statistical evaluation and interpretation of findings.

3.2 Work done

The survey involves selecting a series of hypothetical but realistic steel structures with very common characteristics, as shown in the example given in *Fig. 1*. Varying the parameters with the greatest effect on design (use category, beam span, number of storeys, material strength, permanent loads, and so on) within reasonable ranges to cover the vast majority of the cases encountered in practice yields a representative set of building structural members. For structural steel members, this set consists in 144 different roof girders, 270 floor beams and 12168 kinds of columns. Similar numbers are found for the sets of structures made from other constituent materials. Since the

hypothesis adopted is conservative and in light of their wide use in conventional building structures, only statically determinate members are considered.
The limit state functions for cross-sections are deduced from the codes on, respectively, basis of design [3], actions [4] and resistance [5]. Criteria that are irrelevant to member reliability are disregarded. For steel beams, for instance, one limit state function is adopted for bending moments in the mid-span section and one for shear forces in each support section. For columns, in turn, only one limit state function is adopted, based on the compression force at the base.
The strict design of all beams and columns constituting the aforementioned representative set of structural members is carried out automatically with a computer program specifically developed for this purpose. For the calculation of probabilities, code [8] is used, modified for the present work to include a pre- and post-processor that provide for the automatic reading and processing of the parameters obtained in the strict design of structural members in buildings.

3.3 Results

The results of the present study are given in terms of probabilities of failure, p_f, referring to the entire service life of the structure, namely 50 years in accordance with accepted practice, as established in [1]. *Table 1* summarizes the mean values, μ_{pf}, and the coefficients of variation, v_{pf}, obtained for the probability of failure, p_f, for all structural steel roof beams, floor beams and columns, strictly designed to the rules laid down in Spanish structural codes. The mean value for the probability of roof beam failure was perceptibly – around 20- and 80-fold, respectively – higher than the mean values found for floor beam and column failure. The coefficients of variation reflect a wide scatter of the results. While the coefficient of variation was around 100% for structural steel beams, for columns it rocketed to 270%. Given that rules laid down in the codes whose implicit level of reliability was determined in the present study are not fully calibrated in accordance with consistent methods of structural reliability, the dispersion of results observed is not surprising.

Table 1. Statistical evaluation of the failure probability, p_f, obtained for structural steel members strictly designed to the Spanish building code (Reference period: 50 years)

Structural steel members	Nº of members analysed	Failure mechanisms per member	Nº of results	Probability of failure	
				Mean value μ_{p_f}	Coefficient of variation v_{p_f}
Roof beams	144	2	288	$9.4 \cdot 10^{-4}$	0.96
Floor beams	270	2	540	$4.6 \cdot 10^{-5}$	1.16
Columns	12168	1	12168	$1.2 \cdot 10^{-5}$	2.72

4 CONSEQUENCE MODELS

Of all the possible adverse – direct or indirect – consequences of undesired events, including casualties, environmental damage and financial loss, in (civil) engineering, risks to personal integrity generally prevail, due to both ethical and legal considerations. Consequently, modelling the consequences of structural failure may be simplified by considering only the loss of human life. Pursuant to European standard [1], which distinguishes among three categories of consequences, different models were developed in this study for consequence categories CC2 (residential and office buildings) and CC3 (densely occupied buildings).
The consequences of collapse of a building in a given scenario, j, are expressed in terms of the number of fatalities expected, N_j, as a function of the area affected by the collapse, $A_{col,j}$. The analysis of the relevant data gathered on more than 300 building collapses yields the following model for the number of fatalities expected in the total or partial collapse of a category CC2 building structure:

$$N_j = \left(0.27\left[A_{col,j}\right]^{0.5} - 1\right) \geq 0 \tag{1}$$

Similarly, the model obtained for the number of fatalities caused by the structural collapse of a category CC3 building is:

$$N_j = \left(0.59\left[A_{col,j}\right]^{0.56} - 1\right) \geq 0 \tag{2}$$

Fig. 1. Typical building structure with steel members during erection

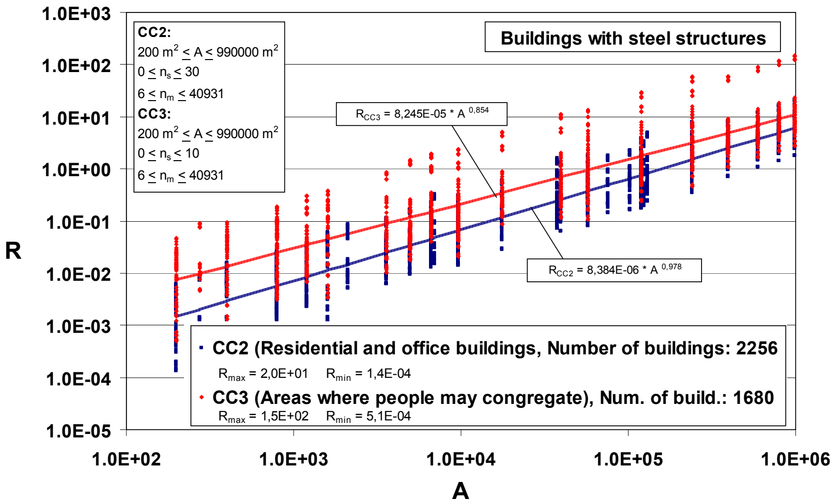

Fig. 2. Collective risks to persons, R, versus total net room area, A, for the buildings analysed with steel structures (Reference period: 50 years)

5 ACCEPTANCE CRITERIA

5.1 Overview

According to the approach adopted in this paper, acceptable risk is equivalent to the risk associated with the structures that are strictly compliant with the safety requirements set out in the applied standards. A procedure was therefore established to determine the risks implicitly accepted by Spanish standards governing the design of building structures. In keeping with the scope of the present paper, this analysis focused on partial and total structural collapse. The survey involved selecting a series of hypothetical but realistic steel structures (*Fig. 1*), constituted by members with common characteristics such as those considered for the establishment of the required reliability level (section 3), strictly designed to the Spanish building code. The structures were regarded to comprise statically determinate members, a conservative hypothesis, for the probability of failure is generally higher in such members than in comparable statically indeterminate members subjected to the same loads. A representative series of building structures was obtained by varying all the relevant parameters, such as geometry (building length, width and height; ratio between exterior dimensions; number of storeys; length of beam spans), use category (residential; office; congregation of people), permanent loads, snow load, steel strength and the like. In all, the structural analysis covered 2256 structural steel residential or office buildings corresponding to European standard [1] consequence category CC2, and 1680 buildings with areas where people may congregate (consequence category CC3), also with steel structures. This set included buildings with total net room areas ranging from 200 to 990000 m^2, up to 10 storeys – excluding ground storey and roof – in category CC3 and up to 30 storeys in category CC2. In both categories of buildings, the number of main structural members ranged from 6 to 40931. Similar sets of representative structures made from other constituent materials have been established and analysed [9]. However, in the present paper only the results obtained for steel structures are discussed.

Risk is a measure for the magnitude of a hazard. The two constituents of risk are the probability of a damaging event and the average expectation of the damage should this event take place. In the present study, this expectation is expressed in terms of casualties. The simplest function relating the two constituents of risk is the product of both quantities. Since probabilities are dimensionless, risk has the dimension of the damage quantity itself and is related to a particular time interval, e.g. the number of fatalities per 50 years, as in the present case.

A set of circumstances, occurring together in space and time and having the potential to cause an undesired event, is called a hazard scenario. In the actual context, with building structures constituted by simply supported members, each failure mechanism of each member may be considered as one statistically independent scenario. The collective risks to persons, R, associated with a given building structure, are obtained as the sum of the collective risks, R_j, for all relevant scenarios, j (*Eq. (3)*). The probability of occurrence, p_j, of a given scenario, j, being equivalent to the probability of failure of the corresponding structural member for a particular failure mechanism, R_j is the product of this probability and the expected number of fatalities, N_j, in the same scenario, j:

$$R = \sum_{j=1}^{n} R_j \approx \sum_{j=1}^{n} p_j \cdot N_j \tag{3}$$

The strict design of beams and columns constituting the building structures of the aforementioned set, as well as the calculation of the probability of structural member failure have been performed as outlined in section 3. The consequences of partial or total collapse of a building due to the failure of structural members in a given scenario are estimated by using the models from section 4, depending on the area affected by the collapse and the category of the building. On the basis of these probabilities of failure and the expected number of fatalities in the relevant scenarios, the collective risks to persons, R, can be established for each building structure. The results obtained for buildings belonging to consequence categories CC2 and CC3, respectively, constitute the basis for the establishment of the proposed acceptance criteria for structure-related risks.

5.2 Results

Since all the members of all the buildings analysed complied exactly with the structural safety requirements laid down in Spanish design codes, the risk associated with each structure was acceptable within the framework of these codes. The differences between consequence categories CC2 and CC3 consisted essentially in a higher number of expected fatalities for CC3 buildings. In addition to this anticipated and reasonable result, the figures on collective risks to persons associated with the structures analysed were observed to be widely dispersed. Such dispersion was due, among others, to a significant problem in connection with the procedure used to deduce the acceptable level of risk. Risks associated with structures designed in strict accordance with the existing legislation rise with the number of failure mechanisms, structural members and combinations of actions and influences that may lead to structural failure, as well as the size of the net room area of the building (scale effect). *Fig. 2* shows the collective risks to persons, R, versus total net room area, A, for each of the structural steel buildings analysed. The scale effect is particularly visible here, a finding that rules out the possibility of defining an acceptable level of structural risk in absolute terms.

6 FINAL PROPOSAL

Despite the intrinsic problems in the obtained results, if suitably adapted and used they can yield valuable information. Adaptation consists essentially in standardizing and adequately grouping the results for interpretation in comparative terms [9]. The scale effect described above, for instance, can be eliminated by standardizing to the net room area of buildings. *Fig. 3* shows the collective risks to persons, R, standardized for total net room area, versus total net room area, A, for each of the structural steel buildings analysed. In this case the scatter observed in the results was due, among others, to the lack of calibration of the rules for the design of structural members laid down in the existing standards.

The graph in *Fig. 3* can be used to define a possible acceptance criterion for individual risks to persons. It represents the number of fatalities per unit of net room area over a 50-year reference period that would be acceptable pursuant to current legislation. Moreover, the number of people per unit of area is a variable that depends on building use, and to a certain extent relates collective and individual risks. For all these reasons, the acceptance criterion for individual risks to persons can be expressed in terms of the mean value, or of a given fractile, of the number of fatalities per unit of net room area and time.

According to the procedures adopted for risk analysis, an acceptance criterion must likewise be adopted for collective risks. For intents and purposes of the present survey, the acceptable level of collective risks may be deduced from the results given in *Fig. 2*. Here, the graph shows the collective risks to persons versus total net room area, for all the buildings analysed with structural steel members. The acceptance criterion could be expressed in terms of the expected value, or a given fractile, of the collective risks as a function of the total net room area of the building.

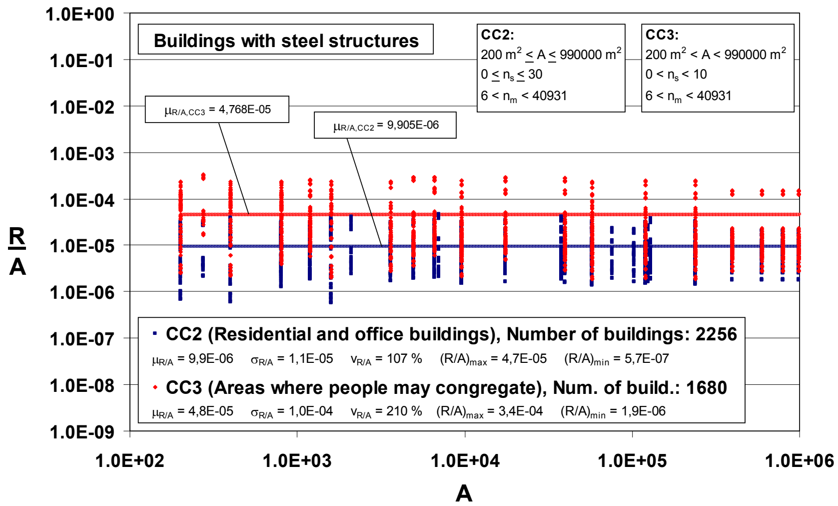

Fig. 3. Collective risks to persons, R, standardized to the total net room area, A, versus A, for the buildings analysed with steel structures (Reference period: 50 years)

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