Special Workshop on Risk Acceptance and Risk Communication
March 26-27, 2007, Stanford University

Structural safety requirements based on notional risks associated with current practice

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Abstract

The treatment of technological risks is a major challenge for modern societies. And indeed, risk is one of the considerations inexcusably addressed in the design of technical facilities in general, and of buildings and civil engineering structures in particular. *How safe is safe enough?* is a key question in this context. Structural design codes must therefore deal with the safety issue either implicitly or explicitly. Under the implicit approach presently in place the risks relating to a specific project are not quantified; the realization of the significant drawbacks this entails has grown in recent years. The development of simple tools and methods geared to the practical application of risk analysis in building design is therefore stressed in this paper. The proposal put forward would establish a rational basis for decision-making in connection with the level of structural reliability required, along with acceptance criteria for structure-related risks.

1. Introduction

The definition of safety and service performance requirements to be met by the structures of both buildings and public works has traditionally been a task entrusted to professionals engaging in structural design and construction. For practical intents and purposes, this self-regulation has materialized in standards for structural design, construction and, more recently, inspection and maintenance. By definition, due application of the existing standards ensures that the structures built meet the aforementioned performance requirements. The acceptance criteria contained in most existing structural standards are not derived from consistent or rational criteria since the requirements are based exclusively on past experience. In practice, this leads to a series of shortcomings:

- Objectively known risks that should be treated as such might not be taken into account in the analyses.
- The probability of failure of structures designed and built according to existing rules is virtually unknown.
- The numerical values of the probability of failure of the structures designed and built to such rules generally range over very wide intervals, of the order of some magnitudes.

Similar situations are observed in other fields of engineering and public health. The lack of consistency and rationality in acceptance criteria for engineering and natural risks leads to the inefficient deployment of the resources available for risk reduction.

2. Risk analysis procedure

In the generic risk analysis process inspired by the Australian and New Zealander code on risk management (AS/NZS 4360, 1999), a distinction can be drawn between qualitative and quantitative analysis. In the first stage of such a process, the hazards involved in the technical system analyzed should be identified, along with any hazard scenario (set of circumstances that may occur simultaneously in a technical system, with the potential for causing events with undesired consequences). This stage is crucial, since the failure to identify and therefore include hazards or scenarios in the rest of the process will necessarily introduce a bias in the decisions adopted. Despite the importance of qualitative analysis, however, the present paper focuses on quantitative risk analysis, which consists in estimating the likelihood of the occurrence of previously identified scenarios and their consequences, for example in terms of loss of human life, if they do materialize. Once these two parameters are determined for all relevant scenarios, the risk associated with the system analyzed can be estimated and compared to the applicable acceptance criterion. Normally, two acceptance criteria are established, one for individual and the other for collective risks to persons, the more restrictive of the two being the determinant. In the event of non-compliance, appropriate measures must be taken to reduce risk to acceptable levels.

3. 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 of any explicitly established probabilistic models on which the standardized rules for structural design are based. The degree of uncertainty associated with the standards in force is therefore unknown. Moreover, since the rules in the current standards have not been calibrated with consistent criteria, the level of reliability required according to such standards 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:

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

4. 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 (JCSS, 2001) probabilistic models.
- Representation of the state of uncertainty associated with the existing codes.
- Applicability to practical situations, representing relevant uncertainties by means of random variables.

The procedure used for deducing these probabilistic models is as follows:

1. Identification of the most representative failure mechanisms. Formulation of such mechanisms in terms of limit state functions (LSF).
2. Definition of the verification format used in the consistent set of codes analyzed.
3. Identification of probabilistic models for the variables found to be relevant to the failure mechanisms considered.
4. Determination of the design values for the relevant variables using the standardized FORM method.
5. Determination of the partial factors from the design and representative values of the variables. For the hypotheses adopted (required reliability index, standardized values for sensitivity factors and so on), the partial factors depend on:

   - The failure mechanism considered and the limit state function.
   - The partial factor format adopted.
   - The probabilistic models for the variables.
   - The definition of the representative values for the variables.

6. Definition of the target function for fitting the models:

$$_{\min}W(\mu,\sigma)=\sum_{j=1}^{L}W_{Y,j}\left(\gamma_{Y,j}(\mu,\sigma)-\gamma_{Y,t}\right)^{2} \tag{1}$$

   Where $\gamma_{Y,j}(\mu,\sigma)$ represents the partial factor taking into account the uncertainties of Y with respect to the failure mechanism j; $\gamma_{Y,t}$ is the target value of the partial factor according to the applied codes; and $W_{Y,j}$ is a weighting factor.

7. Determination of optimum probabilistic models for the target function established.
8. Verification of the results in light of prior information and practical considerations.

The models representing the state of uncertainty associated with the design rules for reinforced concrete structures laid down in the codes on basis of design (CTE DB-SE, 2003), actions on structures (CTE DB-SE-AE, 2003) and design of concrete structures (EHE, 1998) are described in (Tanner and Lara, 2005). Generally speaking, the rules contained in these three standards are compatible with the respective structural Eurocodes.

5. Level of reliability according to current codes

5.1. 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 implicit in the codes analyzed on the basis of both the limit state functions identified in step 3 and the stochastic models determined in 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 4).
- Statistical evaluation and interpretation of findings.

5.2. Work done

The survey involves selecting a series of hypothetical but realistic concrete structures with very common characteristics, as shown in the example given in Figure 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 reinforced concrete members, this set consists in 240 different roof girders, 450 floor beams and 22320 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 basis of design, actions and resistance. Criteria that are irrelevant to member reliability are disregarded. For reinforced concrete beams, for instance, one limit state function is adopted for bending moments in the mid-span section and two for shear forces in each support section: one for the compression forces in the diagonal strut and the other for the tensile forces in the stirrup reinforcement. 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 (VaP 2.0, 2004) 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.

Figure 1: Typical building structure with reinforced concrete members.

5.3. Results

The results of the present study are given graphically in terms of probability of failure, p_f, as a function of the ratio between variable and total loads, ν. Figure 2 shows the findings for the 240 reinforced concrete roof beams. The results obtained for the three failure mechanisms analyzed – mid-span bending failure, shear failure of the support section due to, respectively, yielding of the stirrup reinforcement and concrete crushing in the diagonal compression strut – are shown separately in the figure. In addition to the graphic representation of all the results obtained, the following numerical values are given for the probability of failure, p_f, for each of the roof beam failure mechanisms:

- Mean value, μ_{pf}.
- Standard deviation, σ_{pf}.
- Coefficient of variation, v_{pf}.
- Maximum probability of failure, $p_{f,max}$.
- Minimum probability of failure, $p_{f,min}$.

The above probabilities of failure refer to the entire service life of the structure, namely 50 years in accordance with accepted practice, as established in (EN 1990, 2002), for instance.

The values found for the probability of failure of reinforced concrete roof beams strictly designed to the rules laid down in Spanish structural codes range over a fairly wide interval. Depending on the failure mechanism analyzed, the coefficient of variation for these results amounts to values fluctuating from 71.1% (shear failure due to concrete crushing in the diagonal compression strut) to 129% (bending failure). The coefficient of variation for all the results obtained for the roof beams came to 122.2%.

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 reinforced concrete roof beams, floor beams and columns. The mean values for the probabilities of roof and floor beam failure were practically identical but perceptibly – around 110-fold – higher than the mean value found for column failure. The coefficients of variation reflect the aforementioned wide scatter of the results. While the coefficient of variation was over 100% for reinforced concrete beams, for columns it rocketed to 300%. Given that rules laid down in the codes whose implicit level of reliability was determined in the present study are not calibrated in accordance with consistent methods of structural reliability, the dispersion of results observed is not surprising.

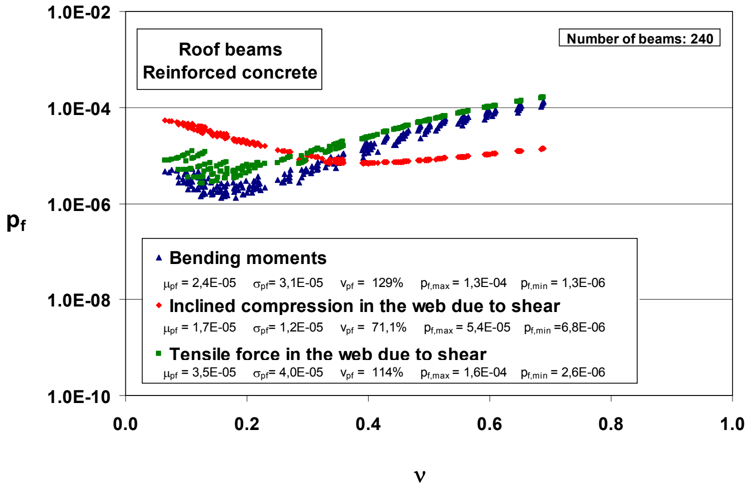

Figure 2: Probability of failure, p_f,, vs. ratio of variable to total loads, ν, for three failure mechanisms for each of the 240 roof beams analyzed (Reference period: 50 years) .

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

Reinforced concrete 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	240	3	720	$2.5 \cdot 10^{-5}$	1.22
Floor beams	450	3	1350	$2.4 \cdot 10^{-5}$	1.04
Columns	22320	1	22320	$2.2 \cdot 10^{-7}$	3.08

6. Consequence models

6.1. Overview

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. This means that fatal accident rates are not only regarded to be representative of potential casualties in general (fatalities and injuries of varying severity), but more broadly of the risk associated with a given technical system, such as a building structure. A model is therefore needed to estimate the number of mortalities caused by a structural collapse.

Pursuant to European standard (EN 1990, 2002), 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 probability of the presence of persons in the event of CC1 collapse, which refers to buildings not usually occupied by people (farm buildings, greenhouses, warehouses), was regarded to be sufficiently small to disregard this category for the intents and purposes of modelling.

Only the scenarios induced by actions, effects or circumstances referred to as *persistent situations* under normal conditions of use in standards (CTE DB-SE, 2003) and (EN 1990, 2002) were addressed to determine the parameters defining consequence models for the partial or total collapse of building structures. In other words, so-called *accidental situations* were not considered.

All the relevant information available on 301 buildings that had collapsed was compiled to develop the consequence models. The accidents analyzed were restricted to Western countries with construction systems comparable to those in place in Spain. In many cases, the information compiled was sparse or scantly reliable, having been drawn, for instance, from news items. The resulting database was modified and completed, as far as possible, with information from other sources, such as National Statistics Institute databases (for the usable area of buildings or the number of users per unit of area, for instance). Reducing the original database to cases with sufficiently significant information, including those completed with data taken from the above sources, yielded a new base with 109 well documented cases, classified under the various consequence categories. A series of statistical analyses of these data sufficed to deduce consequence models for categories CC2 and CC3 mentioned above (Figure 3).

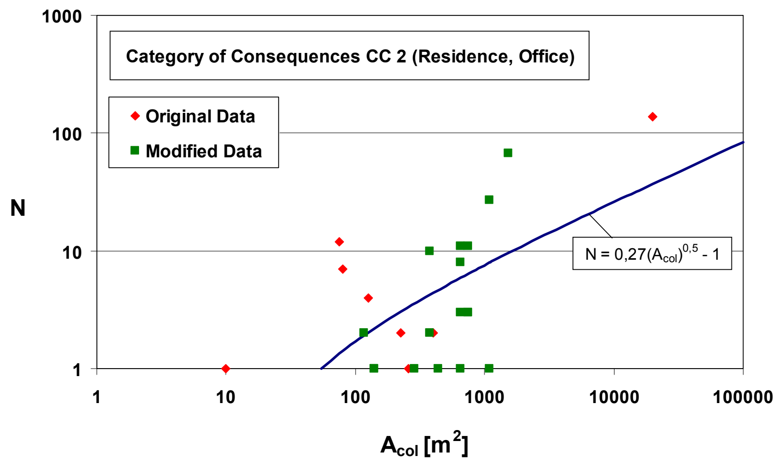

Figure 3: Number of fatalities expected, N_j , per area affected by the collapse, $A_{col,j}$, for a given failure scenario, j, in residential and office buildings.

6.2. Results

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 data shown in Figure 3 yields the following model for the number of fatalities expected in the total or partial collapse of a category CC2 building structure:

$$N_j = \left(0{,}27\left[A_{col,j}\right]^{0{,}5} - 1\right) \geq 0 \tag{2}$$

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

$$N_j = \left(0.59\left[A_{col,j}\right]^{0.56} - 1\right) \geq 0 \tag{3}$$

7. Acceptance criteria

7.1. 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 concrete structures (Figure 1), constituted by members with common characteristics such as those considered for the establishment of the required reliability level (section 5), 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, concrete strength and the like. In all, the structural analysis covered 2256 reinforced concrete residential or office buildings corresponding to European standard (EN 1990, 2002) consequence category CC2, and 1680 buildings with areas where people may congregate (consequence category CC3), also with reinforced concrete 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 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. However, in the present paper only the results obtained for reinforced concrete structures are discussed.

The strict design of beams and columns constituting these building structures, as well as the calculation of the probability of structural member failure have been carried out as mentioned in section 5. 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 6, 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, associated to the relevant scenarios, the risk profile for each building can be established. Figure 4 shows the upper and lower envelopes of the risk profiles for all the buildings analyzed with reinforced concrete structures. The risk profiles and their envelopes are shown in terms of the cumulative probability of failure over a reference period of 50 years, P_f, and the expected number of victims, N. A distinction was drawn between buildings for residential and office use on the one hand (consequence category CC2) and buildings with areas where people may congregate (consequence category CC3) on the other. The area under a risk profile for a given structure corresponds to the collective risks to persons, R, associated with the structure. These risk profiles constitute the basis for the establishment of the proposed acceptance criteria for structure-related risks.

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). Figure 5 shows the collective risks to persons, *R*, versus total net room area, *A*, for each of the reinforced concrete 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.

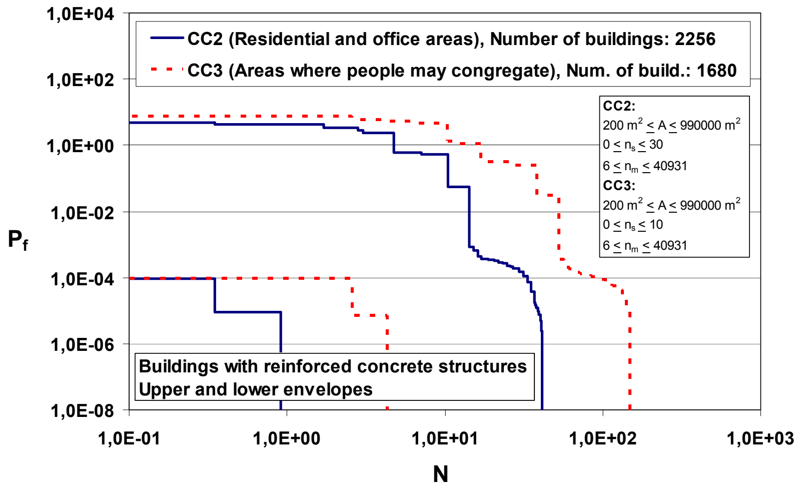

Figure 4: Upper and lower envelopes of the risk profiles for the buildings analysed with reinforced concrete structures (A: total net room area; n_s: number of storeys; n_m: number of structural members).

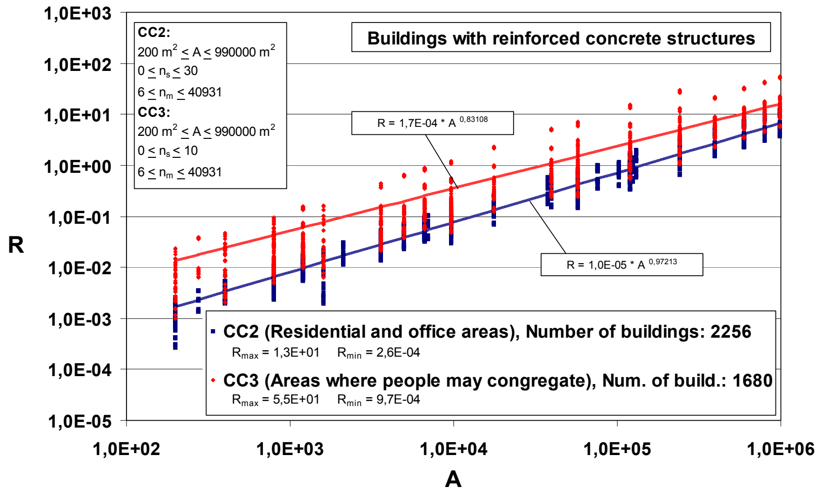

Figure 5: Collective risks to persons, R, versus total net room area, A, for the buildings analysed with reinforced concrete structures (Reference period: 50 years).

7.2. Proposal

Despite the well-known intrinsic problems in F-N plots, if suitably adapted and used they can yield valuable results. Adaptation consists essentially in standardizing and adequately grouping the results for interpretation in comparative terms (Tanner, 2006). The scale effect described above, for instance, can be eliminated by standardizing to the net room area of buildings. Figure 6 shows the collective risks to persons, R, standardized for total net room area, versus total net room area, A, for each of the reinforced concrete 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 Figure 6 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.

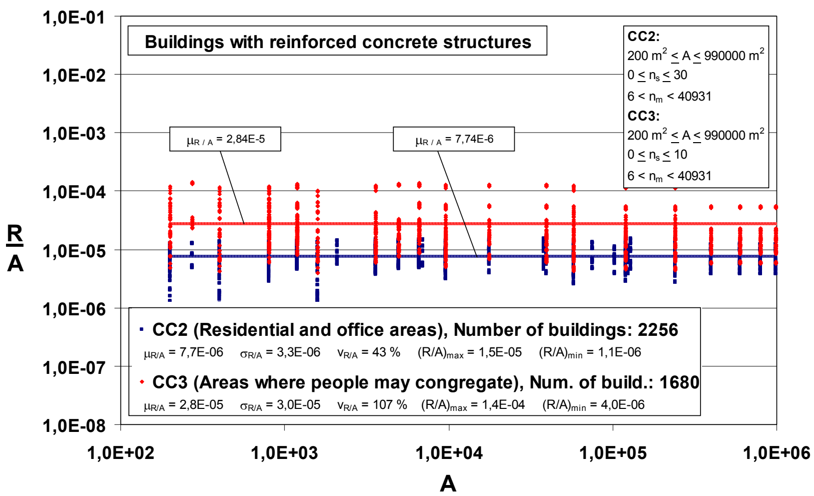

Figure 6: Collective risks to persons, R, standardized to the total net room area, A, versus A, for the buildings analysed with reinforced concrete structures (Reference period: 50 years).

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 Figure 5. Here, the graph shows the collective risks to persons versus total net room area, for all the buildings analysed with reinforced concrete 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.

8. Final remarks

Explicit risk analysis methods contribute to fostering and justifying innovative solutions and optimizing certain structures. Moreover, a higher degree of freedom reached by means of the implementation of performance-based design rules likewise entails greater responsibility since, at least apparently, engineers applying a regulatory document with prescriptive rules delegate part of their responsibility to standard drafting committees or standardization bodies. This consideration led (Jiménez Losada, 2005) to study how the application of explicit risk analysis methods might impact engineers' penal liability. Based on the characteristics of today's society, trends in modern penal law, the general theory of law and case law to date, she deduced, despite the difficulties intrinsic in the subject, that any rise in the number of convictions involving structural engineers is highly unlikely to depend on whether the legislation applied is prescriptive or performance-based.

Acknowledgements

The present study was conducted in the framework of research project FOM-741 funded by the Ministry of Internal Development. Elena Jiménez Losada, lawyer, Andrés Jiménez, mathematician, and Ramón Montis, Carlos Lara and Ramon Hingorani, civil engineers, participated in the project.

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