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# Evaluation method for safety conditions of buildings during the structural repair of columns

# *Método para avaliação da segurança de edificios durante a recuperação estrutural dos pilares*


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## Abstract

The structures deteriorate under the action of environmental hazards and other factors and require interventions which can vary from a simple superficial repair to a more complex reinforcement. The safety analysis in existing buildings differs in several aspects of that established in the project, mainly because the parameters generically adopted by the author of the project can be now studied by an investigation on-site at the moment of intervention. This work analyzes the safety conditions in which the services of structural repair of columns are made, and it presents a methodology that takes into account the reduction of uncertainties related to resistance and load parameters. The structure of the building is modeled in calculation software in order to obtain a more compatible stress with the reduced analyzed period. The methodology indicates the adjustment of the safety factors together with a global factor of safety for columns that allows the determination of a strategy for repairs to be performed, preserving a probability of failure coherent with the existence of the structure. As a result of the proposed methodology, the part of the concrete and steel sections that will be possible to be removed from the columns in a structural repair of a building due to a process of corrosion that will affect the reinforcement structures or deteriorate the concrete. The methodology proposed was applied to three buildings and one of them is shown in this paper. The structural elements of application were the columns of the garage floor of a 25 floor building with a simulation of a structural repair due to a process of initial corrosion of the reinforcement structures. After the analysis, it was possible to define the procedure to be adopted for each column.

***Keywords***: *structural safety; column repair; assessment of structures; safety factors.*

## Resumo

As estruturas se deterioram pela ação das intempéries e de outros fatores requerendo intervenções que variam de um reparo superficial a um reforço mais complexo. A análise da segurança em edifícios existentes difere em vários aspectos daquela estabelecida em projeto, principalmente porque os parâmetros adotados genericamente pelos projetistas poderão ser mais bem definidos mediante uma investigação in loco no momento da intervenção. Este trabalho aborda as condições de segurança em que são realizados os serviços de recuperação estrutural de pilares e apresenta uma metodologia que leva em conta a redução de incertezas relacionadas aos parâmetros de resistência e de solicitação. A estrutura do edifício é modelada em programas de cálculo, buscando-se obter esforços mais compatíveis com o reduzido período analisado. A metodologia indica como ajustar os coeficientes de ponderação e a obter um coeficiente global de segurança para pilares, que permite traçar a estratégia de execução do reparo, mantendo uma probabilidade de falha coerente com a existência da estrutura. Como resultado da metodologia proposta, se obtém a parte da seção de concreto e aço que será, temporariamente, possível retirar do pilar, em um edifício em recuperação estrutural devido a um processo de corrosão das armaduras ou deterioração do concreto. A metodologia aqui proposta foi aplicada em três edifícios sendo que um deles encontra-se relatado neste trabalho. Os elementos estruturais objeto da aplicação foram os pilares da garagem de um edifício de 25 pavimentos com simulação de recuperação estrutural devido a um processo de corrosão inicial das armaduras. Após a análise foi possível definir o tipo de procedimento que seria adotado para cada pilar.

***Palavras-chave:*** *segurança nas estruturas; recuperação de pilares; avaliação estrutural; coeficientes de ponderação.*

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## 1. Introduction

The structures deteriorate under the action of environmental hazards and other factors which require interventions that can vary from a simple superficial repair to a more complex reinforcement to reach the projected service life. The safety analysis in existent buildings differs in several aspects from those established in the project, mainly because the parameters generically adopted by the author of the project can be now studied by an investigation on-site at the moment of intervention.

This work presents a methodology for the safety evaluation of columns of existing buildings, during the structural repair, taking into account the reduction of uncertainties related to resistance and load effect parameters, starting from adjustments of the design criteria, considering the two basic concepts of the philosophy semi-probabilistic of safety in the structures: a) the state limits format and b) application of safety partial factors.

The necessary considerations to obtain the resistances and load effects in the sections of the columns according to updated values will be made in this paper, such as, project values modified by inspection results, on-site tests, and studies of adjusted live loads for the reduced period that corresponds to the structural repair, which reduces the relative uncertainties of the evaluated structure. The methodology still allows a more flexible adoption of the consideration coefficients to be used, due to further knowledge of the influential variables in safety.

The evaluation efforts, coming from the updated parameters of the building, will be generated by commercial calculation software. Therefore, it is possible to utilize any program that accomplishes a more sophisticated structural analysis, such as, a three-dimensional frame, considerations of second order global effects, out of plumb effects, etc.

At the end of this procedure there will be a resistance closer to the real value in the sections of each column under a process of repair, as well as the more probable efforts that will appear in the short period of repair of those structural elements. Comparing the active loads and the resistances at the moment of intervention, it is possible to draw a plan for the retreat of the deteriorated concrete maintaining an acceptable level of safety for the pieces under study. In the same way, it will be possible to identify the pieces that will need shoring during the repair works.

In the structural evaluation, three situations regarding the available documentation are found in general:

**a)** buildings with complete construction plans, technical specifications, material certification (concrete and steel control) etc;

**b)** structural project only;

**c)** Intermediate situation with partial documentation.

In relation to each one of these situations, the necessary tests and studies should be programmed to obtain the information that allows for a better understanding of the building structure. However, there are extreme cases in which there is not any construction record available, besides the structural projects. Such a situation is not encountered in this work, because, in such cases, the expert involved should define a particular procedure, with the accomplishment of a detailed inspection to supply the complete absence of data regarding the structure.

The methodology here proposed was applied to three buildings and one of them is shown in this work. The 25 floor building had it construction paralyzed and the structure was exposed during 10 years which induced the steel corrosion of several structural elements, in which some were columns. The elements analyzed were the columns in the garage floor which underwent a simulation of structural repair, because it was not in service yet, due to a process of initial steel corrosion. After the analysis, it was possible to define the procedure type that would be adopted for each column. The repair of the columns was accomplished simultaneously with the execution of the superior part of the structure. However, it was not necessary to consider the results of the analysis because the active load was inferior to the one of the project considering that the building was under construction.

In the other building on which the methodology was applied, which has been in service for more than 20 years, the repair of the columns of the garage was executed on the building in use. In this building it was more interesting to demonstrate the validity of the methodology, though, because it indicated the need for shoring in 4 columns, due to contractual reasons, it won't be shown herein. For the same reasons, the last building in service, on which the methodology was successfully applied and the repair was accomplished, it will not be shown here either.

## 2. Considerations on Structural Evaluation

The structural evaluations, according to Ellingwood (1996), are determined in several circumstances, such as: change of occupation of buildings; concerns with materials or defective constructive methods; mistakes found when comparing the project and the building itself after it was occupied; structural deterioration due the normal use or environmental conditions; structural damages after extreme events and users' complaints due to the service conditions. One of the characteristics that differs the safety evaluation of existent buildings from that established in design, is the possibility of reduction of uncertainties in relation to the inherent variability of the parameters involved in the load-effect/resistance mechanisms (ACHE, 2003).

The project guidelines defined by the Brazilian Code ABNT NBR 6118:2003, or other design codes, are not applied directly to the evaluation of structures, because of the distinct approach of uncertainties. According to Val & Stewart (2002), the design uncertainties appear from the previous establishment of load parameters and resistance for a "generic" structure that has not been built yet. Such uncertainties represent the variability found in many structures, mainly due to the quality of the materials, construction practice, manpower and the variability of the live loads in time, etc. Thus, the design rules should be conservative to contemplate a variety of situations.

A particular existent structure is evaluated and it can be inspected and tested, which significantly reduces the uncertainties that were considered in design (COST 345, 2004; MELCHERS, 2001). Although the inspection and the tests introduce mistakes and doubts regarding the measured values, for the simple fact that the structure presents a relative quality in the materials, as well as in its execution, a reduction can be expected in its variability when compared to the "generic" structure. This should be taken into account in the safety estimate within a certain period. Besides, because of the structural repair demands, a period of very reduced time (one to three months)

in relation to the service life of the residential building (in general 50 years), it is possible to say that the probable values of occurrence for the variable loads (live loads and winds, mainly) will also have a significant reduction from those proposed in the design codes. This fact will be considered in the safety analysis.
Several works were developed regarding the attempt to establish an acceptance criterion for structures of existing buildings. The majority of the proposals is related to the adjustment of the guidelines of design codes (ALLEN, 1991; VAL & STEWART, 2002), however, a great progress is observed in the research and application for the bridge structures and other road structures (ACHE, 2003; COST 345, 2004).
The steps in the evaluation process do not present great differences among the several researchers. Melchers (2001) presents a typical pattern used for the evaluation process, currently used:

- on-site inspection;
- collection of data and information;
- application of formal outlines for evaluation;
- presentation of results;
- decision.

To evaluate the results of inspection and to judge if the structure is safe, reliability levels should be established, such as those used for the design conditions (ACHE, 2003). In the evaluation, there is a lack of data in the long run for structures submitted to the process, mainly in the case of repair or reinforcements, which constitutes another difficulty for the elaboration of a normative code. Aiming at a better approach of the current situation of constructed buildings, it is still necessary to search for the reduction of the conservatism in the treatment of particular parameters of evaluation. This fact should be lessened with the study, the development and fixation of probabilities of flaws that are coherence with the real conditions of each structure appraised individually, according to general results found in the inspection (MELCHERS, 2001).
The acceptable probability of failure for an existing structure is an arduous task as well as in works of calibration of codes. COST 345 (2004) presented four levels for specifying target reliability that can represent the approach form and the calibration of the great majority of the codes. Those formats can be applied to the treatment of investigated structures, according to different methods:

- **Level A** - Global safety factor formats and acceptable stresses. Level A constitutes a conservative criterion because the reduction of uncertainties cannot be made.
- **Level B** - Semi-probabilistic load and resistance factor formats and the use of the criterion of the state limits. The partial factors are specified according to the current knowledge of the uncertain parameters. Level B is the core in any modern design code.
- **Level C** – Probabilistic-based format on the reliability index and probability of failure. These formats present concepts of state limits, but they use numeric methods for the resolution of complex formulations. Such a format is demanded in more complex analyses in which level B is still conservative.
- **Level D** – Formats which take economical order considerations into account. Basically, they originate from partial safety factors (level B) or the probability of failure (level C), modified by economical criteria.

The aim of this research is based on the current efforts made to estimate the safety and service life of existent structures, submitted to an evaluation. Basically, the difference is in the period of time involved in the estimate. For evaluation purposes, the time used, is the time spent on the repair of columns situated in the garage of buildings, as already pointed out. Such interval comprehends a few months and this mainly affects the loads with significant temporary variation (ACHE, 2003). Therefore, in this study a format for estimating the safety will be adopted in consonance with the level B format, previously mentioned, by using the criteria of state limits and partial factors of safety. The updating of the resistance parameters and load effects at the moment of the analysis will be done, and the observation period will be extended according to necessity for the structural repair.

## 3. Methodology Proposal

The proposed methodology uses some studies carried out in structural repair which point to the use of rules and similar formats adopted by design codes of most countries, containing the basic guidelines: a) maintenance of state limits and b) application of partial factors of safety (MELCHERS, 2001; VAL & STEWART, 2002).
Other considerations were incorporated to the methodology, for instance, the one suggested by Allen (1991) who proposes that an evaluation criterion should be delineated according to more specific situations than the design criterion and the professional should consider consequences of failure in certain situations in critical structures. The evaluation should incorporate all the information obtained in the inspections, including the performance of the structure.
Alike other researchers, load effects were compared for the sake of safety, such as (bending moments and compression loads) that were more probable to occur in the columns of the first level starting from the foundations, usually in the garage of residential buildings, with the effective resistances of their cross sections at the moment of the intervention, through an ultimate limit state equation (LARANJA & BRITO, 2003).
In order to obtain the evaluation load effects, an adjusted structural modeling was used which was calculated by the program. The capacity of the current calculation programs in the market (spatial analysis, application of effects of local and global imperfections, wind load, and considerations of second global order effects, etc.), is capable of providing a more precise analysis than that made in the conception, mainly when the buildings are more than 20 years old. In garage columns, the evaluation load effects obtained by the program, despite its location, will be represented by biaxial bending moments (X and Y) and axial force.
The first step of the methodology is to perform the modeling of the existing building, taking into account all of the defined aspects in projects, delineating the behavior of the structure in a computerized simulation for more reliable results. After the application of the software, it is convenient that the reactions obtained in the foundations are closer to the project, because those were the reactions by which the structure was designed and executed. Thus, adjustments should take place in the continuity of the structure, in order to approach the reaction values supplied by the structural designer. At the end of the adjustments, the structure modeled in the software will be similar to the one projected, without the alterations which were incorporated during the construction, though.

Consequently, for use in the evaluation, the defined calculation model will be used with the input parameters. Those parameters, not possessing the generic characteristic of structures, hold peculiar considerations of the analyzed case, (LARANJA & BRITO, 2000). Thus, in the treatment of the relative uncertainties of the loads, the methodology here proposed, foresees the updating of the cumulative distributions, allowing the adjustment of the loads to value compatible to short periods and the use of live loads measured directly in the structures.
Safety partial factors to be applied for the generation of load effects and resistances of evaluation in the sections will be adjusted starting from the reduction of the uncertainties of these parameters and the definition of a reliability index for the existing structure. This will be made by using the formulations and simplifications utilized by the design codes for the determination of the design safety factors.
The evaluation parameters regarding the loads and their partial factors, defined in function of the short period analyzed for the structural repair, will be introduced into the program that will generate the load effects coherent with the current conditions of the structure, serving as a base for the safety quantification. Comparing the load effects generated and the resistances of the structure at the moment of the evaluation, a convenient method can be drawn to execute the intervention.
The definition of a coefficient that represents the state of the structure at the moment of the intervention is desirable for an evaluation method based on the ultimate limit state and on the global factor of quantification of the safety. A form of relating the load effects and the resistance of the section at the moment of the intervention is the equation ultimate limit state (equation 1) that will have, for this occasion, to relate the load effects and the resistances of the sections.
where:

$$\frac{R_{d,eval}}{S_{d,eval}} \geq 1 \quad (1)$$

$R_{d,eval}$ = factored resistance on evaluated column cross section
$S_{d,eval}$ = factored load effects on evaluated column cross section
By using the same philosophy of the state limits, global safety coefficient in the evaluation can be given by equation 2:

$$\gamma_{eval} \geq \frac{R_{k,eval}}{S_{k,eval}} \quad (2)$$

where:
$\gamma_{eval}$ = global safety factor on evaluated columns
$R_{k,eval}$ = nominal resistance on evaluated column cross section
$S^{k,eval}$ = nominal load effects on evaluated column cross section
The bending moments and the compressive axial loads act together and they can reach their maximum values simultaneously. The bending moments should be taken into account because their effects can cause collapse in repair sections. To obtain a global safety factor that involves only axial load effects it is possible to change the evaluation load effects (bending moment and axial force) in a concentric compressive axial load.
The procedure for the mentioned transformation will be accomplished with the calculation program used in the modeling of the structure. Starting from the evaluation load effects, proceed to the calculation of the reinforcement of the columns, maintaining the real concrete sections, using the load effect parameters and strengths of the materials with the partial factors for the evaluation. These reinforcements correspond to the necessary ones for the column during the intervention period and they will be defined as $As_{(int)}$.
Usually all of the columns of the buildings are reinforced symmetrically in function of the characteristics of the loads, generated mainly by the wind and by the simplicity to be executed. This criterion is common in the design of columns, besides, the version of 1978 of Brazilian Code ABNT NBR 6118 contained a simplified method for some load combinations and several researchers present interaction diagrams that allow the manual calculation of sections to be submitted to the combined axial load and biaxial bending moments with symmetrical bars.
Thus, an equivalent axial load capacity (Nd), obtained by the sum of the forces of the concrete and steel, $As_{(int)}$ (stresses multiplied by the corresponding areas) obtained on design presented similar results in the section such as those from the combined axial load and biaxial bending moments. This procedure can also be applied to the probabilistic treatment because the solution represents a safety margin. Therefore, the possible combinations generated in the random process will be contained in the safety area with a reliability index greater than the minimum acceptable index.
As the strengths of the column materials will be determined by the inspection procedures and accomplished tests, the axial load capacity of the section can be computed by equation 3.

$$N_d = K_{MOD} \times A_c \times f_{cd} + A_s \times \sigma'_{sd} \quad (3)$$

where:
$K_{MOD}$ = modified factor that takes into account some aspects that influence the strength of the concrete in the structure
$A_c$ = concrete cross area of the column
$f_{cd}$ = factored concrete compressive strength
$A_s$ = steel cross area on the column
$\sigma'_{sd}$ = steel stress related to 2‰ strain of reinforcement
Equation 3 will be used for the calculation of the sectional load capacity during the intervention $R_i$ considering the existing reinforcement in the section and for the calculation of the equivalent axial load capacity $S_{eval}$ with the reinforcement $As_{(int)}$ determined by the program. Thus, by using equation 2, the relation between the equivalent axial load and the axial load capacity of the section will be defined and both will be factored by the evaluation partial factors. The type of intervention to be adopted will be defined by applying the same equation 3, using the concrete section reduced by the cut and the bars partially corroded when this happens, which will orientate the works of structural repair. A chart of the methodology can be seen in Figure 1.
When the global factor is greater than the unit, even with the smallest reduction of the concrete section, the section will be capable of receiving the intervention. Otherwise, the necessary shoring should be provided, which is not included in the scope of this study.
In summary, the program provides the calculation of the necessary

symmetrical bars for the state of existing loads during the repair, usually an axial load and moments (F; Mx; My), are determined, if considered equal to the resistance of the sections. This allows calculating a concentric axial load that represents the nominal load effects on evaluated column cross section in the repair. The real section of concrete and reinforcement will supply the nominal resistance on the evaluated column cross section.

## *3.1 Load specifications on the evaluation*

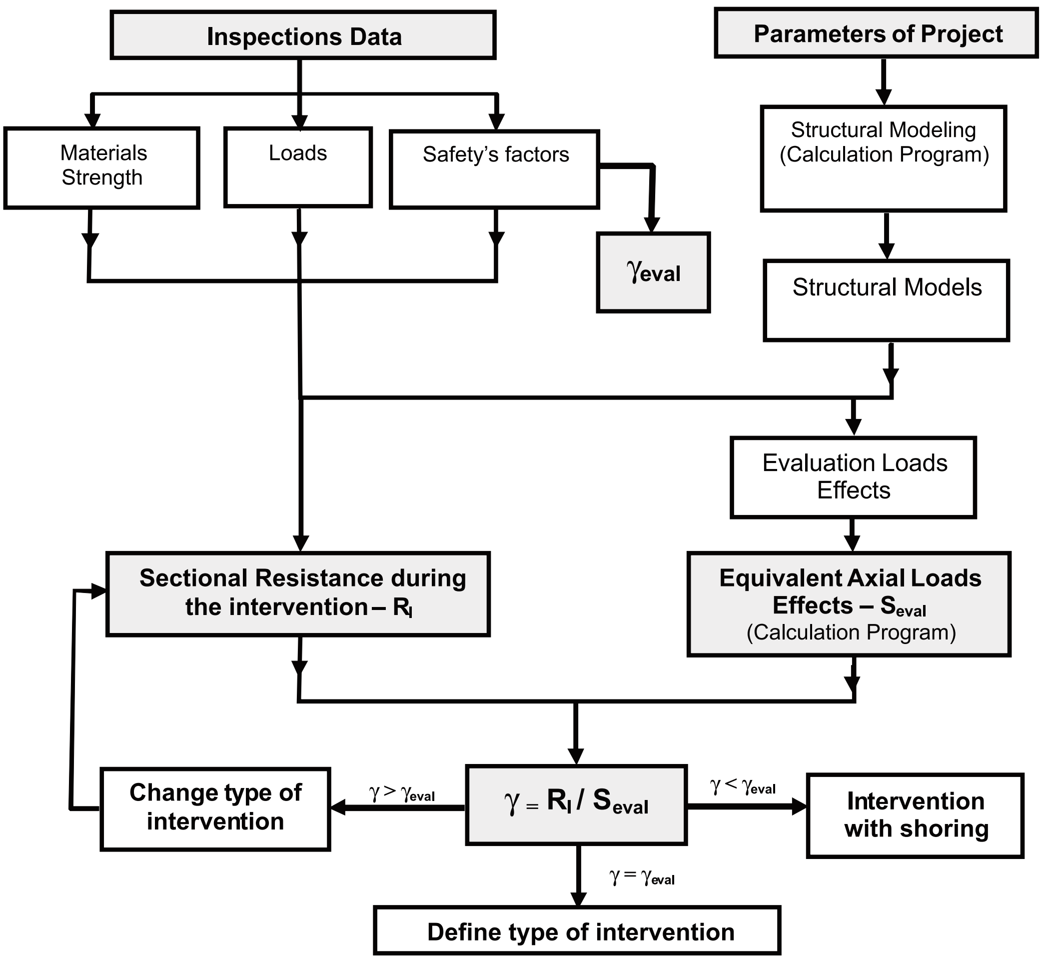
### 3.1.1 Dead loads

Basically, the dead loads are represented by self-weights of structural elements (slabs, beams, columns, etc.), permanent partitions (walls, finishings, etc.) and permanent equipment. Laranja & Brito (2000) mention that in safety analysis on a superior level, it is usual to assume that the load type has a normal distribution, with a mean value equal to the nominal value and that the coefficient of variation be between 5% and 10%.
In the structural evaluation, the acting permanent loads can be obtained with considerable precision through the geometric characterization of dimensions in cross sections, thickness of finishings in general, thickness of walls, etc. These geometric variations originated from the different phases during execution, and they depend on the construction technique, equipment and quality of the labor force (DA SILVA, 2002). In the design, they must be protected by the increase of their nominal values.
By the means of techniques that use chemical or physical mechanisms, it is possible to determine the real specific weight of the concrete and other materials involved in the production of the dead loads. This procedure, together with the dimensions obtained by inspections of geometric characterization of sections, reduces the uncertainties in the treatment of that load type in built structures (CABRÉ, 1994).

### 3.1.2 Live loads

The live loads presented in design codes, incorporate some basic procedures in its definition, such as the temporary variability that is represented by two components: i) a quasi-permanent one, due to

**Figure 1 - Methodology to repair of columns of reinforced concrete buildings**



the weight of furniture pieces and people in the several occupation changes in the buildings; ii) an intermittent portion of extraordinary active overloads in short periods of time (MELCHERS, 1999).
The live load design recommended by ABNT NBR 6120:1980 uses a return period between 140 to 200 years, with a low occurrence probability during the service life of the structure (between 25% and 35%). The conception of nominal values for these loads is based on extreme value type 1 distribution, and can result in the adding of the two components of the temporary variability according to the rule of Turkstra (COROTIS et al, 1981):

- quasi-permanent maximum value in the service life, added to the temporary maximum value in an occupation;
- temporary maximum value in the service life, added to the quasi-permanent value in an occupation;
- adding of the maximum values, in service life, of both components.

Considering the extremely reduced time interval, as in the the structural repair of columns, the live loads will be basically restricted to quasi-permanent component (ELLINGWOOD, 1996). Based on other authors' data, Corotis and Doshi (1977) analyzed the collected data of instantaneous live loads in buildings, obtaining the gamma distribution, as a better adjustment. This function was used by Hahn and Shapiro (1969), which presents the general formulation of the cumulative gamma distribution by equation 4.

$$F_L(x;\eta,\lambda)=\frac{\lambda^{\eta}}{\Gamma(\eta)}\int_0^{X} X^{\eta-1}e^{-\lambda X}dX \quad (4)$$

where:
$F_L$ = cumulative distribution function
x = random value
Γ (η) = gamma function
λ e η = parameters of distribution function
For integer values of η, the function gamma is transformed in to equation 5.

$$\Gamma(\eta)=(\eta-1)! \quad (5)$$

Using the expressions for λ and η presented by Hahn and Shapiro (1969) and the data of residential buildings obtained in data collection, for instance, with the mean of 0.544 kN/m² and standard deviation of 0.193 kN/m² one can obtain the cumulative distribution function, equation 6, to live loads (quasi-permanent) instantaneous.

$$F_L(x;\eta,\lambda)=\frac{15^{8}}{(\eta-1)!}\int_0^{X} X^{7}e^{-15X}dX \quad (6)$$

Monteiro (2006) used equation 6 and integrated it with inspection data presented by Corotis and Doshi (1977), according to an accumulated value of 95% (characteristic fractile), found, for the instantaneous live loads for the short periods of structural repair, the value $X_{95\%} = F_{q,eval}$ = 0.875 kN/m².

**Table 1 – Adjustment factors for wind speed**

| Period | Adjustment factor for wind speed (Sajust) |
|---|---|
| < 1 year | 0.80 |
| 1-5 years | 0.90 |
| 5-10 years | 0.95 |
| 25 years | no reduction |
| 50 years | no reduction |
| 100 years | 1.10 |

**Source:** Adopted by Rosowsky (1995)

### 3.1.3 Wind loads

The Brazilian Code ABNT NBR 6118:2003 requires that forces caused by the wind ought to be taken into consideration when designs are carried out. In the previous version of the code, this was not necessary. Although the buildings were projected using the previous version of the code, the reduced computer time required by the current programs, to analyze the horizontal forces in buildings, the evaluation in the repair interventions were allowed, such effects will always be considered.
The loads introduced by the wind are calculated from the pressure generated by the basic speed (V0). It is a representative value of a "3 second gust" with a probability of 63% of being exceeded once, on average, in the period of 50 years.
In the modeling of the incidence of the wind, according to Turkstra & Madsen (1980), the use of the gamma function distribution on the collected data by anemometers, is satisfactory in the specified observation periods. Melchers (1999) adopts the extreme value type I distribution as a good approach to represent such phenomenon.
In the present work, the use of the factors developed by Rosowsky (1995) is proposed. They are capable of converting wind speeds of periods of 50 years to other periods, according to the purpose of the evaluation.
In the determination of the adjustment factors, Rosowsky (1995) used an analysis of wind speed data obtained at airports as a reference. This composed the basis of the formulation of winds for designs under the North American codes. Table 1 presents the values obtained by the researcher for the adjustment of the basic speed.
From Table 1 and knowing that the repair services are, in general, of the order of a few months, one can adopt Sadjust as equal to 0.80. This generates equation 7 to determine the value for the nominal speed of the wind for evaluation.

$$V_{k,eval}=0.80\times V_k \quad (7)$$

Where as:
$V_{k,eval}$ = nominal wind speed in evaluation
$V_k$ = nominal wind speed according to ABNT NBR 6123:1988 code.

## 3.2 Considerations on material strength in evaluation

### 3.2.1 Steel strength

In steel yield tests, when their results are up to design values, they won't be used in the updating and considerations on steel yield property in the evaluation. Thus, for reasons of safety warranty, the strength to 2‰ strain will be adopted for evaluation ($\sigma'_{sd,eval}$ = 420 MPa) or the same strength used in design. In the cases of lower results than those in the designed nominal face value, such information will be taken into account because it is an attempt against safety.

### 3.2.2 Concrete strength

The concrete compressive strength is one of the problems faced by engineers during evaluations, due to the need of measuring the potential safety presented by the evaluated structure. Basically, the complexity of the behavior of this material in service over time is due to two evidenced phenomena (FUSCO, 1993):

- increase of strength due to the slow hydration;
- loss of strength due to the sustained loads.

Both phenomena, together with the influence of the specimen dimensions, compose the partial factor to compressive concrete strength ($K_{MOD}$). This factor should be taken into consideration in design to avoid that structures collapse because of exhaustion of the resistant capacity during a certain period of its life cycle.
The compressive concrete strength in evaluated structures usually follows two obtaining procedures: a) extrapolations based on the control concrete data during the construction, taking into account the age and the effect of sustained loads; b) tests in drilled cores in case no results of concrete control exist, considering, however, the effect of duration of the loads. In any of the ways, one can use additional tests such as sclerometry and ultrasound scan.
The effect of the sustained loads in the loss of concrete strength refers to the slow propagation of cracks in the matrix of the hardened paste. The phenomenon, which initially was studied by Rusch (1960), happens for load effects that generate strengths above 70% of the strength obtained by standard axial tests in concrete specimens. For values below this limit, the material presents stable strength, in spite of the occurrence of creep.
In relation to the variability of the concrete strength, in general, the reduced number of specimens or drilled cores constitutes a barrier for the necessary estimate of the coefficient of variation ($\delta_c$). This parameter is important because it constitutes a measure of dispersion of the values of this random variable and contributes for the estimate of reduced partial factor in the concrete strength.
By such facts, a usual procedure consists of the execution of tests of sclerometry or ultrasonic pulse on the concrete of the studied elements for the verification of $\delta_c$. The objective of using the sclerometry test parallel to axial standard tests of specimens is to help in the obtaining of additional information that can reduce the possibility of mistakes in the evaluation of several properties of the concrete (ALCÂNTARA, 2002). The use of the ultrasonic pulse test is indicated for definition of the characteristics of the concrete, its homogeneity and even the compressive strength. With the results of these non-destructive tests and the use of the Bayesian updating, one can improve the representativeness of the group of information already existent which are derived from the specimens or drilled cores (VAL & STEWART, 2002).

In the determination of the concrete strength in existent structures, two situations are frequently found:

*a) Existence of technological control*

The structures that nowadays go through interventions for rehabilitation, are, in most of the cases, more than 20 years old, for which, the estimation of the current concrete strength, demands the use of specific evolution strength curves of cements used at that time. The Model Code CEB 1990 presents an equation for the estimation of the evolution of the concrete strength over time, by which one can obtain $K_{MOD1}$. The Brazilian Code ABNT NBR 6118:2003 also allows the application of a similar equation 8, that can be applied to the specified compressive strength ($f_{ck,est}$) obtained by compression test of control specimens at the time of construction, when one wants to obtain the strength at the end of a given period of time:

$$K_{MOD1} = \frac{f_{c,t}}{f_{c,28}} = \exp\left\{ s\left[1 - \sqrt{\frac{28}{t}}\right]\right\} \quad (8)$$

where:
$K_{MOD1}$ = factor related to concrete strength (related to 28-day-old strength)
$f_{c,t}$ = "t"-day-old concrete strength
$f_{c,28}$ = 28-day-old concrete strength
S = coefficient that depends on cement type. The following equivalence is permitted:
S=0.2 to cement ARI (high early strength)
S=0.25 to cement CP I, CP II grade 40
S=0.38 to cement CP III and CP IV
In the evaluation of columns of existent structures, knowing the dimensions of the cross sections and the loads, it is possible to estimate the strengths above the limit of 70%, which would cause a reduction of the concrete strength for the Rüsch effect. With this methodology, in consideration of the Rüsch effect, one suggests that the load effects are the total of the calculation, applied in a fictitious way, on the 28th day. If the computed determined limit is overcome as such, the reduction in the strength can be calculated by equation 9, extracted from the Model Code CEB 1990. In case such a limit is not overcome, it will be assumed that $K_{MOD2,eval}$=1.

$$K_{MOD2} = \frac{f_{c,t}}{f_{c,t_0}} = 0.96 - 0.12 \times \sqrt[4]{\ln 72 \times (t - t_0)} \quad (9)$$

where:
$K_{MOD2}$ = reduction factor to compressive concrete strength to sustained load effect (related to 28 day strength)
$f_{c,t}$ = compressive concrete strength at the age of ($t+t_0$) under high load and sustained until the t0 age
$f_{c,t0}$ = compressive concrete strength at $t_0$ age, defined in standard axial test
The influence of the dimensions of the tested specimens in the real strength of the concrete, according to Fusco (1993), should be taken into account in project and structural evaluation. Rusch (1980) observes that cylindrical specimens of 15 cm of diameter

by 30 cm of height possesses strength, in general, to the order of 5% larger than the one of the same concrete in the structure. Thus, one can adopt the relation by equation 10.

$$f_{c,structure} = 0{,}95 \times f_{c,cylindric,15x30cm} \text{ or } K_{MOD3} = 0{,}95 \quad (10)$$

In general the smaller the h/d ratio of the specimens, the larger is the obtained strength. A diagram of conversions for other h/d ratios can be found in Fusco (1993).
Having defined $K_{MOD1}$, $K_{MOD2}$ and $K_{MOD3}$, the $K_{MOD}$ can be composed by equation 11, emphasizing the importance of the effects of the behavior of the concrete over time for the evaluation:

$$K_{MOD,eval} = K_{MOD1} \times K_{MOD2} \times K_{MOD3} \quad (11)$$

The concrete compressive strength to use in this case is given by equation 12:

$$f_{cd,eval} = K_{MOD,eval} \times \frac{f_{ck}}{\gamma_{c,eval}} \quad (12)$$

where:
$K_{MOD,eval}$ = factor of compressive concrete strength in evaluation
$f_{ck}$ = specified compressive concrete strength at the age of 28 days
$\gamma_{c,eval}$ = strength reduction factor in evaluation

*b) Inexistence of technological control*

The technological control of the materials has been necessary for all of the works for many years, as well as the file of the whole documentation. However, few buildings have conserved the documentation, or the entire control was not always accomplished. In these cases, the extraction and tests of cores must necessarily proceed directly from the studied columns. The lots are taken into agreement with the Brazilian Code ABNT NBR 7680:2007, of those which, depending on the size of the sample, allow obtaining relative conclusions about the concrete strength submitted to the evaluation.
The Rusch effect should be evaluated assisting to the same criterion of situation "a", analyzing the limit of 70% of the strength admitted by the cylindrical specimens after the 28-day-old process. The relative influence of the dimensions of the drilled cores is quite accentuated for this situation, as not always a relationship h/d=2 is attained. The diameter of the core should not be less than three times the aggregate diameter, or less than 10 cm. The ABNT NBR 7680:2007 presents a table for the correction of values for several h/d relationships.

### 3.3 Considerations on safety factors

The uncertainties of the loads and resistances in the design stage

**Table 2 – Factors contributing to the adjustment of the reliability index**

| Factor assessment | $\Delta i$ |
|---|---|
| **Inspection / Performance - D1** | |
| Without inspection or design of implementation | -0.40 |
| Inspection for identification / location | 0.00 |
| Satisfactory performance[a] or measurement of permanent loads[b] | 0.25 |
| **Behavior of structural system - $\Delta_2$** | |
| Failure leads to collapse, likely occurrence of injuries | 0.00 |
| Intermediate situation | 0.25 |
| Local collapse, unlikely occurrence of injury | 0.50 |
| **Category of risk for failure - $\Delta_3$** | (b) |
| Very High (post-disaster or n > 1000)[c] | 0.00 |
| High ( n= 100–1000)[d] | 0.00 |
| Normal (n= 10 –99)[d] | 0.25[d] |
| Low (n= 0 –9)[d] | 0.50[d] |

**Remarks:**
a) Applied for permanent load factors and variables, at the age of 50 years or more, without structural deterioration;
b) Applies only to the permanent load factor;
c) Parameter n is determined as the maximum number of persons exposed to failure;
d) Reduce to 0.25 for loads of occupation of meeting or structures of wood.

**Source:** Allen (1991)

are reflected in the partial factors (ALLEN, 1991). According to Montoya et al (1973), in the structural safety establishment, which are accomplished with simplified considerations of level B. The several causes of mistakes and uncertainties in which there is some knowledge, are attributed to two variables: resistance of the materials and the value of action taken. The characteristic values of the mentioned variables are considered by partial safety factors, in order to take into account the remaining random factors that influence the process, about which, knowledge is still incomplete.
Val & Stewart (2002) point to an adjustment of these partial factors in the evaluation for the updating of the distribution functions of the variables by them factored, by means of inspections and tests on site, with consequent reduction of the uncertainties inherent to them.

### 3.3.1 Target reliability index of existents structures

The founding of the reduction factors retake, besides the variability assumed for the actions, a probability of failure (Pf) acceptable for the structures and which is implicit in design codes, under the form of a reliability index (β). In projects of new buildings, the value of β is close to 3.8. In some countries, the values of β are close to 3.5 (Pf=2.33x10-4) (COST 345, 2004). In Brazil, during a process of reliability analysis in structural elements designed with the Brazilian codes, Santos & Eboli (2006), found that few were the accomplishments in which the reliability index β = 3.8 was attained. In the graphs presented by the authors, it was verified that, for columns of buildings, with a permanent load/total_load relation, close to 0.85, which is a relation usually found, the value of β = 3.5 was also less attained.
It has been outlined that the founding of a probability of acceptable failure for the calibration of future evaluation codes is necessary, and that it be coherent with the current situation of the structure and its performance, presented until the moment of the evaluation,(MELCHERS, 2001). As a certain work was done in order to obtain partial factors for the evaluation of structures of bridges and existent buildings, it's possible to assume that the consideration of the same β used in project (Val & Stewart, 2002) is satisfactory. A similar consideration was proposed in a work in which the index of acceptable reliability in the evaluation should stay closer to the value presented by the structure when built (Tanner, 1995).
Due to the fact that existing structures presented satisfactory performance, at the same time in which they were inspected in a careful way, the evaluation criteria for these should not be as conservative as those in the project of new buildings (ALLEN, 1991). This fact induced the author to introduce different levels of safety for existing structures, through adjustment done with the individual contribution of the factors given in Table 2.
Such a procedure demands common sense on behalf of the structural engineers responsible for the evaluation (Laranja & Brito, 2003), as well as involving a subjective criteria as inspection quality and probability of personal risks. However, the method allows determination of the reliability index in the evaluation ($\beta_{eval}$) starting from the β of design, applying some reduction terms that vary from one structure to another, according to equation 13:

$$\Delta = \beta - \beta_{eval} \tag{13}$$

where:
Δ = contributory factor to adjustment of the reliability index for structural evaluation
β = design reliability index
$\beta_{eval}$ = reliability index for structural evaluation
For the evaluation of columns in existing structures with steel corrosion, detected in a routine inspection, a reliability index could be reduced, in agreement with the methodology proposed by Allen (1993), resulting in the value of 3.25 (*Pf*=5.77x10$^{-4}$).

### 3.3.2 Safety factor to load effects

*a) Dead loads*

With the reduction of the relative uncertainties to the active permanent loads in buildings, provided by the procedures of measurements and risings made directly in the transverse sections, as well as the lengths of pieces and the characterization of other fixed non-structural elements, it is reasonable to adopt less conservative coefficients for the evaluation of a particular structure (VAL & STEWART, 2002).
A reduction in 10% in the partial load factor of the permanent actions was proposed by Cabré (1994) in his study of the residual life of existent buildings. Using this information for Brazilian reality, in the evaluation procedure, under precise inspection conditions and tests, the value proposed for the partial factor for permanent actions is:

$$\gamma_{f,g,eval} = 0.90 \times \gamma_{f,g} = 0.90 \times 1.4 = 1.26$$

However, $\gamma_{f,g,eval}$, would still be admitted, with a value close to 1.2, for structures with residual life service quite reduced, submitted to geometric characterization, inspections, tests and without the presence of sensitive damages (LARANJA & BRITO, 2000).
For the variable actions it's also possible to reduce the partial factor when limiting the development of extreme transient loads and when limiting the analysis interval to the short period regarding the structural repair.
The ACHE (2003) shows the formulation used to obtain the partial factors for the permanent actions in design, considering a normal distribution. Such information, adapted to the obtaining of the coefficients for the structural evaluation can be represented by equation 14.

$$\gamma_{G,eval} = 1 + (\alpha_{G,eval} \times \beta_{eval} \times \delta_{G,eval}) \tag{14}$$

where:
$\gamma_{G,eval}$ = partial dead load factor in evaluation
$\beta_{G,eval}$ = reliability index in evaluation
$\alpha_{G,eval}$ = influence factor for permanents actions in evaluation
$\delta_{G,eval}$ = coefficient of variation of permanent actions obtained by on-site measurements
The average coefficient of variation for dead loads, adopted in design, is estimated at 10%. An expressive reduction of that parameter can be obtained by measurements of the structure (ALLEN, 1991). In this case, according to Laranja & Brito (2003), $\delta_{G,eval}$ could be reduced to 5%. The factor to permanents actions ($\alpha_{G,eval}$) can be

taken according to the calibration values for design codes (ACHE, 2003). For a situation as such, α with the value of 0.70 can be adopted (RILEM, 1996).

*b) Live loads*

For live loads, the inherent variability is represented by a coefficient of variation in a 10% to 30% range (ALLEN, 1991). Val & Stewart (2002), in their work in order to define coefficients for evaluation, used a 30% value for the coefficient of variation.
The calibration for live loads uses an extreme type I distribution system, as a base (ACHE, 2003). The same calibration is used to adjust the life service of the design (LARANJA & BRITO, 2000). That distribution function is inappropriate for reduced periods. The approach is made using a gamma distribution system such as that proposed by COROTIS & DOSHI, (1977).
In the methodology presented here, a 95% fractile transformation is used, which was adopted after the definition of the characteristic value of the loads, for a 99.5% fractile, according to the implicit probability in the design codes for that partial factor (FERRY-BORGES & CASTANHETA, 1971). Therefore, equation 15 should be satisfactory.

$$\gamma_{f,q,eval} = \frac{F(x)_{99.5\%}}{F(x)_{95\%}} \tag{15}$$

In which:
$\gamma_{f,q,eval}$ = partial load factor in evaluation
$F_{(x)99.5\%}$ = cumulative distribution function to 99.5%
$F_{(x)95\%}$ = cumulative distribution function to 95%
In the case of live loads, integrating equation 6 for both fractiles (95% and 99.5%) and applying equation 15 to the obtained values, the result for the partial load factors for live loads is $\gamma_{f,q,eval}$ = 1.30.
Due to the variable behavior of load effects, produced by wind forces, and for being out of human control, is an unsuitable possibility for the reduction of the partial load factor (ACHE, 2003). By the methodology here proposed, in case this type of action occurs, safety partial factors in the evaluation will be considered according to the same values established in design, as a form of guarantee of minimum safety.

### 3.3.3 Partial concrete strength factor

The concrete strength given in tests is frequently represented by normal distribution (e.g. FUSCO, 1976; MELCHERS, 1999; LARANJA & BRITO, 2003). The partial factor in terms of a semi-probabilistic method can be defined by equation 16, valid for the normal distribution (TANNER, 1995).

$$\gamma_m = \frac{(1-1.645\times\delta_R)}{(1-\beta\times\alpha_R\times\delta_R)} \tag{16}$$

In which:
$\delta_R$ = coefficient of variation of resistances
$\alpha_R$ = influence factor
β = adopted reliability index

The influence factor ($\alpha_R$) is a function of the standard deviation of resistances and actions, which is in the 0.7 and 0.8 range (RILEM, 1996). In this methodology $\alpha_R$ will have the value of 0.75.
Global safety factor for columns
In a simplified way, it can be admitted that the global safety coefficient is measured by the product of two partial coefficients mentioned previously. A probable failure in columns due to the materials will happen because of low resistance of compressed concrete (MONTOYA et al, 1973). In that way, the global safety will be established by the marginal safety credited to the concrete and the dominant action, in this case represented by the permanent loads. In such situations, the structure is safe against possible collapse by the coefficient expressed in equation 17.

$$\gamma = \gamma_c \times \gamma_f \tag{17}$$

According to the design coefficients, recommended by ABNT NBR 8681:2003, the expected global safety for columns in a structure recently built, with γ = 1.40×1.40=1.96 is obtained, using equation 17. This value is in accordance with the conventional range of design, between 1.7 and 2.0 (MELCHERS, 1999). Under the evaluation conditions, new partial factors are defined regarding particular information. A new coefficient of global safety is defined for the columns of the existing buildings studied, starting from the considerations related to the new partial coefficients of safety.
For the evaluation of columns, if the specific conditions in which the structure of the building is going under structural repair services are satisfactory, the safety global coefficient is given by equation 18. Thus, one can conclude by the success of the services executed, a probability of failure given by the reliability index ($\beta_{eval}$).

$$\gamma_{eval} = \gamma_{C,eval} \times \gamma_{G,eval} \tag{18}$$

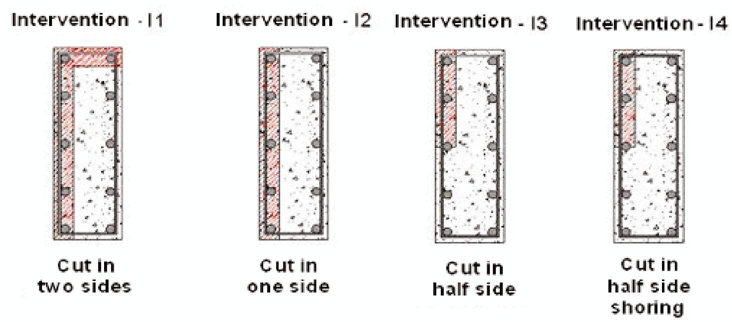
The coefficient regarding permanent load effects is used because it is related to the principal action in the structural safety (COST 345, 2004). The value given by equation 18 will be useful for the determination of a general plan to act in the section of the column during the period of the repair services. Starting from the global coefficient $\gamma_{eval}$, it can be determined the need of shoring.

## 4. Definition of intervention type

The term "intervention type" refers to the cut of the concrete section that will be made during the repair. Although other forms may exist, four types were considered. After the calculation and analysis of the global safety coefficient of evaluation defined for the building, it is decided which repair procedure to adopt:

- establishment of the amount of deteriorated concrete to be extracted from the section at once, maintaining pre-established safety terms; or
- the shoring of the structure when the global coefficient exceeds the safety global coefficient.

Figure 2 – Types of proposed interventions – cross section view




A general orientation in repair works, proposed as standardization for analysis, is presented in Figure 2.

For the removal of deteriorated concrete portions that guarantees the perfect asepsis of the bars and the suspension of the corrosive process, it is necessary to remove the material not only from the cover layer, but also from a deeper part of the concrete. It was considered necessary to remove and perform the scaling of a depth of 1.0 cm in the interior part of the longitudinal bars, which usually results in a final extraction depth of 5.0 cm in average (1.5 cm of bar covering, 0.63 cm of tie and 1.6 cm of longitudinal reinforcement). A scheme of the extraction is shown in Figure 3.

The height of removal should be extended to the points where corrosion in the bars still exists. In general, in the garages of buildings, the deterioration extends up to 1.0 m from the floor. This is the usual height for depassivated bars with enough humidity for the propagation of the corrosive process.

In the cases where the cut is bigger than 1.0 m high and there is a need to extract the deteriorated concrete in more extensive strips of the column, it is convenient to remove the material in parts, fractioning the execution of the service to respect a maximum height of concrete removal at once, which will be the function of the bar diameter. This procedure seeks to protect the longitudinal bars against the possibility of local buckling, usually restricted by the ties and by the concrete cover on the bars.

When the resistance of the cross section of the column allows the possibility of intervention for the removal of deteriorated concrete, according to the evaluation loads, it is necessary to regulate the form of execution of the services. In general, if the global coefficient allows, the possibility of cut and removal of material of the whole section should be discarded. That is because of the possibility of local buckling of the longitudinal bars in case they remain, at the same time, without a due concrete cover, even if not very thick. The intervention is also discarded without shoring when the safety global coefficient is very close to the minimum.

The interventions should always be done by specialized companies for this kind of service. The indications presented in Figure 3 don't constitute the only possibilities for execution of the services. An interaction between consultant and the specialized company should be developed, which can generate other types or intervention schemes. Other considerations will still be able to be made in agreement with the labor force and available repair products.

## 5. Case Study

### 5.1 Building "A" characteristics

The building "A" (Figure 4) was designed for 25 floors and it had the construction interrupted on the 17th floor in the year of 1995. Because of the retaking of the works, it was necessary to carry out works of evaluation of the general conditions of the existing structure, because it presented several deterioration indications. This propitiated the obtaining of the general data used in this research. For this case, a need for intervention was simulated for the repairing of the columns of the garage on the current date, supposing

Figure 3 – Schematic representation of the removal depth of deteriorated concrete in the column

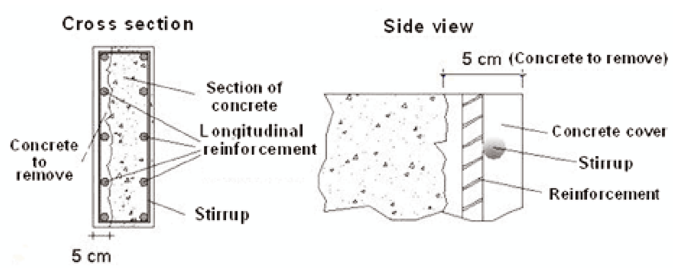


Figure 4 – Aspects of building "A"

that the building had been concluded, was in use for several years, and that the data used, had been obtained especially for such an unreal purpose.
The data for the technological control of the concrete or of the steel was not available. Only the architectural and structural data from existing projects were made available for the research. Some general information from the building is presented.

- Number of floors: 25
- Floor area (tower): 225.00 m$^2$
- Number of columns (tower): 14
- Concrete strength of columns: 25 MPa (read mix)

Data from the columns of the main tower are presented in Table 3.

## *5.2 Field's data and tests*

### 5.2.1 Geometric characterization of sections

For the geometric characterization, the dimensions of the beams, columns and slabs were obtained by inspection. The statistical results were fitted in normal distribution. Some data and results are shown in Table 4.
In general, the dimensional behavior of the inspected elements, as

Table 3 – Garage columns information – building "A"

| Column | Cross section (cm) | Bars diameter (mm) | Quantity of bars | Vertical characteristic load in foundation – project (kN) |
|---|---|---|---|---|
| P1 | 20x120 | 20.00 | 34.00 | 2990.0 |
| P2 | 30x150 | 25.00 | 46.00 | 7210.0 |
| P3 | 20x120 | 20.00 | 34.00 | 4620.0 |
| P4 | 50x80 | 25.00 | 32.00 | 7180.0 |
| P5 | 72x100 | 25.00 | 58.00 | 13740.0 |
| P6 | 30x120 | 20.00 | 50.00 | 6590.0 |
| P7 | 20x120 | 20.00 | 44.00 | 5090.0 |
| P8 | 20x120 | 20.00 | 44.00 | 5570.0 |
| P9 | 20x150 | 20.00 | 44.00 | 5810.0 |
| P10 | 52x52 | 20.00 | 32.00 | 4800.0 |
| P11 | 30x120 | 20.00 | 50.00 | 6970.0 |
| P12 | 20x80 | 16.00 | 28.00 | 2390.0 |
| P13 | 20x193 | 20.00 | 48.00 | 6710.0 |
| P14 | 30x100 | 16.00 | 32.00 | 4060.0 |

**Table 4 – Geometric characterization of columns, beams and slabs – statistical results**

| Element | Dimension | Design value (cm) | Mean (cm) | Standard deviation (cm) | Coef. of variation % | Evaluation value (95%) (cm) |
|---|---|---|---|---|---|---|
| beam | Width (b) | 12 | 12.4 | 0.56 | 4.52 | 13.32 |
| | Height (h) | 55 | 54.9 | 0.53 | 1.20 | 55.77 |
| column | Smalle rside | 20 | 20.3 | 0.29 | 1.44 | 20.7 |
| | | 30 | 30.6 | 0.32 | 1.05 | 31.1 |
| | | 50 | 50.5 | 0.33 | 0.66 | 51.0 |
| | | 52 | 52.8 | 1.03 | 1.95 | 54.5 |
| | | 72 | 72.4 | 0.29 | 0.40 | 72.8 |
| | Larger side | 80 | 80.4 | 0.66 | 0.83 | 81.4 |
| | | 100 | 100.7 | 1.21 | 1.20 | 102.7 |
| | | 120 | 119.7 | 0.65 | 0.54 | 120.8 |
| | | 50 | 149.5 | 0.61 | 0.41 | 150.5 |
| | | 193 | 193.3 | 0.20 | 0.1 | 193.6 |
| slab | Thickness | 10 | 10.15 | 0.53 | 5.3 | 11.02 |
| | | 12 | 12.3 | 0.50 | 4.06 | 13.10 |
| | | 14 | 14.6 | 0.45 | 3.08 | 15.34 |

well as the standard deviations found, came in the range of tolerances allowed for the design of new structures.

### 5.2.2 Concrete and steel strength tests

Because of the inexistence of any registrations of the technological control of the concrete and steel of the columns, the execution of several tests for the investigation of their real conditions on the current date were carefully carried out. With this purpose, there were drilled and tested cores, besides the use of non-destructive testing of ultrasonic and sclerometry. The tested elements included the columns of the garage, beams, slabs and columns, which served as samples on other pavements.
From the core tests one can calculate the compressive strength of the existing concrete, while the ultrasonic test aided in the estimation of the coefficient of variation. The results for the columns are presented in Table 5.
The number of cores of the sample is in the interval: $6 \leq n \leq 20$
where as:
n = number of cores of the sample
By applying the Brazilian Code ABNT NBR 12655:2006 to the results shown in Table 5, respecting their particular conditions of use, the characteristic compressive strength for the evaluation $f_{ck,eval} = f_{ck,est} =$ 24.66 MPa, was obtained. With the same results, one can also obtain an average of 30.1 MPa and 0.13 for the coefficient of variation.
The results of the statistical calculation of the resistance for the compressive strength obtained through ultrasonic tests from columns of several floors are presented in Table 6.

**Table 5 – Strength compression tests in concrete cores**

| N° of cores | Cores | Strength (MPa) | $f_{ck,est}$ (MPa) |
|---|---|---|---|
| 7 | 1 | 27.32 | 24.66 |
| | 2 | 35.71 | |
| | 3 | 35.51 | |
| | 4 | 25.76 | |
| | 5 | 29.44 | |
| | 6 | 30.78 | |
| | 7 | 26.22 | |

**Table 6 – Statistical results for concrete compressive strength by ultrasonic tests**

| Element | Floor | Number of tests | minimum value (MPa) | Maximum value (MPa) | Standard deviation (MPa) | Coefficient of variation (%) |
|---|---|---|---|---|---|---|
| Column | Garage | 30.0 | 21.3 | 28.2 | 1.94 | 8.0 |
| | 1$^{st}$ | 9.0 | 23.0 | 27.7 | 1.52 | 5.8 |
| | 2$^{nd}$ to 8$^{th}$ | 18.0 | 20.4 | 27.5 | 23.4 | 10.1 |
| | 9$^{th}$ to 15$^{th}$ | 21.0 | 12.7 | 23.3 | 2.18 | 13.2 |

For the columns of the garage floor, the results obtained for compressive strength of the sample were the mean of 24.96 MPa and the standard deviation of 1.94 MPa. These values result in a coefficient of variation of 0.08 of the sample that, as already defined, was used as the partial factor of the concrete strength in the evaluation as $\delta_{c,eval}$. This value is lower than that obtained for the cores, which was 13%, but it was considered because the number of tests were higher than the number of cores, and the mean value of compressive strength estimated by the cores and ultrasonic tests, were closer.
The tested bars of steel were extracted from the lap splices on the last casting floor (17th floor). Due to the damage already presented in the bars and taking into account that the sample tested probably didn't correspond to the same steel used in the garage columns, the obtained results didn't supply effective conditions for evaluation. Therefore, these results were not incorporated in the model of uncertainty reduction proposed in this work.

# 6. Application of the Proposed Methodology

Following the methodology and starting from the existing structural project, the modeling of the structure was made in commercial software program. After several adjustments in the calculations, the loads obtained in the foundation were satisfactorily closer to the project loads. The steps for the application of the methodology are developed as follows.
The obtained data of the inspections and tests, as well as the studies of the live loads of reduced periods, allowed the obtaining of more appropriate parameters for the structure being studied, according to current conditions. On the basis of the methodology presented previously, the information regarding the building could be updated and it happened in the following manner.

## 6.1 Material strengths

### 6.1.1 Concrete strength and factor in evaluation ($K_{MOD,eval}$)

As already exposed, the concrete strength of the columns of the garage, by the tests made in cores, was of 24.66 MPa, which didn't cause an increase in the value used in design. In reality, the strength for the tests with cores was lower compared to that proposed in design. Therefore, for the structure in study, the $K_{MOD1}$ value was 1.0.
The factor $K_{MOD2}$ was obtained by the analysis of the calculation loads considering, in a theoretical way, the full application after 28 days. As presented, the reduction of the strength happened in the cases in which the load effect was beyond 70% of the calculation stress. The load effects were obtained from the model in the calculation program, with input parameters and the resistance of the section of the columns starting from the individual characteristic resistances of the concrete and steel, lessened by their respective partial factors of calculation. Table 7 presents the results of the analyses.
The $K_{MOD3}$ adopted for building "A" was 0.95, the ratio h/d of the drilled cores was in the order of 2.0. Thus, the following $K_{MOD,eval}$, (Table 8) were used to modify the concrete strength in case of a necessary safety evaluation of their columns, through equation 11.

### 6.1.2 Properties of steel reinforcing bars

For building "A" no tests were run for the steel used in the columns of the garage, the same yield strength of design (CA50), a strain of 2.0‰ and modulus of elasticity of 21 GPa, was used for the proposed evaluation, thus, the σsd,eval value was 420 MPa.

Table 7 – $K_{MOD2}$ analysis for building "A"

| Column | Factored axial load $S_d$ (kN) | *Factored axial resistance $R_d$ (kN) | Ratio $S_d/R_d$ | ** $K_{MOD2}$ |
|---|---|---|---|---|
| P1 | 4959.36 | 7990.41 | 62% | 1.00 |
| P2 | 11076.38 | 15870.52 | 67% | 1.00 |
| P3 | 6468.00 | 7990.41 | 79% | 0.75 |
| P4 | 9693.95 | 12590.64 | 77% | 0.75 |
| P5 | 19236.00 | 22740.17 | 82% | 0.75 |
| P6 | 9226.00 | 11880.21 | 75% | 0.75 |
| P7 | 7749.22 | 11220.77 | 69% | 1.00 |
| P8 | 7523.45 | 11220.77 | 67% | 1.00 |
| P9 | 7835.27 | 10150.63 | 78% | 0.75 |
| P10 | 7603.96 | 8310.88 | 91% | 0.75 |
| P11 | 10060.79 | 11880.21 | 85% | 0.75 |
| P12 | 3808.98 | 4810.17 | 78% | 0.75 |
| P13 | 9394.00 | 12120.83 | 75% | 0.75 |
| P14 | 6202.42 | 7590.09 | 79% | 0.75 |

**Notes:**
* Obtained by the contribution of the strength of the concrete, considered equal to the $f_{ck}$ value and the contribution of the reinforcement with tension corresponding 2‰ strain and steel yield strength considered equal to the one in project.
** See 3.3.1

| Table 8 – $K_{MOD,eval}$ value – building "A" | | | | |
|---|---|---|---|---|
| Column | $K_{MOD1}$ | $K_{MOD2}$ | $K_{MOD3}$ | $K_{MOD,eval}$ |
| P1, P2, P7 e P8 | 1.0 | 1.00 | 0.95 | 0.95 |
| P3, P4, P5, P6, P9, P10, P11, P1, P13 e P14 | 1.0 | 0.75 | 0.95 | 0.71 |

## 6.2 Actions updating

### 6.2.1 Permanent actions

The values of the dimensions of the related cross sections were obtained by the characteristic fractile of 95%, using data of geometry of the building structural elements, according to data from Table 4.

### 6.2.2 Actions due to live load

As an input parameter for the calculation of the evaluation load effects, only an occupancy load was adopted, according to the accumulated fractile of 95%, in the adjustment of inspection data by gamma distribution function and given by equation 6.

### 6.2.3 Actions due to wind load

In the determination of the actions due to the wind, for the specified period of structural evaluation, a basic speed of 80% of the amount used in the design was admitted. The reduction factor defined by Rosowsky (1995) was used.
Taking into account that the basic speed adopted for design in the city of Uberlândia is 34 m/s, a value equal to 27.2 m/s was obtained, for evaluation, within a timeframe of two to three months. The factors $S_1$, $S_2$ and $S_3$, as well as the drag coefficients, were taken in agreement with those established by ABNT NBR 6123:1988, considering the location of the building and its dimensions.

## 6.3 Probability of failure

For the settling of the reliability index for evaluation purposes, it was considered that the need of the supposed intervention had been defined by a complete inspection of the construction, by which the occurrence of the pathological problem of steel corrosion in all of the columns of the garage, and its deterioration be in the initial state. It was admitted that these columns possess such duty that any failure would lead to collapse, putting over 100 people in risk. Considering that in Brazil a defined reliability index doesn't exist for designs or for existing structures, Table 2 will be used. Considering a study made by Santos & Eboli (2006), a project β with the value of 3.5, which, by equation 13, results in a $\beta_{eval}$ equal to 3.25.

## 6.4 Partial materials and load factors

### 6.4.1 Concrete partial factor

Due to the particular characteristics of the analyzed building, it was possible to determine new partial factors keeping, however, an acceptable probability of failure, whose reliability index was defined by $\beta_{eval}$.
For the concrete of the analyzed columns, the partial factor was obtained from $\beta_{eval}$ and from the coefficient of strength variation, defined by ultrasonic tests. Considering 3.25 as a reliability index, $\delta_{c,eval}$ equal to 8% and considering αR equal to 0.80, with the use of equation 16, a $\gamma_{c,eval}$=1.10 value was obtained. This value represents a reduction of 21% in relation to that established for design of new structures. This high reduction was due to the low variation coefficient obtained by the ultrasonic tests and would be advisable to extract a larger number from cores and to use the coefficient of variation from them.

### 6.4.2 Steel partial factor

Due to the abscence of technological control in the execution and tests not done with a sample of bars of steel from the garage columns, a partial factor was adopted equal to the design value, that is, $\gamma_{S,eval}$ = 1.15.

### 6.4.3 Dead loads partial factor

In the case of the permanent actions, in which inspection was done, the design uncertainties could be reduced. Thus, the coefficient of variation of this parameter would be reduced to the value of 5% (ALLEN, 1991). However, the value of 7.5% was adopted in $\delta_{G,eval}$ to avoid possible mistakes for the standard sampling of measurements of the structural elements. Using equation 14, with this value, considering the $\beta_{eval}$ value already defined and using $\alpha_S$ equal to 0.75, a $\gamma_{G,eval}$ = 1.19 was obtained.

### 6.4.4 Live load partial factor

For the live loads, the procedure adopted to define the partial factor was defined by equation 15, in which equation 6 was integrated for two accumulated levels, 95% and 99.5%, according to the definition of partial live load factor (FERRY-BORGES & CASTANHETA, 1971). Thus, by using the integration, $X_{95\%}$ equals 0.875 kN/m² and $X_{99.5\%}$ equals 1.14 kN/m², was obtained. With these values, the live loads partial factor for evaluation given by equation 15 was $\gamma_{Q,eval}$ = 1.14 / 0.875 = 1.30.

### 6.4.5 Wind load partial factor

As previously pointed out, no reduction was applied to the wind load partial factor. Thus, the design factor of 1.40 was adopted for $\gamma_{W,eval\,l}$.

**Table 9 – Design and evaluation parameters – Building "A"**

| Situation | Strength (MPa) | | Loads ($kN/m^2$) | | Wind (m/s) | Partial factor | | | | | $\gamma$ | $K_{MOD}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Occupancy | | | Strength | | Actions | | | | |
| | Conc | Steel | Q/S/C | A.S. | | Conc | Steel | Perm | Sobr | Vent | | |
| Design | 25.00 | 420.0 | 1.50 | 2.00 | 34.00 | 1.40 | 1.15 | 1.40 | 1.40 | 1.40 | 1.96 | 0.85 |
| Evaluation | 24.66 | 420.0 | 0.875 | 0.875 | 27.20 | 1.10 | 1.15 | 1.19 | 1.30 | 1.40 | 1.31 | Tab. 8 |

**Notes:**
Q/S/C – live load for rooms and kitchen;
A.S. – live load for service areas;
Conc – concrete;
Perm – related to permanent actions;
Sobr – related to live loads of use and occupancy;
Vent – related to actions due to wind.

**6.4.6 Global safety coefficient in the evaluation**

From previous considerations, the global safety coefficient was determined and was to be considered in case the necessity of intervention in the columns arose, in compliance with the conditions of building "A". According to the conditions and the available information for the reduction of design uncertainties and using the reliability index ($\beta_{eval}$) already informed, the global safety coefficient during the structural repair of columns given by equation 18, was $\gamma_{eval}$ = 1.31.

### *6.5 Summary of adopted parameters in the evaluation*

From considerations made for building "A", based on the theoretical-experimental research and on inspections, the influential parameters and the values of loads and resistances were lifted for the evaluation. Table 9 shows a comparative view between design and evaluation values.

## 7. Results and Discussion

With the necessary considerations made for load effects and resistances of the cross sections in the columns of the building being studied, using the calculation program, many results were obtained and some partial conclusions were attained regarding loads and resistances.
An increase in the permanent loads was verified, which was produced by a fraction of the weight of the structural elements.

**Table 10 – Factored load effects with factors defined to intervention**

| Columns | Factored load effects | | | |
|---|---|---|---|---|
| | *Total loads on X axis (kN) | **Total loads on Y axis (kN) | Moment on X (kN.m) | Moment on Y (kN.m) |
| P1 | 3,819.4 | 3,593.0 | 17.4 | 140.4 |
| P2 | 8,449.7 | 7,441.5 | 902.1 | 59.9 |
| P3 | 4,978.6 | 4,531.9 | 26.7 | 455.0 |
| P4 | 7,243.3 | 7,878.9 | 880.3 | 44.4 |
| P5 | 15,072.8 | 13,968.3 | 577.8 | 608.5 |
| P6 | 6,825.0 | 6,607.3 | 87.5 | 591.1 |
| P7 | 5,701.3 | 6,298.0 | 53.2 | 531.4 |
| P8 | 5,837.6 | 6,012.4 | 575.1 | 34.6 |
| P9 | 5,346.4 | 6,119.8 | 35.3 | 390.4 |
| P10 | 6,397.4 | 6,552.1 | 126.6 | 91.0 |
| P11 | 7,600.2 | 8,044.9 | 67.8 | 462.1 |
| P12 | 2,973.2 | 2,909.8 | 17.6 | 79.2 |
| P13 | 7,297.2 | 6,238.1 | 46.2 | 1,059.5 |
| P14 | 4,877.9 | 4,892.1 | 26.2 | 155.8 |

**Notes:**
* dead load+live load+wind in X (factored) = 1.19G1 + 1.19G2 + 0.7x1.3Q + 1.4Vx
** dead load+live load+wind in Y (factored) = 1.19G1 + 1.19G2 + 0.7x1.3Q + 1.4Vy

The inspection demonstrates the there were larger dimensions for sections than those used in the project itself. In general, this increase was around 5%. The total load was 34,987.9 kN, calculated for the design data, and 36,770.11 kN, obtained from the evaluation information.
A decrease was verified in the axial live load value per column, around 40% in average. The same was verified regarding the vertical actions caused by the wind. In the case of bending moments, that reduction was around 37% in average for X axis, as well as for Y axis.
Table 10 presents the values of the axial loads and evaluation moments.
Table 11 presents the calculations of the resistances on sections of the garage columns considered for design sake and the resistances obtained in the tests and procedures of the evaluation. The latter values represent the real values for repair intervention instance.
After the procedures, the calculation of the garage columns was done within the evaluation parameters to determine the equivalent axial load effect. Such calculations were obtained by a calculation program, which had its sections returned to the characteristics described in Table 12.
The equivalent axial load effect, as already pointed out, acts in the section for the occasion of a structural repair, taken into account all particularities considered previously. Thus, this artifice was used to obtain the dummy axial load that produced equal load effects to bending moments and axial loads for evaluation. In this way, it was considered that the bars of the columns are symmetrical in all of the cases and the neutral axis is out of the cross section and the ultimate limit state was reached.
The comparison between the resistance of the section, obtained by inspection data (Table 11), and the equivalent load effects (Table 12) for each column, are the basis for the state limit equation, according to what equation 1 prescribes, should be preserved. In these conditions, when the unit is exceeded by the application of equation 1, it means that there is the possibility for extraction of deteriorated concrete, taken into account the types of interventions proposed. The computed remaining load was converted into a concrete area to be extracted. Once the thickness of the cut and stress of the concrete layer is known, the length of the cut is computed. The results of the analysis are shown in Table 13.
From the results of Table 13, it is noticed that according to the types of interventions proposed, for 36% of the columns, the intervention type to be used will be I1, for 7%, type I2 for 21%, the type I3 and 36%, type I4. Figure 5 represents the referred percentages.
Since the application of the methodology proposed in this research was a simulation, a possible loss of cover layer was not considered in the resistance of the sections of columns. In cases of real repair, the detection of cracking or spalling of the concrete cover layer, due to the steel corrosion, the total or partial area of the concrete could be disregarded. In the other two buildings where the methodology was applied, there also was no spalling detected on the concrete cover.

**Table 11 – Axial loads resisted by columns in the garage**

| Columns | Computed data from existent section obtained in inspections and tests | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Dimensions a (cm) | h (cm) | Cross-section areas *Steel ($cm^2$) | **Concrete ($cm^2$) | ***$f_{cd,eval}$ kN/$cm^2$ | Load resisted by concrete (kN) | Load resisted by steel (kN) | Total axial load capacity (kN) |
| P1 | 20.7 | 120.8 | 101.42 | 2,393.80 | 2.1297 | 5,098.14 | 3,704.11 | 8,802.25 |
| P2 | 31.1 | 150.5 | 214.40 | 4,454.86 | 2.1297 | 9,487.64 | 7,830.38 | 17,318.02 |
| P3 | 20.7 | 120.8 | 101.42 | 2,393.80 | 1.5973 | 3,823.61 | 3,704.11 | 7,527.71 |
| P4 | 51.0 | 81.4 | 149.15 | 3,994.40 | 1.5973 | 6,380.24 | 5,447.22 | 11,827.45 |
| P5 | 72.8 | 102.7 | 270.33 | 7,192.00 | 1.5973 | 11,487.74 | 9,873.08 | 21,360.83 |
| P6 | 31.1 | 120.8 | 149.15 | 3,599.88 | 1.5973 | 5,750.07 | 5,447.22 | 11,197.29 |
| P7 | 31.1 | 120.8 | 131.25 | 3,618.72 | 2.1297 | 7,706.89 | 4,793.55 | 12,500.44 |
| P8 | 31.1 | 120.8 | 131.25 | 3,618.72 | 2.1297 | 7,706.89 | 4,793.55 | 12,500.44 |
| P9 | 20.7 | 150.5 | 131.25 | 2,977.19 | 1.5973 | 4,755.45 | 4,793.55 | 9,549.00 |
| P10 | 54.5 | 54.5 | 95.46 | 2,869.77 | 1.5973 | 4,583.87 | 3,486.22 | 8,070.09 |
| P11 | 31.1 | 120.8 | 149.15 | 3,599.88 | 1.5973 | 5,750.07 | 5,447.22 | 11,197.29 |
| P12 | 20.7 | 81.4 | 53.46 | 1,628.71 | 1.5973 | 2,601.53 | 1,952.28 | 4,553.82 |
| P13 | 20.7 | 193.6 | 143.18 | 3,856.80 | 1.5973 | 6,160.45 | 5,229.33 | 11,389.78 |
| P14 | 31.1 | 120.7 | 61.09 | 3,689.46 | 1.5973 | 5,893.16 | 2,231.18 | 8,124.34 |

**Notes:**
*The steel contribution for the resisted load was considered by the number of bars according to the structural project with a 2‰ strain, thus the compressive strength was 420 MPa. The reduction of 5% in the steel cross section area was admitted due to the possible damage by corrosion.
** The gross area of the section ($A_c$ – $A_s$).
*** The concrete strength is given by $K_{MOD,eval} \cdot f_{ck,est/\gamma c,eval}$

**Table 12 – Equivalent axial load effects. - Building "A"**

| Columns | Computed with evaluation data | | | | | | | Equivalent axial load effect (kN) |
|---|---|---|---|---|---|---|---|---|
| | Steel | | | | Concrete | | | |
| | Bars quantity | Bar diameter (mm) | Steel cross section ($cm^2$) | Load fraction supported by steel (kN) | Concrete cross section ($cm^2$) | $f_{cd,eval}$ ($kN/cm^2$) | Load fraction supported by concrete (kN) | |
| P1 | 46.0 | 10.0 | 36.11 | 1,516.62 | 2,464.45 | 2.1297 | 5,248.61 | 6,765.23 |
| P2 | 96.0 | 12.5 | 117.75 | 4,945.50 | 4,562.80 | 2.1297 | 9,717.52 | 14,663.02 |
| P3 | 74.0 | 12.5 | 90.77 | 3,812.16 | 2,409.79 | 1.5973 | 3,849.15 | 7,661.31 |
| P4 | 40.0 | 20.0 | 125.60 | 5,275.20 | 4,025.80 | 1.5973 | 6,430.39 | 11,705.59 |
| P5 | 58.0 | 20.0 | 182.12 | 7,649.04 | 7,294.44 | 1.5973 | 11,651.38 | 19,300.42 |
| P6 | 56.0 | 16.0 | 112.54 | 4,726.58 | 3,644.34 | 1.5973 | 5821.09 | 10,547.67 |
| P7 | 66.0 | 12.5 | 80.95 | 3,400.03 | 3,675.93 | 2.1297 | 7,828.72 | 11,228.75 |
| P8 | 80.0 | 10.0 | 62.80 | 2,637.60 | 3,694.08 | 2.1297 | 7,867.38 | 10,504.98 |
| P9 | 58.0 | 16.0 | 116.56 | 4,895.39 | 2,998.79 | 1.5973 | 4,789.96 | 9,685.34 |
| P10 | 44.0 | 16.0 | 88.42 | 3,713.74 | 2,881.83 | 1.5973 | 4,603.13 | 8,316.87 |
| P11 | 48.0 | 16.0 | 96.46 | 4,051.35 | 3,660.42 | 1.5973 | 5,846.77 | 9,898.12 |
| P12 | 50.0 | 10.0 | 39.25 | 1,648.50 | 1,645.73 | 1.5973 | 2,628.72 | 4,277.22 |
| P13 | 128.0 | 10.0 | 100.48 | 4,220.16 | 3,907.04 | 1.5973 | 6,240.70 | 10,460.86 |
| P14 | 62.0 | 10.0 | 48.67 | 2,044.14 | 3,705.10 | 1.5973 | 5,918.14 | 7,962.28 |

**Table 13 – Proposal intervention type - building "A"**

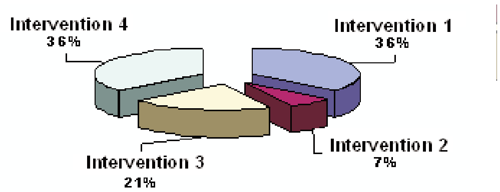
| Columns | Equivalent axial load effect $S_{d,eval}$ (kN) | Total load capacity $R_{d,eval}$ (kN) | Rd,eval - $S_{d,eval}$ (kN) | Ratio $R_{d,eval}$/ $S_{d,eval}$ | *Length of concrete to be cut (cm) | **Proposal intervention type |
|---|---|---|---|---|---|---|
| P1 | 6,765.23 | 8,802.25 | 2,037.02 | **1.30** | **191.0** | I1 |
| P2 | 14,663.02 | 17,318.02 | 2,655.00 | **1.18** | **249.0** | I1 |
| P3 | 7,661.31 | 7,527.71 | -133.60 | **0.98** | **-17.0** | I4 |
| P4 | 11,705.59 | 11,827.45 | 121.86 | **1.01** | **15.0** | I4 |
| P5 | 19,300.42 | 21,360.83 | 2,060.41 | **1.11** | **258.0** | I1 |
| P6 | 10,547.67 | 11,197.29 | 649.62 | **1.06** | **81.0** | I3 |
| P7 | 11,228.75 | 12,500.44 | 1,271.69 | **1.11** | **119.0** | I2 |
| P8 | 10,504.98 | 12,500.44 | 1,995.46 | **1.19** | **187.0** | I1 |
| P9 | 9,685.34 | 9,549.00 | -136.34 | **0.99** | **-17.0** | I4 |
| P10 | 8,316.87 | 8,070.09 | -246.78 | **0.97** | **-31.0** | I4 |
| P11 | 9,898.12 | 11,197.29 | 1,299.16 | **1.13** | **163.0** | I1 |
| P12 | 4,277.22 | 4,553.82 | 276.60 | **1.06** | **35.0** | I3 |
| P13 | 10,460.86 | 11,389.78 | 928.92 | **1.09** | **116.0** | I3 |
| P14 | 7,962.28 | 8,124.34 | 162.06 | **1.02** | **20.0** | I4 |

**Notes:**

* **Length of concrete to be cut:** $L = \dfrac{R_{d,eval} - S_{d,eval}}{5 \times f_{cd,eval}}$

**Intervention type (Figure 3):
I1 – Intervention with concrete cut on two sides – one bigger side and another smaller side;
I2 – Intervention with concrete cut on a larger side;
I3 – Intervention with concrete cut on half of the larger side;
I4 – Intervention with concrete cut on half of the larger side using appropriate shoring.

Figure 5 – Proportion of interventions type




## 8. Conclusions

This methodology represents an important step for the success of the services made during the repair of columns of buildings regarding the safety level for the involved structural elements. It appears as an alternative for the actual practice that uses simplified and empiric criteria for the problem. Thus, the necessary study for the development of a methodology that approaches the theme of the evaluation of existent buildings was made, according to similar guidelines used in design. On the benchmark of the evaluation methods, it represents a progress towards probabilistic methods, which will be widespread in the future.

This paper presents the development of a general methodology for the treatment of the random parameters involved in the load effect mechanisms and structural response for existing buildings, affected by pathologies in columns, taking into account the short time necessary to the repair services and a probability of an acceptable failure. Considerations were made that aim to obtain more accurate values for the permanent loads that allow refinement of the partial factor for this type of action. The live loads were established for reduced periods of time and the actions, due to the adjusted wind strength according to the evaluation. The partial load factor of the variable actions was also established according to semi-probabilistic criteria and the obtained information.

The existing concrete strength in the considered age, according to the action of permanent loads over time, was still considered. The partial factors of strength were adjusted according to a knowledge degree of the properties in the structures.

By using techniques of structural reliability, the method can be considered innovative, which will allow it to develop together with the calculation methods. Thus, this methodology fills in a gap generated by the absence of safety quantification during the execution of structural repairs.

The feasibility of the proposed method was checked in the example of an existing building. In the example, some of the analyzed columns had the global safety coefficient a little below that permitted $\gamma_{eval}$. That coefficient just demonstrates, for the evaluation situation, that the repair activity should be done with special care not to expose the integrity of the element in excess.

In the application of the method, different values were observed for the random parameters of evaluation and design. The partial factors for actions and resistances were also changed.

For the live loads, an only value of 0.875 kN/m$^2$ for all the compartments of the construction was obtained, considering the necessary period for the intervention. The basic wind velocity of the evaluation could be reduced to 80% of that admitted in design. This produced a considerable decrease of the moments and maximum vertical loads attributed to that load effect type. The measurements of cross sections of structural elements contributed for an increase of the permanent loads around 5%.

The partial factors were influenced by a better level of acquired knowledge of the analyzed structure. The partial load factor of permanent actions could be reduced from 1.4 to 1.19. The partial live load factor, for the building, was susceptible at the value of 1.30. On the other hand, the partial factor, due to actions produced by the wind loads, didn't present reduction in comparison to the design factor.

Due to the fact that no technological control of the execution was available, several tests in the concrete of the columns were made. It allowed the adoption of a concrete strength partial factor as low as 1.10. The reinforcement, without test results, was considered in the evaluation by the design factor.

The inspection information influenced the safety global coefficient of the studied columns. The same fact influenced the intervention type adopted in each situation. In design, the columns are protected against collapse by a $\gamma$ in the range of 1.7 to 2.0 depending on the used code. Thus, building "A", was submitted to a good number of tests, admitted a $\gamma_{eval}$ =1.31 to be followed during an intervention.

Although the tests executed in building "A" resulted in a reduced $\gamma_{eval}$, it is noticeable that an intervention in some columns is critical. This is due, maybe, to the structural calculation, because this building is relatively slender. Out of the 14 existent columns in the building, 5 of them would demand intervention type I4 for which shoring would be necessary. However, 5 columns could be repaired by intervention I1, having the concrete cut in two faces (one larger and the other smaller) without shoring, which would increase the speed of the service a little. Out of the 4 remaining columns, 1 would admit intervention type I2 and the other 3, intervention type I3.

From what was presented in this paper, one can conclude that the proposed methodology is consistent, because it contemplates most of the problems generated in a structural repair of columns. It is also practical when using procedures and tools that are already used by the professionals of the calculation field and structural repair.

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