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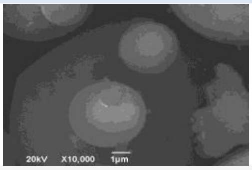

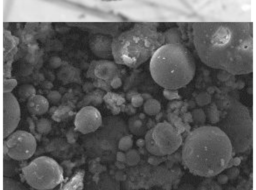

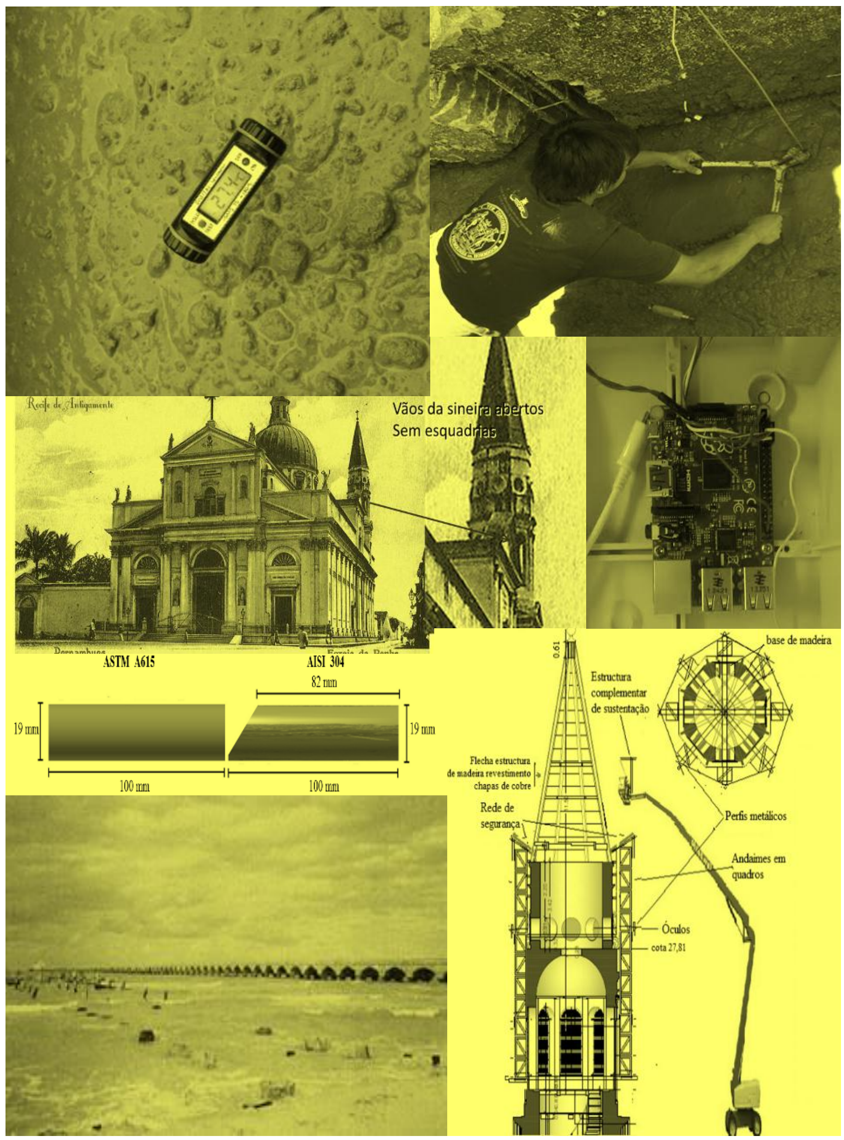

Journal of the Latin-American Association of Quality Control, Pathology and Recovery of Construction

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RAV6N3, September - December 2016

Message from the Editor-in-chief

LATIN AMERICAN JOURNAL ON QUALITY CONTROL, PATHOLOGY, AND THE RECUPERATION OF CONSTRUCTION

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It is gratifying for the team of the ALCONPAT Journal to see the second issue of our sixth year published.

The purpose of the ALCONPAT (RA) Journal is to publish case studies related to the topics of our association, such as quality control, pathology, and the recuperation of constructions, all the while motivating the presentation of basic or applied researches, revisions, or documental researches.

This edition V6N3 begins with a work from **Brazil**, in which Carlos Welligton Pires Sobrinho y Antonio Carlos Costa, they talk about the effects of applying badly designed repair techniques in 1981, which, together with the absence of preventive maintenance, leaks and even the growth of shrubs embedded in the masonry, led to the instability of the bell towers of the Basilica of the Church of Penha The authors present and discuss the historical, current situation, techniques and original strategies used in the development of structural reinforcement design of both bell towers of the Basilica of the Penha Church.

The second work, from **Argentina**, by Jorge Daniel Sota et. al., present a work on the determination of the degree of maturity of concrete in situ in a structure. Their results allow to determine the curve of maturity of the concrete studied and to establish the degree of maturity in each of the differentiated parts of the structure. The use of this methodology and equipment allows to control the totality of the received concrete, its homogeneity and to monitor its resistance in real time.

In the third article, from **Brazil**, Gustavo Macioski et. al., they analyze how the steel type, pH of the medium and the surface protection of the steel bar can change the electrochemical properties of this metal. For this, they apply the linear polarization resistance technique on steel bars to evaluate the corrosion of the samples. The study evaluated the CA-50, CA-60 and CP-175-RB steels, with or without surface protection. From the results it is possible to observe how the three variables influence the results of the current density and corrosion rate.

The fourth article by Jennifer A. Canul et al., is from **Mexico**. The authors analyze the concrete made with crushed limestone of high absorption (ACTAA) from Yucatan, Mexico, which is considered of low quality. Their results indicated that the CV can be used in concrete with ACTAA as a fine inert aggregate since it manages to maintain a compressive strength like the reference. Equations are presented for the prediction of mechanical properties.

The fifth work in this issue is from **Mexico**, by Trinidad Pére et al., the influence of Inconel 182 as buttering material in the mechanical properties of dissimilar metal welds between plain carbon steel and stainless-steel bars welded using SMAW has been investigated by using microstructural analysis, very common in field welding for construction. Their results indicated that even if the joints contain defects generated by the welder, the mechanical properties of dissimilar welded joint without buttering are higher than the properties of joints with buttering.

The sixth article, of documentary research, is from **Brazil**, in which B. Fernandes et al. contribute to contribute to the analysis of non-conformity concrete, focusing on long time effects. A review of compressive strength evolution, results variability and acceptance criteria was made. In addition, it is presented a case study of a nonconforming concrete used in composite structures (concrete-filled steel columns) that present 28 days strength below the specified. This analysis, associated with a revision of the structural design and a carefully assessment, could help decision taking in case of non-conformity concretes.

The seventh work is by Yolanda Hernández et. al, from **Venezuela**, they investigate an empirical correlation between the rebar corrosion rate and the corrosion-induced crack width propagation rate produced on beam's concrete cover, with or without load application to these beams. The results showed a direct relation between crack width propagation and rebar corrosion rate, showing wider cracks in the loaded beams.

The closing article is by Diego Jesus de Souza et al., from **Brazil**, they evaluate the possibility of negotiating corporate real estate, considering the expectations of users of real estate companies. Thus, the purchase opportunities and its alternatives were evaluated: the purchase of property, short-term rentals, long-term rental, custom construction and durability. The result of the analysis is the recommendation of possible choices between the shown alternatives, starting from the one that best meets the criteria prioritized by stakeholders.

The ALCONPAT journal recently was described by CONACYT as competent at international level, obtaining a higher score than other journals of its classification in Q4 and Q3 of Scopus and ISI.

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By the Editorial Board

Pedro Castro Borges
Editor-in-chief

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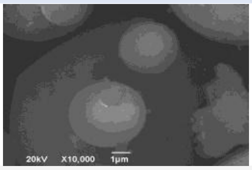

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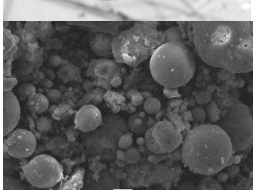

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Journal of the Latin-American Association of Quality Control, Pathology and Recovery of Construction

History, situation and reinforcement of the bell towers of the basilica of Penha-Recife-Brazil

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ABSTRACT

This paper presents and discusses the history, current situation, original techniques and strategies used in the development of structural reinforcement design of both towers of the Basilica of Penha Church. Repair techniques poorly designed, conducted in 1981, along with lack of preventive maintenance, leaks and even the growth of bushes embedded in the masonry led to the instability of the towers of the Basilica of Penha Church.

Keywords: reinforcing masonry; historic monuments; reinforcement techniques; execution strategies; carbon fibers.

RESUMO

Este artigo apresenta e discute a história, situação atual, técnicas e estratégias utilizadas no reforço estrutural desenvolvimento design original de ambas as torres da Basílica da igreja de Penha. Mal concebido técnicas de reparo, realizada em 1981, juntamente com a falta de manutenção preventiva, vazamentos e até mesmo o crescimento de arbustos embutidas na alvenaria levou à instabilidade das torres da Basílica da igreja de Penha.

Palavras chave: reforço em alvenarias; monumentos históricos; técnicas de reforço; estratégias de execução; fibras de carbono.

RESUMEN

En este trabajo se presentan y discuten el histórico, situación actual, técnicas y estrategias originales empleadas en el desarrollo del diseño de refuezo estructural de ambos campanarios de la Basílica de la Iglesia de Penha. Técnicas de reparación mal diseñadas, llevadas a cabo en 1981, junto con ausencia de mantenimiento preventivo, filtraciones e incluso el crecimiento de arbustos incrustados en la mampostería, llevaron a la inestabilidad de los campanarios de la Basílica de la Iglesia de Penha.

Palabras clave: fortalecimiento de albañilería; monumentos históricos; técnicas de refuerzo; estrategias de implementación; fibras de carbono.

Autor de contacto: Carlos Welligton Pires Sobrinho (carlos@itep.br)

1. INTRODUCTION

The Basilica of Our Lady of Penha, Order of Capuchin Friars Minor, is an imposing building in the urban landscape in the district of San Jose - strongly marked by the presence of towering tall towers and a huge dome with cruise, symbols a strong religiosity - Urban environment of the initial formation of the city of Recife. Between the beginning of construction (1656) and the completion of his work, which spent more than 200 years, due to the expulsion of French Calvinists from Recife, by order of the Portuguese court (CECI, 2014).

Figure 1 shows the historical record of the early 20th century building. The building has a central nave and a pair of towers (Epistolary and Belfry), each with eight columns called "minarets". In these columns a number of cracks and detachments of coatings were observed, due to the oxidation of the internal reinforcement, as well as pathologies due to the action of shrubs that grew on the outside and branched into the masonry of one of the towers.

In 1981 reinforcement interventions and filling the windows with cobogos (Araujo 2010) were made, in the reinforcements were identified with the inclusion of some columns and bars of steel in elements of reinforced concrete.

In 2010, due to the aggravation of pathological manifestations in the openings fissures in some columns and loss of internal and external lining, reinforcements in wood structure were inserted in the windows between 8 columns of towers of structure of masonry.

Figure 1. Historical image of the Penha-detail Basilica of the bell tower without cobogo.

Figures 2 and 3 show of one of the towers as well as details of the type of reinforcement used in the columns the towers.

Figure 2. Current view of the tower

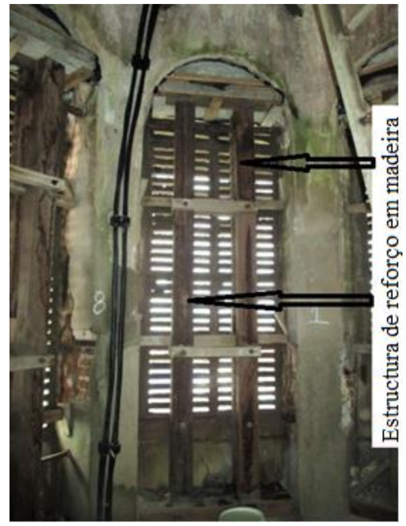


Figure 3. Reinforcement in tower columns

In Figures 4 to 6 it is possible to observe aspects of large cracks in columns generated by the oxidation of the reinforcements incorporated.

Figures 4 to 6. Fissures caused by the oxidation of the iron clamps inserted in the columns

2. INVESTIGATIONS

Aiming to support the reinforcement project for the two towers, activities were carried out to characterize the compressive behavior in samples taken from the building and a numerical analysis to determine the actions that work in the towers.

2.1 Physico-mechanical characterization of the building

The physical and mechanical characteristics of the building were obtained through inspection by prospecting in areas in the region of the tower of the Epistle, through drill and double disk cutter, with the samples sent to the laboratory of the ITEP-Institute of Technology of Pernambuco.

Attempts to obtain samples using a 4" diamond drill bit were not efficient, since the need to cut with hydraulic lubrication favored the solubilization of the mortar and the brick itself, since both the lime-based mortar and the Not completely burned brick suffered with action of disk movements and water action. In order to obtain samples of the masonry, it was necessary to use a double diamond disc cutter, as shown in figures 7 and 8.

Figure 7. Sample removal process using double diamond disc.

Figure 8. Sample removed from masonry sent to the laboratory.

Samples were cut and rigged in four specimens for achieving the compressive behavior tests, being used press with displacement control, with capacity of 300kN, allowing register the post rupture behavior.
For the determination of the longitudinal and transverse modulus, deflectometers with precision of thousandths of a millimeter were installed in the cross section of load application and LVDts were used in the measurement of longitudinal and transverse displacements. The composition of Figures 9a-d shows characteristics of the tests on the compressive behavior of the samples.

a) sample 1 b) sample 2 c) sample 3 d) sample 4

Figures 9a-d. Compressive behavior of the assays in the samples

From this evaluation, on analysis of determination of the characteristic resistance, according to the recommendations of ABNT NBR 15182-2 was obtained

$$f_{pk} = 1{,}15\ MPa$$

2.2 Active stress

The tensile stresses were obtained based on numerical modeling based on the finite element method (Mamaghani, 2004). The masonry structure was modeled with solid elements of various shapes, the floors in plate elements combined with membrane elements and the arrow covering the towers in bark elements. The SAP2000 computational system was used to obtain tensions, and the densities and characteristics of the compressive behavior (modulus of elasticity and Poisson's coefficient) were reported.
Figure 10 shows aspects of the solid elements used and the results of the tensions in the various elements that make up one of the towers.

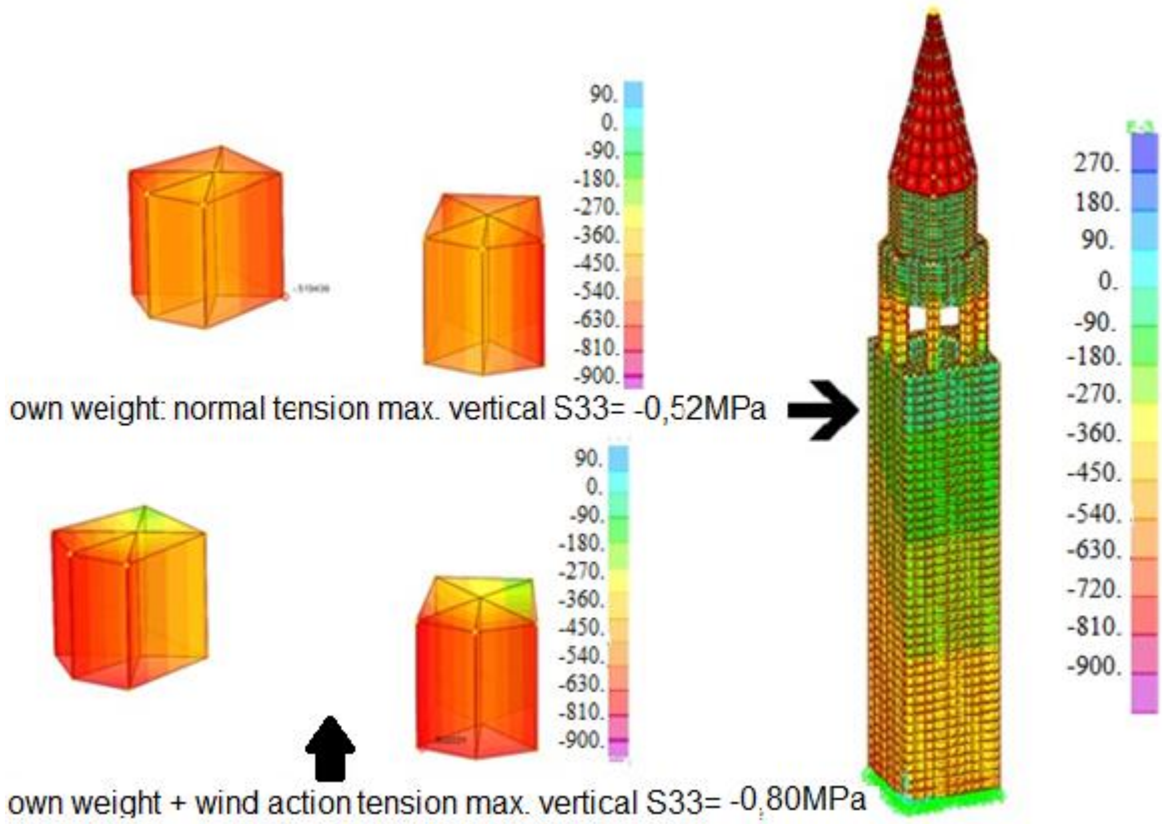


Figure 10. Results of the numerical analysis of the tower of the Basilica

The images in figure 10 show that the most critical regions of stress concentration are located at the base of the columns, reaching a value of 0.52MPa due only to its own weight and of 0.80MPa when considering the combined action of weight and wind action.

2.3 Structural security analysis

Considering the results of the requesting tensions, especially in the region near the bases of the columns, reaching maximum values were between 0.52MPa to 0,80MPa and considering that the characteristic resistance of the samples was determined at 0.63MPa.

These values show that in the performance of the wind the tensions surpass the resistant capacity of the columns, even without considering the safety factors, normally existing when proceeding to a dimensioning. In this way the temporary reinforcement structures, built with wooden structures in the windows of the towers are acting decisively, avoiding collapse in this region.

The results of these analyzes are very coherent with the situation that presents the columns of the tower of Basilica, presenting a high state of cracking and indicative of localized ruin.

Thus, it is concluded that it is extremely necessary to use reinforcement that allows to raise at least twice the resistant capacity, thus meeting the normative principles of structural safety.

2. STRUCTURAL REINFORCEMENT PROJECT

3.1 Principles of reinforcement design

Composite systems structured with carbon fibers are efficient for the absorption of tensile stresses, avoiding, through the confinement of the section of the axially required pieces, the growth of the transversal deformation of the materials, resulting from the action of the axial load.

The effect of the confinement pressure is to induce a triaxial stress state in the masonry and under these conditions masonry, or other fragile material, substantially alters its compressive behavior, both in strength and ductility (Fiorelli, 2002).
Figures 11 and 12 show the difference in compressive behavior of a concrete element, which could be masonry, without and with transverse confinement.

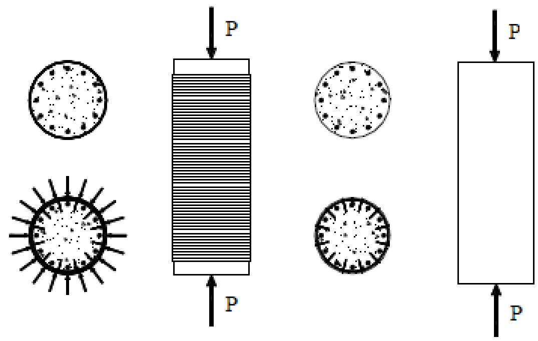


Figure 11. Tensions and strains in confined and unconfined systems

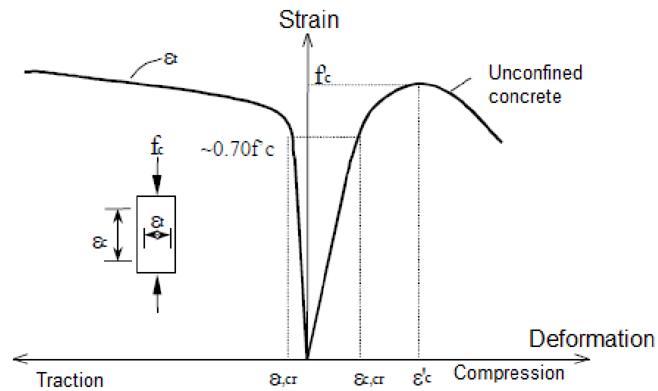


Figure 12. Typical configuration of a piece of concrete confined and unconfined

In addition to the effect of confinement promoted by the use of a carbon fiber and epoxy resin system, the lime-based mortar coating will be replaced with cement-based polymer mortar based coatings and chemical additives.

3.2 Determination of the influence of reinforcement
The compressive characteristics of polymer mortar in relation to the lime mortar are substantially larger, being able to overcome the compressive strength by 15 times and the value of the longitudinal modulus of elasticity by more than 35 times, see table 1.

Table 1. Mechanical characteristics of materials.

Materials	**Compressive strength (MPa)**	**Elasticity module (GPa)**	**References**
Masonry bricks and lime mortar	2,0	0,40	Ensaios em amostras (ITEP)
Polymer mortar	30,0	15,0	Product characteristics (Viapol,2015)

The effect of the confinement, promoted with the use of a system composed of carbon fibers and epoxy resin, can take up to 30% the resistant capacity of a compressed part.
Thus combining the effects of coating replacement with confinement on the outer cross sections of the columns provides an increase in the resistive capacity of these elements that make up the towers.
Figures 13 and 14 show the positioning of the columns that present the most critical situations in terms of stress concentration.

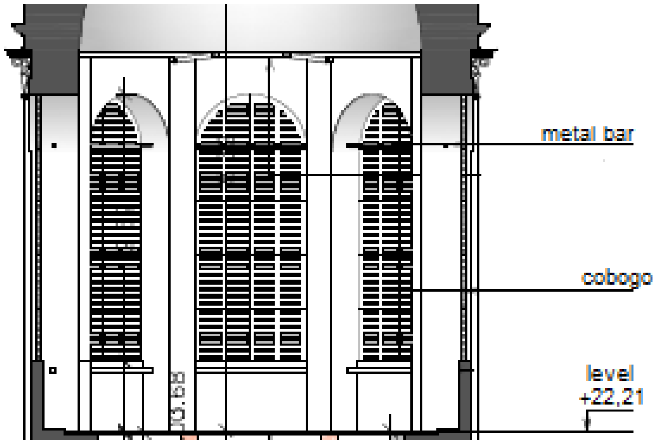


Figure 13. Drum region

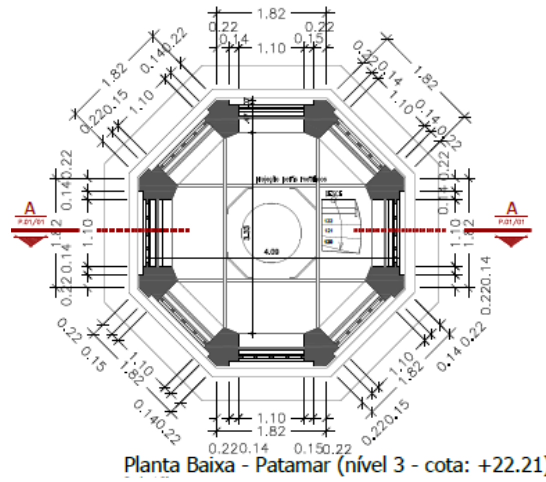
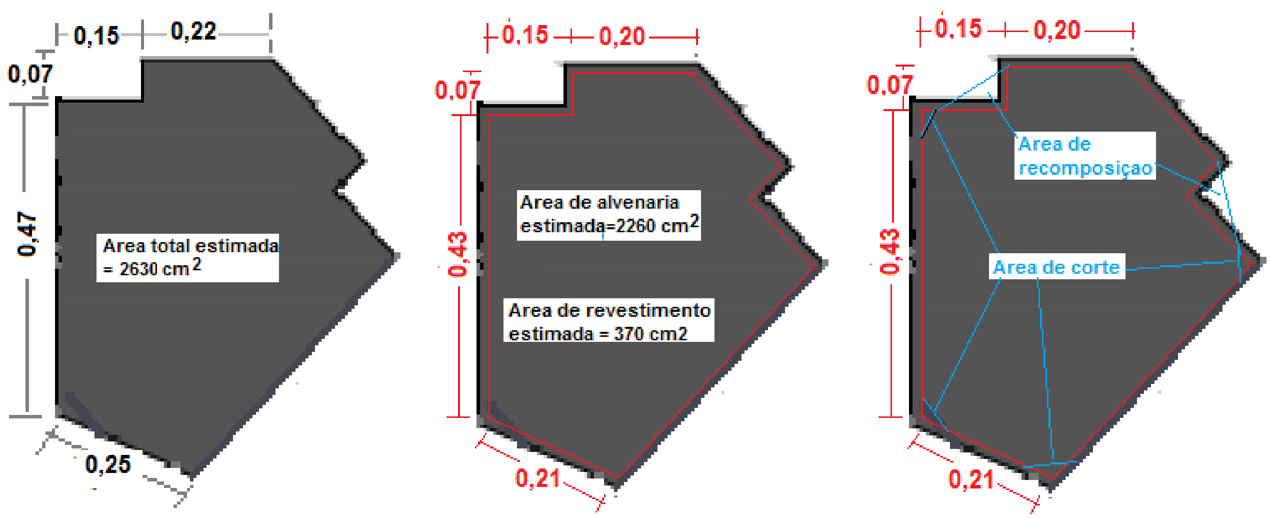


Figure 14. Low plant at the base of the columns

The evaluation of the cross-sectional areas of one of the columns to be reinforced is considered in figure 15.

Practically after the cutting interventions (corners roughing) and rigging, necessary to enable cross-sectional casting of the columns, the estimated areas of masonry and cladding do not change.



a) cross section of a column b) masonry sections and coating c) treatment and recomposition of areas

Figure 15. Evaluation of areas after interventions required for treatment to strengthen

The evaluation of the active and resistant loads at the base of columns can be estimated in:

a) Total active load, due to its own weight, considering the numerical modeling

$$\mathbf{S_{pp}=5{,}20*2630=13{,}676\ ton}$$

b) Total active load, due to the combined action of own weight and wind

$$\mathbf{Spp=8{,}00*2630=21{,}040\ ton}$$

c) Resistance of the current masonry, considering the characteristic resistance obtained in the test.

Ra= 6,30 * 2630 =16.569 ton

d) Estimated strength for reinforced masonry with replacement of lime mortar coating by polymer mortar

Rr1=6,30*2260 + 300*370 = 127,569 ton

e) Estimated strength for reinforcement with the use of the carbon fiber belt

Rr_2= 127,569 * 1,20 = 153,082 ton

In this way, it can be considered that the proposed reinforcement allows a resistance increase of the columns in 7 times its resistant capacity and if compared to the load acting on the base of the columns due to combined action of own weight and wind action. In this way, the proposed reinforcement presents a security coefficient of the order of 7.0, well above the 2.0 recommended by masonry standards.

4. REINFORCEMENT PROCEDURES

Following are the procedures for implementing structural reinforcement.

4.1 Basic procedures for strengthening masonry structures.

The basic procedures to be followed for execution of reinforcement in masonry:

4.1.1 Demolition and removal of the existing coating, with thinning of the corners and cleaning of the areas to be reinforced

In the areas to be reinforced, the existing coatings must be removed, using corodur grinding wheel coupled to a rotating sander, in this way there will be no significant impacts on the masonry structure, as shown in figure 16a.

After skirting the corners and removing the entire coating, the areas should be clean and free of dust and can be used with slightly moist air, as shown in the diagram of figure 16b.

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Figure 16a. Roughing process with grinding wheel

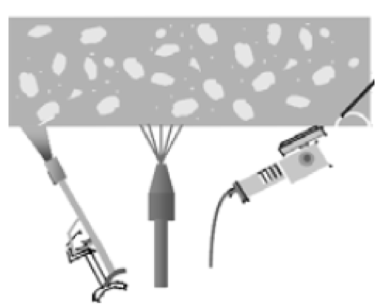

Figure 16b. blasting scheme and thinning of the corners

4.1.2 Surface preparation and rigging

In the areas that will receive reinforcement, outlined by the project, they must prepare the surfaces receiving application of primer to make it possible to fill the voids to receive the layer of polymerizing mortar, as indicated in figures 17a and 17b.
This mortar should have adequate consistency with its application with a metal trowel on the primed surface.
The final surface must be smooth and compact.
After 3 days of application of the mortar, a new primer coat is applied.

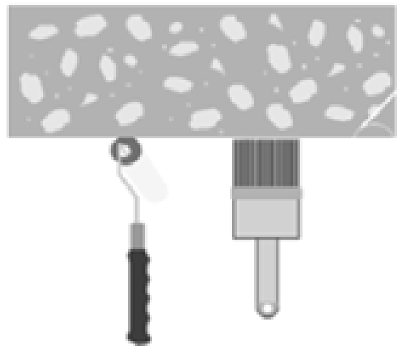

Figure 17a. Scheme of the preparation process

Figure 17b. Primer application as surface preparation

4.1.3 Procedure for applying carbon fiber blankets

The surface of the masonry already prepared an epoxy resin layer is applied with a roller. Typically, this product has a low viscosity - which facilitates its penetration into masonry. The function of this layer is to provide adequate adhesion to the surface of the structure (Grande 2011).
After application mass regularization is then applied an epoxy mass + carbonates to correct and eliminate surface defects that can detract from the application.
After perfect regularization, the first layer of resin is applied. The surface of the structure is covered with epoxy resin saturation. This resin, high viscosity, helps to maintain the CFC and the correct position. The impregnated saturation in the blanket being applied, it also helps in the efforts of the fibers and abrasion protection.
The application of the carbon fiber blankets cut into the size of the surfaces and the area geometry is applied to the epoxy resin saturation.
Continues with saturated resin application of the reinforced top layer are made throughout the entire area so that the system is hidden.
After all CFC layers have been applied, regularization with polymer composite and plastic fibers is promoted.
Figure 18 shows the procedures for the application of carbon fiber blankets.

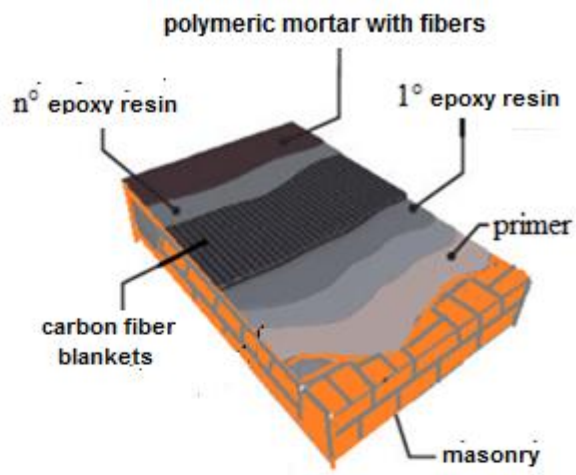

Figura 18. CFC System

4.1. Strategy for recovery of towers

Considering the critical situation of the elements that make up the towers, from the loss of coating of the copper pieces on the cover of the arrows, causing rotting of structural elements of this cover, from the degradation action of bushes of grew and branching in the drum of the tower to and the signs of ruin at the base of the columns. In order to proceed with the recovery of the towers, a strategy must be developed to carry out the reinforcement steps of the towers.

4.2.1 Procedure for cleaning and injection of cement paste into fissured columns

In the fissured columns there is a need for cleaning and filling with epoxy resin for recovery of the moniliticity, shown schematically in figure 19 [5].

A) Cleaning of cracks with compressed air blasting;

B) removal of parts of impregnated reinforcements inside the cracks, so as not to cause major damage;

C) Drilling of holes along the fissure and placement of masters, filling the outer side with cement grout tix;

D) Injection of cement paste, with low pressure, in the pugadores, from bottom up, in order to fill the fissures.

Figure 19. Injection process: placement of masters and injection through injection pump

4.2.2 Procedures for removing the copper plates from the cover of the arrows.

Preparation of an auxiliary structure to support a scaffolding platform: Insert into the bases of the wooden elements 4 transverse metal profiles, which supported the scaffolding lines.

A) Mount the 8 columns of scaffolding until reaching the edge of the base of the arrow, securing them against the walls of masonry;

B) Place a grid / net of protection on the scaffolds in order to avoid falling objects of the arrow;

C) With the aid of a boom crane coupled to a protective structure, remove the copper pieces from the cover of the arrows.

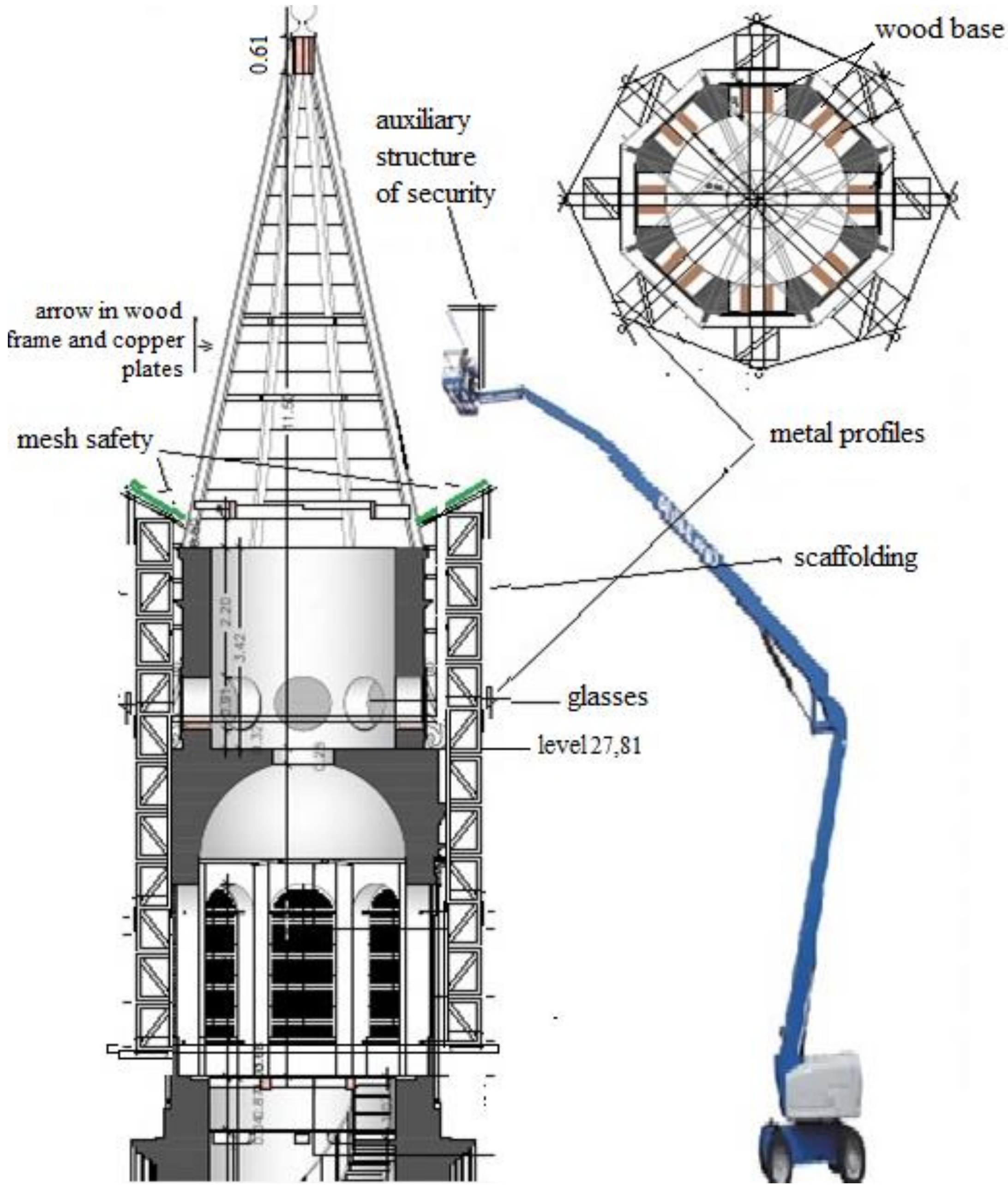

Figure 20. Scheme of the process

3.3 Procedure for reinforcing tower columns

The eight columns that make up each one of the towers, are currently with provisional reinforcement fomó by wooden structure and with closure in cobogo.

To promote reinforcement of these columns, in pairs and opposites, the reinforcement technique should be used with replacement of the existing coating and carbon fiber sheeting.
A) It starts soon after the treatment of fissures, according to 2.2.1.
B) It is promoted the cut of the area of cobogó surroundings of the pair of selected columns;
C) The coating and the thinning of the corners are removed, as described in 2.1.1;
D) The reinforcement is promoted as described in 2.1.2 and 2.1.3;
E) Repeat this procedure for other pairs of opposing columns.

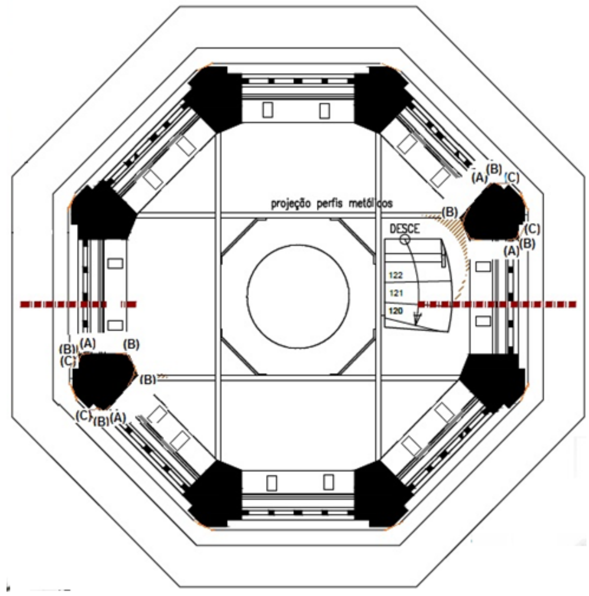

Figura 21. Bottom floor at level 22.21

4.2.4 Procedure for removal of the rooted shrubs in the tower of the Epistle and execution reinforcement of the wooden structure of the cover.
The shrubs that have grown and rooted in the tower of the Epistle need to be carefully removed.
After the removal of the coating of the arrows the investigation of the wood structure that composes the covering arrows of the towers is carried out.
A) Identify the damaged wood elements and promote their reinforcement;
B) The reinforcements can use steel plates and stainless steel screws, and it may be necessary to fit new pieces of wood.

Figure 22. Cover situation detail

4.2.5 Reinforcement procedure for the outer contour of the towers
In the outer contour regions of the towers, at levels 18.00, 25.00, 27.00, 29.00 and 31.00, reinforcements will be developed, according to the following steps and illustrated in figure 23:
A) Preparation of an auxiliary structure to support a scaffolding platform: Insert into the base of the columns wooden elements that will support 4 transverse metal profiles, which supported the scaffolding lines.
B) Mount the 8 columns of scaffolding until reaching the edge of the base of the arrow, securing them against the walls of masonry;
C) Place a grid / net of protection on the scaffolds in order to avoid falling objects of the arrow;
D) With the aid of a boom crane attached to a protective structure, remove the copper parts from the cover of the arrows.

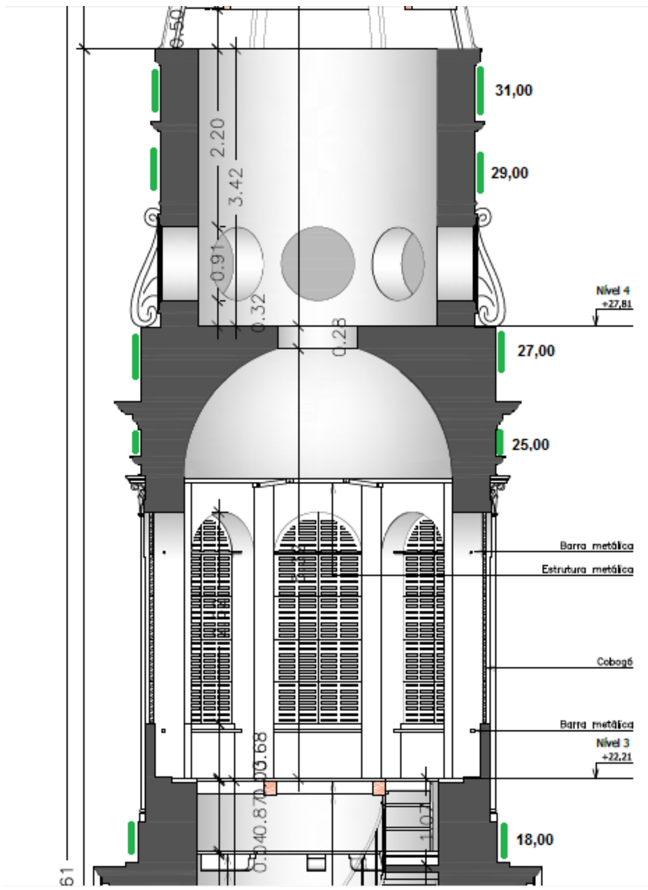

Figure 23. Reinforcing by grating the columns

5. FINAL CONSIDERATIONS

The situation of the towers of the Basilica of Penha shows signs of concern regarding structural stability. In the evaluation carried out there is no safety reserve, the temporary reinforcement implemented is acting in an effective and full, but new signs of ruin are on display. The proposal of reinforcement presented makes possible not only the removal of the provisional reinforcement the cobogó closing built after the construction of the basilica in the XVIII century. The use of reinforcement on the basis of replacement of lime mortar by polymer mortar and carbon fiber sheeting, may be covered by a new layer of mortar based on lime with reconstitution of architectural details and frescoes similar to the original ones.

The reinforcement in carbon fibers and polymer mortar does not suffer degradation with the humidity and natural weathering actions, are thus considered durable.

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Measure of maturity of the concrete structure

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ABSTRACT

In this paper, we determine the degree of maturity in situ concrete structure. To do this, temperatures are measured in the concrete foundations of a structure, from the first hours of hydration, to 28 days, with a device developed at the Faculty. Simultaneously testing compressive strength are performed establishing the relationship with temperature, with expressions Nurse-Saul and Arrhenius. The results allowed to determine the maturity curve of the studied concrete and establish the degree of maturity in each of the different parts of the structure.
The use of this methodology allows to control the entire concrete received, its homogeneity and to monitor resistance in real time.
Keywords: maturity; concrete; temperature; resistance.

RESUMEN

En este trabajo realizamos la determinación del grado de madurez del hormigón in situ en una estructura. Para ello, se miden las temperaturas en las bases de hormigón de una estructura, desde las primeras horas de la hidratación, hasta los 28 días, con un equipo desarrollado en la Facultad. Simultáneamente se realizan ensayos de resistencia a la compresión estableciendo la relación con las temperaturas, con las expresiones de Nurse-Saul y Arrhenius. Los resultados permitieron determinar la curva de madurez del hormigón estudiado y establecer el grado de madurez en cada una de las partes diferenciadas de la estructura.
El uso de esta metodología permite controlar la totalidad del hormigón recibido, su homogeneidad y monitorear su resistencia en tiempo real.
Palabras claves: madurez; hormigón; temperatura; resistencia.

RESUMO

Neste trabalho, determinar o grau de maturidade na estrutura de concreto in situ. Para fazer isso, as temperaturas são medidas nas fundações de betão de uma estrutura, desde as primeiras horas de hidratação, a 28 dias, com um dispositivo desenvolvido na Faculdade. Simultaneamente testar resistência à compressão são realizados estabelecer a relação com a temperatura, com expressões Nurse-Saul e Arrhenius. Os resultados obtidos permitem determinar a curva do betão estudada maturidade e estabelecer o grau de maturação em cada uma das diferentes partes da estrutura.
A utilização desta metodologia permite controlar todo o betão recebido, a sua homogeneidade e monitorar a resistência em tempo real.
Palavras-chave: maturidade; betão; temperatura; resistência.

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1. INTRODUCTION

The maturity method provides a simple and useful middle to estimate the strength gain of concrete in early ages (generally less than 14 days).
It should be mentioned that it is necessary to have the maturity curve of the dosage to be used, because the curve is typical of all materials used.
This method recognizes the combined effect of time and temperature, providing a basis for estimating the concrete strength development of "in situ" by controlling the temperature and time (Peter C. Taylor, Steven H. Kosmatka, Gerald F. Voigt, et al, 2007).
The effects of time and temperature in the increased concrete strength are quantified by a function of maturity, that is indicative of the level of resistance developed by the concrete. The two functions of maturity used for this purpose are for Nurse-Saul and Arrhenius (325.11R ACI-01, 2001).
The equation of Nurse-Saul, developed in the 50s and the most widely accepted to measure maturity, is the accumulated product of the time and temperature, equation 1.

$$M(t) = \sum(T_a - T_0)\,\Delta t \qquad (1)$$

where:
M (t) = maturity (temperature-time factor) at age t, in ° C.days or ° C.hours,
Δt = time interval, in days or hours,
T_a = concrete average temperature during interval Δt, en °C, and
T_o = reference temperature in ° C.

The reference temperature is the temperature at which ceases the strength gain of concrete; therefore, the periods during which the temperatures are at or below this temperature reference, do not contribute to resistance. Generally, a value of -10 ° C to the reference temperature in Nurse-Saul equation (325.11R ACI-01, 2001) is used.
The maturity can also be determined using the Arrhenius method, considering the nonlinearity in the rate of hydration of cement. The Arrhenius' method produces a maturity index in terms of an "equivalent age", representing the equivalent time of curing, to a reference temperature, usually 20 ° C, required to produce a maturity equal to that achieved for a period of curing at temperatures other than the reference temperature, equation 2.

$$t_e = \sum e^{-Q\left(\frac{1}{Ta} - \frac{1}{Ts}\right)}\Delta t \qquad (2)$$

where:
t_e = equivalent age to a reference temperature T_s, in days or hours,
Q = activation energy divided by the general gas constant, in K,
T_a = concrete average temperature during the interval Δt, in K,
T_s = reference temperature, in K and
Δt = time interval, in days or hours.

The Arrhenius' equation is a better representation of the temperature-time function that equation of Nurse-Saul, when a wide variation expected in the concrete temperature. In addition, the focus of Nurse-Saul is limited according to assume that the rate of strength gain is a linear function. However, the formula of Nurse-Saul is more widely used, primarily because of its simplicity. Both functions of mature are considered in ASTM C 1074 (Barreda M. F., M. J. Naber, Quispe Sallo I., J. D. Sota, 2003).
Because maturity is dependent only on the history of time and temperature of the concrete, the most basic equipment requirements to determine maturity they are a thermometer and a clock. However, over the years, it has developed several devices maturity to monitor and record automatically concrete temperatures versus time. These devices connect to thermocouples embedded in the concrete and can be computed by the equation maturity of Nurse-Saul and the Arrhenius equation, at defined intervals (ASTM C 1074, 1998).
In the case of this work, we have developed a prototype measurement equipment together with software, in order to perform the experiments (Sota J. D., F. A. Avid, Chury M., P. Moreira, 2014).

2. METHODOLOGY

An equipment measurement was developed and supplemented with software that allowed to handle the data. The system design includes a series of temperature sensors connected to a microcomputer (Figure 1), which also recorded the temperature on the concrete surface. (Figure 2). The microcomputer reads the temperature of sensors and records your values. (Figure 3). A program performs a continuous reading of the information generated that then stored in a database, enabling processing using the expressions for calculation Nurse-Saul maturity and / or Arrhenius.

Figure 1. Microcomputer (RasberryPi B+)

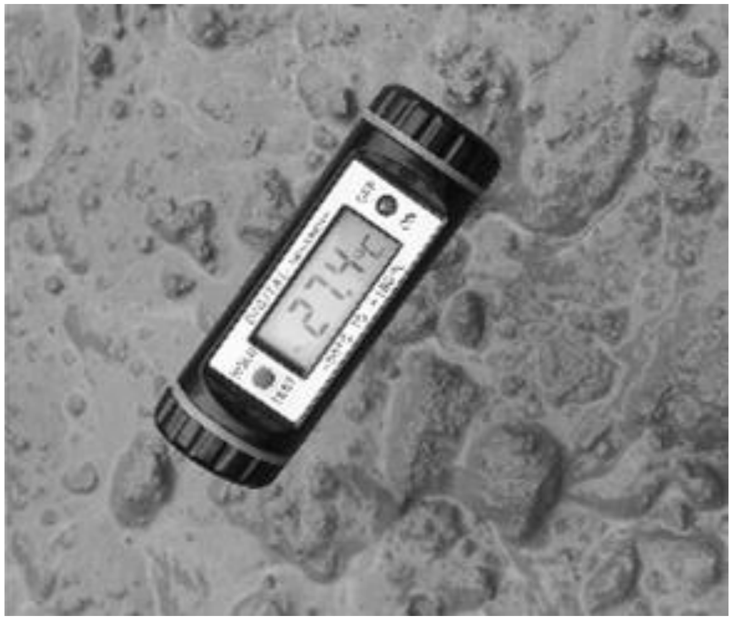

Figure 2. Digital thermometers

Figure 3. Sensors

Were studied the concrete bases of a structure in an extension of the laboratories of the Faculty, with monitoring the development of resistance to the measure of the maturity of concrete, with sensors placed in them. (Figure 4). The bases to place the sensors were chosen based on their location in the structure and the collocation steps of the concrete during the day. Which allowed have values of register in concretes placed in the morning (Base 7), averaged the concrete cast (Base 3) and end of the same (Base 10).

Figure 4. Attaching sensors

The dosage was composed of a CPC-40 portland cement, portland composite cement (up to three additions) mortar resistance of 40 MPa (IRAM 50000); thick silica sand from a quarry in the area; siliceous boulder sizes 1: 3 and 1: 2 and superplasticizer additive. The characteristics of the aggregates are reported in Table 1.

Table 1. Aggregate characteristics

Material	**Fineness modulus**	**Maximum size**	**P.U. volume**
Thick silica sand	2,69	--	1, 5
Boulder 1:3	7,26	1”	1, 7
Boulder 1:2	6,70	3/4”	1, 6

The proportions of the members dosing materials are summarized in Table 2.

Table 2. Dosing of concrete used in the experiences

Material	**P.e (g/cm3)**	**Volume (liters)**	**Weight (kg)**
Water	1,00	158	158
Cement	3,11	101	315
Thick silica sand	2,62	309	811
Boulder 1:2	2,66	167	444
Boulder 1:3	2,67	249	666
Additive	2,5 kg. / m3		
Air (%)	2		
Asentamiento (cm)	10		
Average resistance 28 days	25 MPa		

Were prepared 15x30 cylindrical specimens for resistance determination to different ages studied, simultaneously with the placement of concrete. So, that the resistances of the specimens they corresponded to the concrete placed on the base they were sensors placed. Temperature measurements were made on the bases 3, 7 and 10 of the structure. Strength tests were performed with a Digital Press PILOT 4 Automatic (Controls Italy) 200 tons of capacity; with real-time graphical display of test data, load curve / time and the actual load speed and simultaneous display of load, stress and actual load speed depending on loads or stresses.The tested specimens were kept in the environment of the basis on which the measurements were made during the experience, under the same conditions of humidity and temperature (25.5 to 27.5 ° C and 75% RH).

3. RESULTS

It obtained the data of resistance in compression tests of specimens and temperature with equipment designed for these experiences (located at bases 3, 7 and 10), it proceeded to correlate these to the split times used. (Details of the madurómetro and resistance at the same age).
The Nurse-Saul formula was used - Maturity (ºC.h) for variables, time, temperature and strength - . Figure 5 shows the time relationship vs. maturity to the base 10 (as an example) and Figures 6, 7 and 8 the strenght relationship vs. maturity for bases 3, 7 and 10.

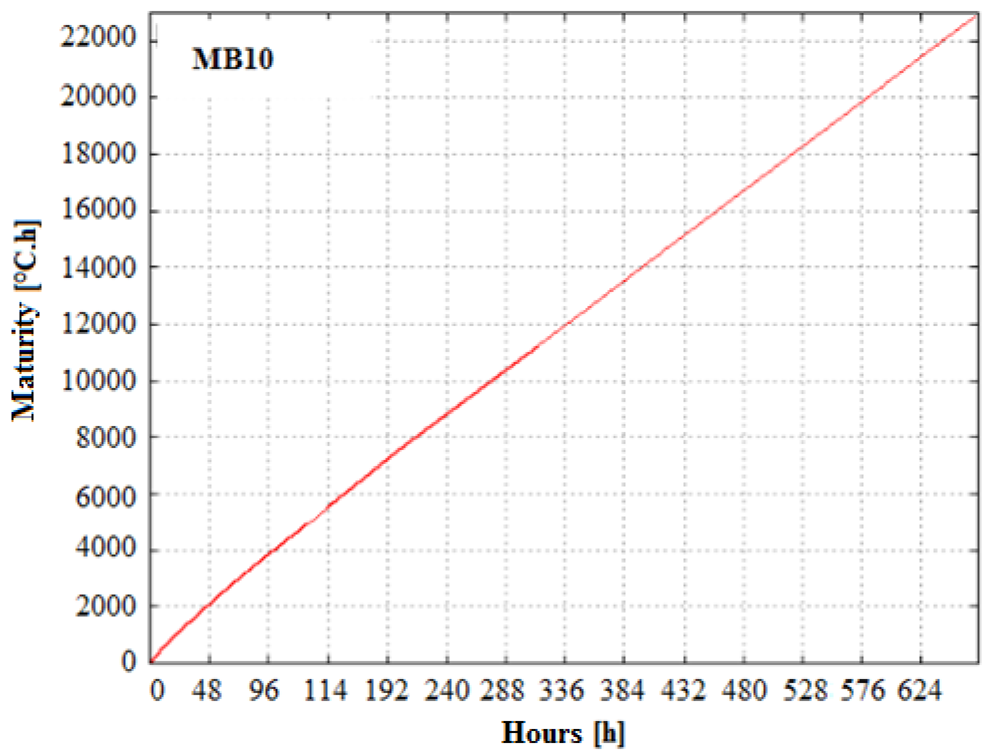

Figure 5. Time vs maturity Base 10

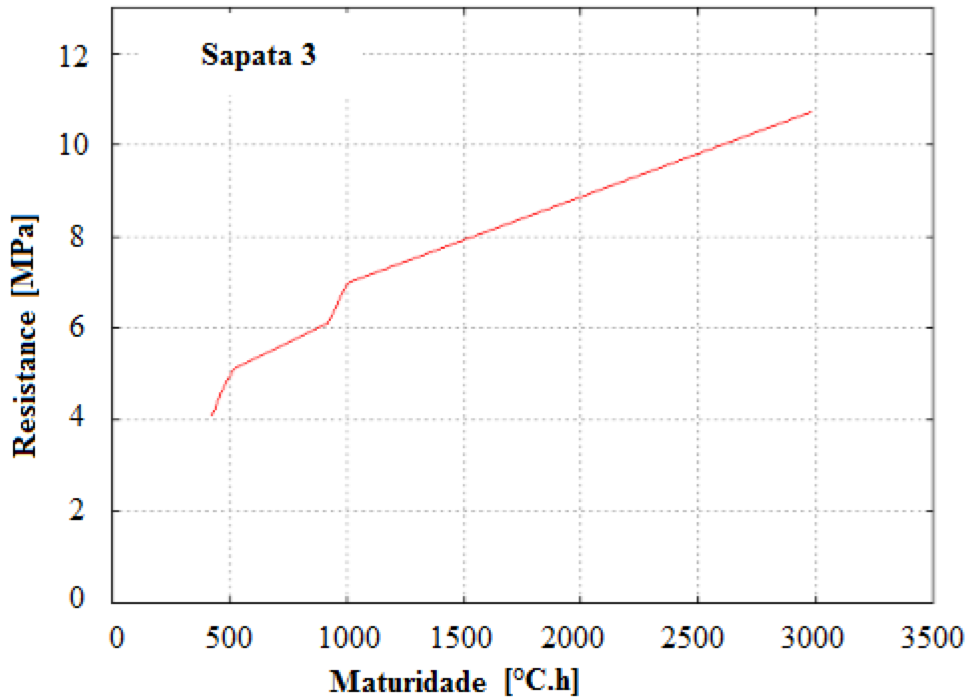

Figure 6. Resistance vs maturity Base 3

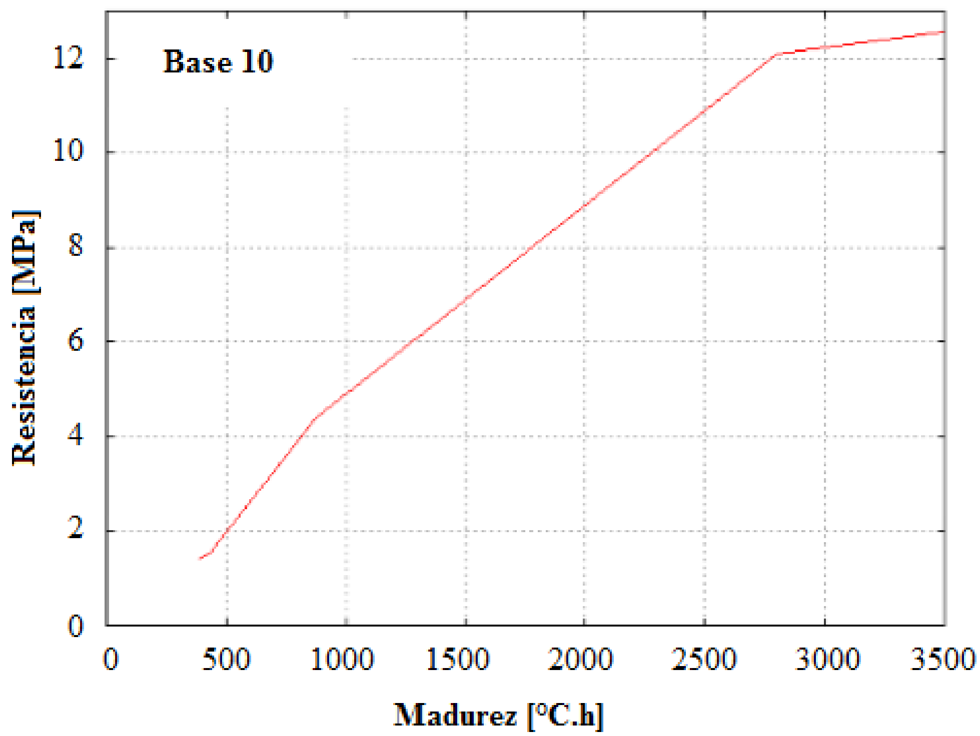

Figure 7. Resistance vs maturity Base 10

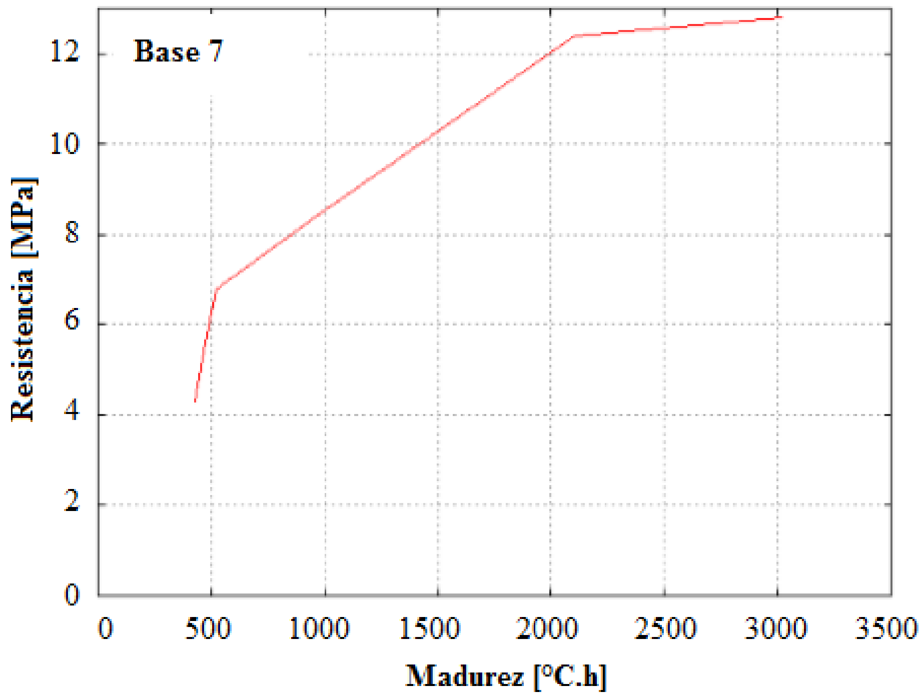

Figure 8. Resistance vs maturity Base 7

A good correlation between the measured values of maturity and corresponding resistances are observed therein. The graphics express the actual resistance values determined with each base test specimens. Table 3.
The sensors confirm that the concrete delivered to the work met the resistance value required by the specification (H21). The values at 28 days of resistance confirm this.

Table 3. Resistances certain of the bases with concrete specimens at different ages.

Base 3		**Base 7**		**Base 10**	
Age hours	**Strength in MPa**	**Age hours**	**Strength in MPa**	**Age hours**	**Strength in MPa**
9	1,4	--	--	--	--
10	1,5	10	4,1	10	4,0
11	--	11	4,7	11	5,6
12	--	12	5,0	12	6,8
20	4,4	20	--	20	--
21	--	21	6,1	21	--
22	--	--	--	22	8,4
23	--	23	7,0	23	--
68	12,1	73	10,7	74	12,8
667	25,0	--	--	--	--

4. FINAL CONSIDERATIONS

In function on the results, obteined of this first experience can make the following considerations: The use of this methodology allows to control the entire concrete received, without taking a significant amount of samples for further testing.

The lecture of sensors allows monitor homogeneity of the concrete and the development of resistance day to day. The methodology is applied in our next experience monitoring resistance in a complete concrete structure (slabs, beams and columns).

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Analysis of steel bars corrosion as a function of the environment pH

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ABSTRACT
The aim of this study is to analyze how the steel type, the environment pH and surface protection of steel bars are able to change the electrochemical properties of this metal. Therefore, it was applied the linear polarization technique to steel bars to assess the corrosion of the samples. The study evaluated the CA-50, CA-60 and CP-175-RB steels bar, with and without surface protection. Studies like this are essential for the improvement of reading techniques, especially for the understanding of the results obtained in repairs already made. From the results, it was possible to observe how the three variables influenced the results of the current density and corrosion rate.
Keywords: corrosion rate; durability; pH; linear polarization.

RESUMO
O objetivo deste estudo é analisar como o tipo de aço, o pH do meio e a proteção superficial da barra de aço são capazes de alterar as propriedades eletroquímicas deste metal. Para isso, foi aplicada a técnica de polarização linear em barras de aço para avaliar a corrosão das amostras. No estudo foram avaliados os aços CA-50, CA-60 e CP-175-RB, com e sem proteção superficial. Estudos como este são essenciais para o aprimoramento das técnicas de leitura, em especial para o entendimento dos resultados obtidos em reparos já realizados. A partir dos resultados foi possível observar como as três variáveis analisadas influenciaram os resultados da densidade de corrente e da taxa de corrosão.
Palavras-chave: taxa de corrosão; durabilidade; pH; polarização linear.

RESUMEN
El objetivo de este estudio es analizar cómo el tipo de acero, el pH del medio y la protección de la superficie de la barra de acero son capaces de cambiar las propiedades electroquímicas de este metal. Para esto se aplicó la técnica de polarización lineal en barras de acero para evaluar la corrosión de las muestras. El estudio evaluó los aceros CA-50, CA-60 y CP-175-RB, con y sin protección superficial. Estudios como éste son esenciales para la mejora de las técnicas de lectura, especialmente para la comprensión de los resultados obtenidos en las reparaciones ya realizadas. A partir de los resultados fue posible observar cómo las tres variables influyeron en los resultados de la densidad de corriente y velocidad de corrosión.
Palabras clave: velocidad de corrosión; durabilidad; pH; polarización lineal.

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1. INTRODUCTION

The problem of corrosion consumes directly or indirectly about 5% of GDP of an industrialized nation and involves major disasters when not properly treated (Cunha *et al*., 2013). The main corrosion problems are associated with the lack of suitable coverings of concrete. Since concrete gives the steel a double protection: first a physical protection, separating the steel direct contact with the external environment, and second, a chemical protection conferred by the high pH of the concrete, which promotes the formation of a passivating film that covers the steel (Figueiredo and Meira, 2012).
Corrosion may be defined as the deterioration of a metal or alloy, from its surface, by the environment into which it is inserted. The process involves oxidation and reduction reactions (redox) that convert the metal or the metal component into oxide, hydroxide or salt (Silva *et al*., 2015).
There are several factors influencing the corrosion parameters of concrete immersed metal: the diffusion coefficient, the water/cement ratio, the thickness of the coating, the presence and amount of additions, the humidity, the pH of the concrete and the temperature of exposure (Andrade, 2001; Gu and Beaudoin, 1998). In addition, the type of protection system applied to the metallic material are relevant, because the concrete or steel bars can receive surface protection (Figueiredo and Meira, 2012).
When the strategy is to protect the steel bar, the protection technique can be applied in the construction of new structures or in limited repair areas when there is a concern for the reinforcement bars corrosion (Araujo, Panossian and Lourenco, 2013).
Intending to offer safety and durability for structures, the chemical and the civil construction industry sell products for corrosion protection in different forms (Vieira *et al*., 2010). In the case of repairs, some authors describe the main protection methods used in Brazil (Figueiredo and Meira, 2012), as shown in Figure 1.

Electrochemical techniques	Coating	Spercial reinforcing steel	Corrosion inhibitors
• Cathodic protection • Realkalisation	• Galvanization • Epoxy resin	• Stainless steel • Plastic without fiber	• Organic • Inorganic

Figure 1. Protection methods most used in steel bars for structural repairs
Source: Adapted from Figueiredo and Meira (2012)

It is noteworthy that in the area of diagnosis of pathological manifestations is inherent in reconciling research, testing, interpretation of results and mastery of the latest developments regarding the prognosis of structure deterioration mechanisms and its influential (Medeiros *et al.* 2012). Thus, this study aims to assess how the type of steel, the pH of the environment and the presence of surface protection used in the steel bar can alter the electrochemical characteristics of the corrosion mechanism. This type of study is essential to understanding the influencing factors of corrosion and how effective repair products are. As well as forecast the life of reinforced concrete structures.

2. LINEAR POLARIZATION RESISTANCE (LPR)

There are several types of measure for corrosion controlling, but the current density and corrosion potential are most commonly used in the diagnosis of reinforced concrete (Tavares, 2006). Table 1 and Table 2 show the corrosion level in terms of current density values and corrosion potential.

Table 1. List of current density by level of corrosion

i_{cor} (μA/cm²)	Corrosion rate (μm/year)	Corrosion level
< 0,1	< 1,16	Passive state
0,1 a 0,5	1,16 a 5,80	Low to moderate state of corrosion
0,5 a 1,0	5,80 a 11,60	Moderate to high state of corrosion
> 1,0	>11,60	High corrosion rate

Source: Cunha *et al.* (2003)

Table 2. Potential corrosion evaluation criteria.

Corrosion potential value	Probability of occurring corrosion
< - 350 mV	90 %
- 200 mV a - 350 mV	Uncertainty
> - 200 mV	10%

Source: ASTM C 876 (2009)

Among various techniques used to study corrosion and used for the determination of the current density, corrosion potential and corrosion rate, the linear polarization resistance technique (LPR) is the most applied (Alves *et al*., 2012). Its wide application is due to the speed and ease with which can determined the variables measured. The purpose of this technique is to measure the resistance that a material, exposed to particular environment, offers to oxidation during the application of an external potential.

To perform the technique is common to use a potentiostat for applying different potentials (voltages) in the reference electrode (Flores *et al.*, 2013). Thus, if the potential is different than the corrosion potential, the equipment registers the applied current (Fofano, 1999). Thus, is is obtained the variation of the current as a function of the applied potential (E vs I) as shown in Figure 2. Another control parameter obtained is the polarization resistance, which can be considered as the ratio between the potential difference and the applied current (Liu, 1993).

From these data it can be made quantitative measurements of various parameters of electrochemical corrosion, based on the equations presented by the ASTM G 59 (1997) and by Wolynec (2003). The current density, for example, can be calculated as shown in Equation 1 and Equation 2.

$$i_{cor} = \frac{B}{R_p} \qquad (1)$$

$$B = \frac{\beta_a . |\beta_c|}{2{,}303 . (\beta_a + |\beta_c|)} \qquad (2)$$

Which: i_{corr} is the corrosion current density (A/cm²), β_a is the Tafel anodic slope and β_c is the Tafel cathodic slope (V/decade), e R_p is the polarization resistance (ohm/cm²).

The corrosion rate – CR (mm/year), on the other hand, can be determined from Equation 3, where Eq is the electrochemical equivalent of the corroded species (g), and ρ is the density of the corroded material (g/cm³).

$$CR = 3{,}27.10^{-3}.\frac{i_{corr}.Eq}{\rho} \tag{3}$$

The corrosion rate CR (µm/ano) can also be considered equivalent to 11.6 times i_{corr} (µA/cm²) for a steel bar (RILEM, 2000).

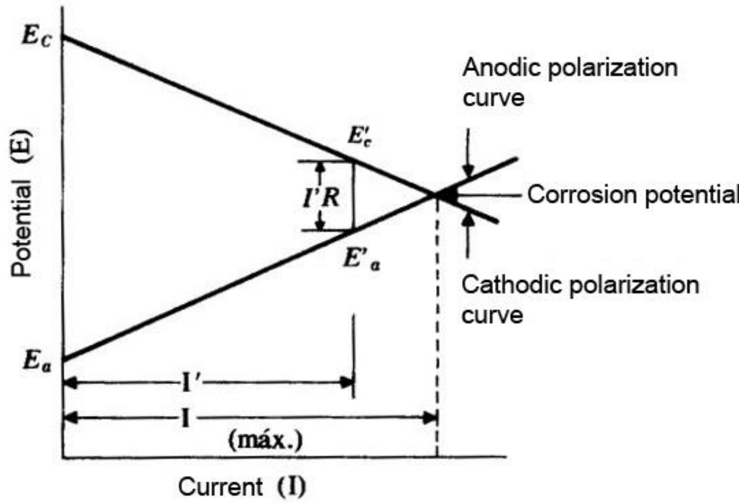

Figure 2. Representative cathodic and anodic polarization curves of metal.
Source: Wolynec (2003)

3. MATERIALS AND METHODS

In the ollowing, the materials used in this research and the test methods adopted will be presented. The steel types were chosen from the NBR 7480 (1996) which deals with the steel destined for reinforcement of concrete structures and the NBR 7482 (2008) which deals with steels for prestressed concrete structures. Another criteria for the selection of steel types was the proximity of the diameters between classes, to reduce variability in the results. The selected types are displayed in Table 3.

Table 3. Tested steel bars

Steel	Diameter
CA – 50	6.3 mm
CA – 60	5.0 mm
CP – 175 RB E	6.0 mm

For each steel from Table 3, there were four specimens of 15 cm in length, wiped with a hydrochloric acid solution according to ASTM G 1 (2011).

Tests were performed with bars immersed in solutions of different pH, altered by the addition of sodium hydroxide and controlled by a benchtop pHmeter at 25° C. The solutions had the values of 7, 9 and 11 initial alkalinity, and there were no pH adjustment after start of the tests. Measurements were taken at 7 and 30 days. It should be noted that the steel bars were in containers, sealed, and immersed in an aggressive solution, thus, there was no entrainment material contamination or addition of substances and gases.

Before the tests, part of the bars were subjected to a surface protection method. The protection system used was a rust converter applied in two coats with a 60 minute interval, with subsequent application by

an acrylic painting. The protection product used had density of 1.03g/cm³, solids content of 10 to 15%, pH of 2.6 and chemical composition of organic extract of *Acacia Mearnsi* (3-15%), citric acid (2-10 %), acrylic co-polymer (5-20%) and 2-butoxy ethanol additive (3-15%).
The linear polarization resistance tests (LPR) were performed as prescribed by the ASTM G 59 (1997) with a SP-200 BioLogic potentiostat - Figure 3 - which used an electrode of copper/copper sulphate ($Cu/CuSO_4$) by applying voltages ranging from -2V to + 2V.

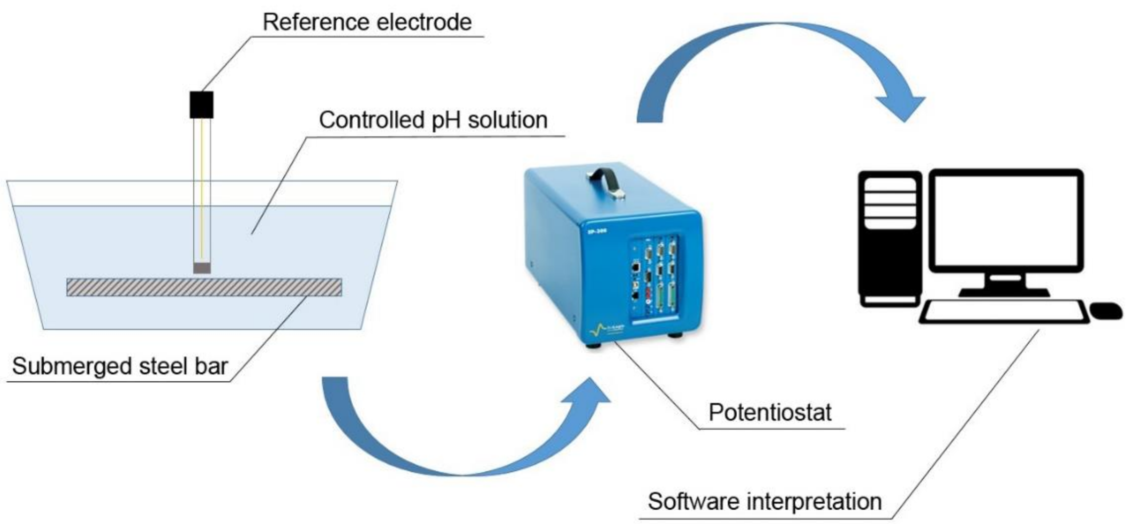

Figure 3. Carried out test with SP-200 Potentiostat.

For each test condition (pH and protection) four measurements were performed, and the electrode was placed as close as possible to the steel bar during the tests. After the test, the polarization curves were analyzed and thereby it was possible to obtain the corrosion potential (Ew), corrosion current density (Icorr), the corrosion rate (CR) and polarization resistance (Rp) for each of the steel bars - with and without protection - at different pHs.
To validate the results, the Tukey statistical test for multiple comparison of results was applied. Thus demonstrating the differences between the results obtained with a confidence level of 95%. We emphasize that the statistical analysis were performed only for the results of the current density and corrosion rate.

4. RESULTS AND DISCUSSION

The table below shows the results of corrosion measurements of steel bars under different pH levels with and without surface protection in the bar. Figure 4 and 5 show the results of potential corrosion to the steel bars after 7 and 30 days, respectively. Data were ranked based on the probability of corrosion as is shown in Table 2.

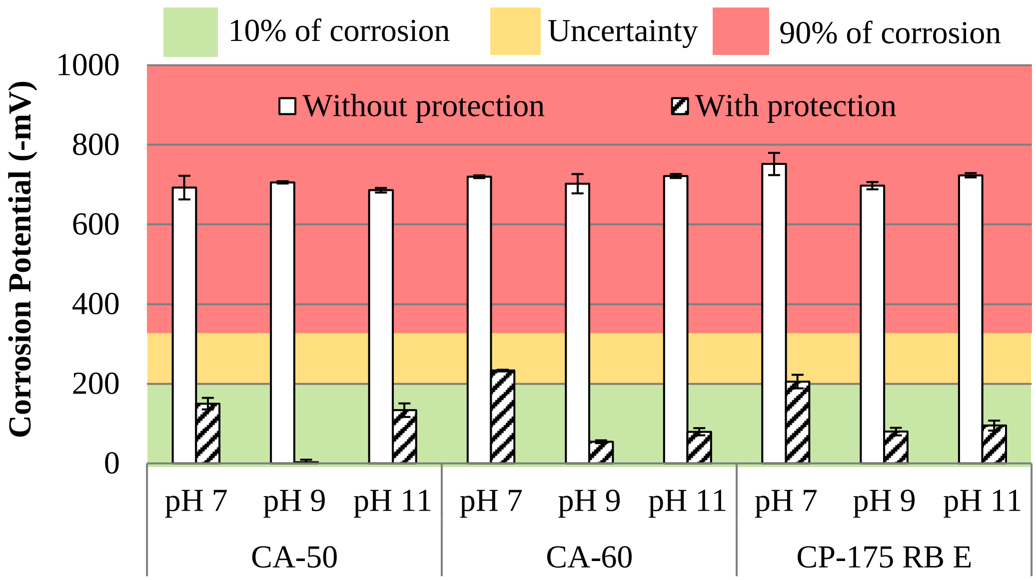

Figure 4. Potential corrosion of the bars at 7 days (Reference electrode: copper/copper sulphate-Cu/$CuSO_4$).

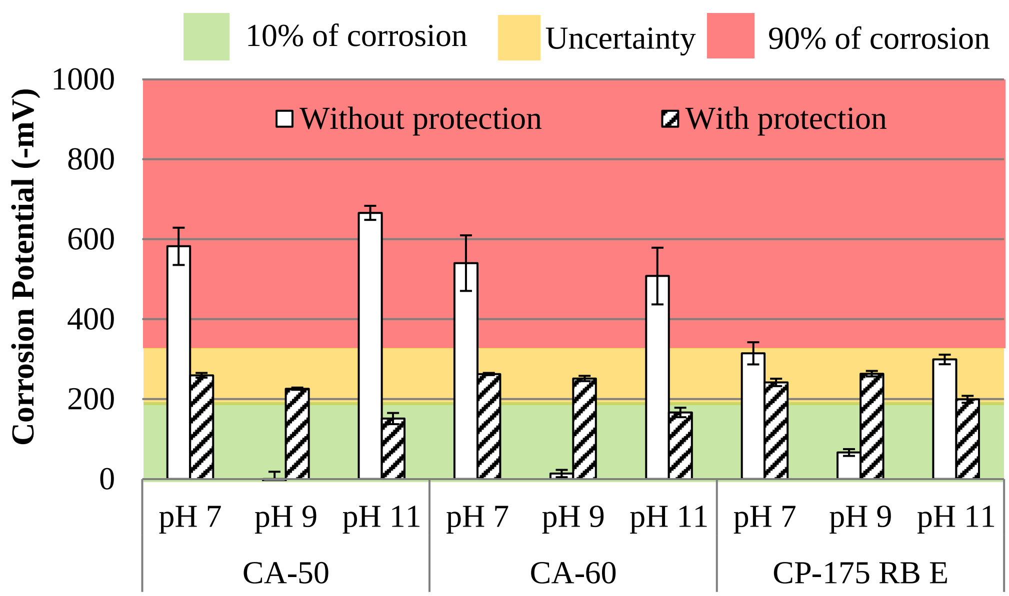

Figure 5. Potential corrosion of the bars at 30 days (Reference electrode: copper/copper sulphate-Cu/$CuSO_4$).

It is observed in Figure 4 that, at 7 days, all unprotected steel bars have probability of 90% of corrosion, averaging -711mV. While 78% of the bars are protected with a low probability of corrosion (10% occurrence), with data ranging from -3mV and -205mV. The results in Figure 4 show the rust converter action in corrosion potential application in early ages.

In 30 days (Figure 5), the surface without protection had increased corrosion potential, presenting results from -6mV and -582mV. Even after 30 days, the bars with protection have undergone an increase in corrosion potential, with a reduction to 33% of the samples in the low likelihood of corrosion area (with -224mV reading average). Also, we evaluated the results of the current density and corrosion rate depending on the level of corrosion shown as in Figures 6 and 7.

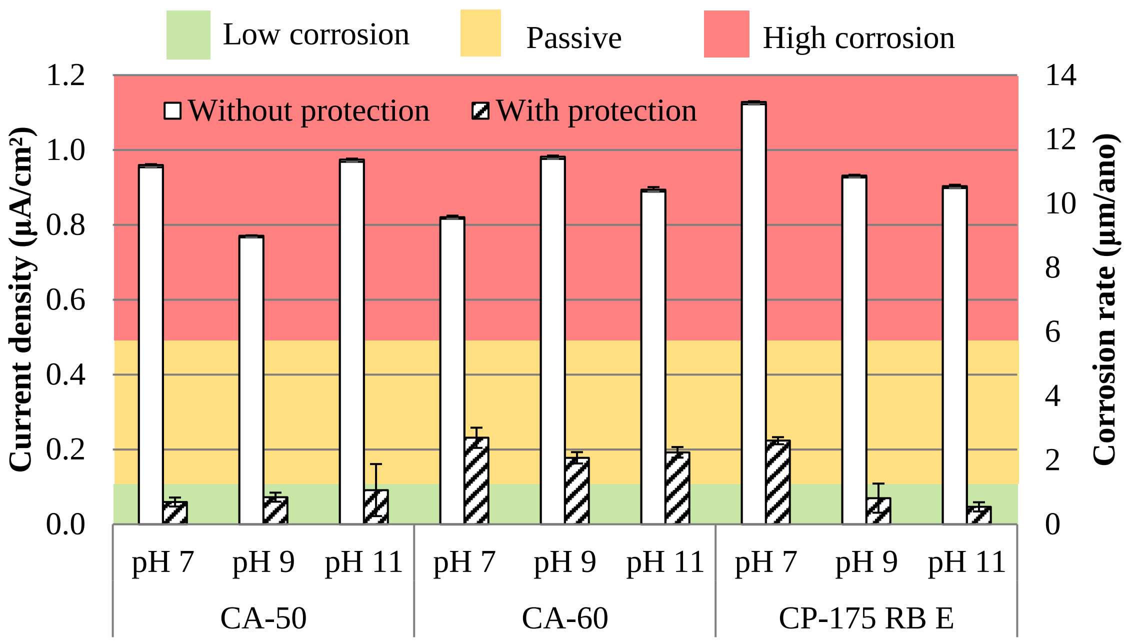

Figure 6. Current density and corrosion rate of the bars at 7 days (Reference electrode: copper/copper sulphate- $Cu/CuSO_4$).

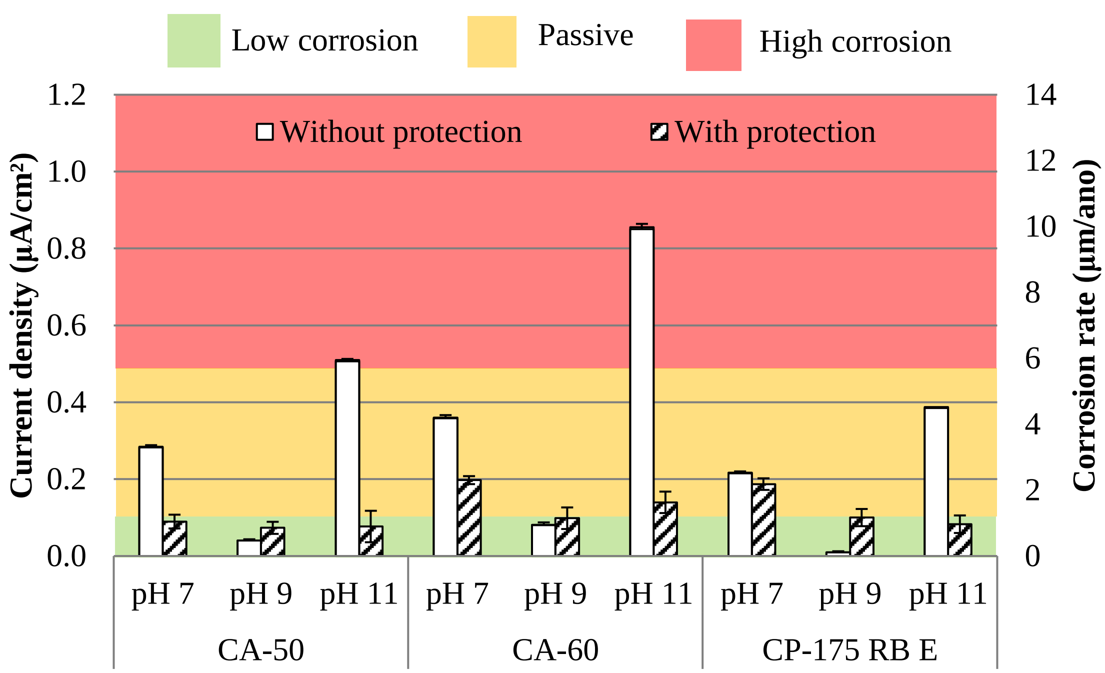

Figure 7. Current density and corrosion rate of the bars at 30 days (Reference electrode: copper/copper sulphate- $Cu/CuSO_4$).

With the data presented in Figure 6, it can be seen that for analysis at 7 days occurring high corrosion rate value in bars without protection, or a state of general corrosion (with all the results classified as moderate corrosion to high, and current densities greater than 0.77 uA / cm²). It should be noted in Figure 6 that in all cases, the corrosion rate was larger in the bars without protection, if compared with steel bars that have reached their protected surface current densities lower than 0.23 uA / cm², with 78% of the results in the passivity zone. Note that the CA-60 steel with protection showed the highest corrosion rate values when compared with the other results of steel protected surface.

As for the analysis carried out after 30 days, (Figure 7) has a reduction of corrosion rate in all cases. Being the most part included in the first two groups of corrosion status classification: passive and low to moderate state of corrosion, with the exception of the CA-50 and CA-60 steel in the environment for pH = 11.

It is known that during the initial periods of exposure to the atmosphere, the corrosion rate of carbon steel is usually high. This is due to the high porosity of the rust initially formed, consisting mainly of iron oxides. After this initial period, the improved protective properties and the corrosion rate decreases (Panonni et al., 1993). Because of this behavior, exponential models are used to represent the corrosion rate over time (Hakkarainen, 1982; Barton, 1980; Pannoni and Marcondes, 1991). Thus, the results obtained are consistent with the results observed by other authors.

Therefore, the decrease in corrosion rate could have been caused by severe initial corrosion in the equipment, which resulted in the formation of a layer of corrosion products on the exposed surface of the armor which, in other words, may have limited contact between steel samples and solutions. Another possibility not covered by others is the variation of oxygen present in the solution, that is, with the iron rust was a decrease in the concentration of oxygen present; ie, the corrosion product itself tended to reduce the speed of progress of corrosion of reinforcement, as it presents as a physical barrier to the electrolyte access.

Note also that the steel bar CA 60, generally had higher values than the others, indicating a greater susceptibility to corrosion, but without statistical significance. It should also be noted that the protection capacity of the steel surface treatment system does not have to be more effective after 30 days of exposure to the corrosion conditions. To allow a better analysis of the influence of solution pH, Figures 8 and 9 were prepared according to the pH of the solutions.

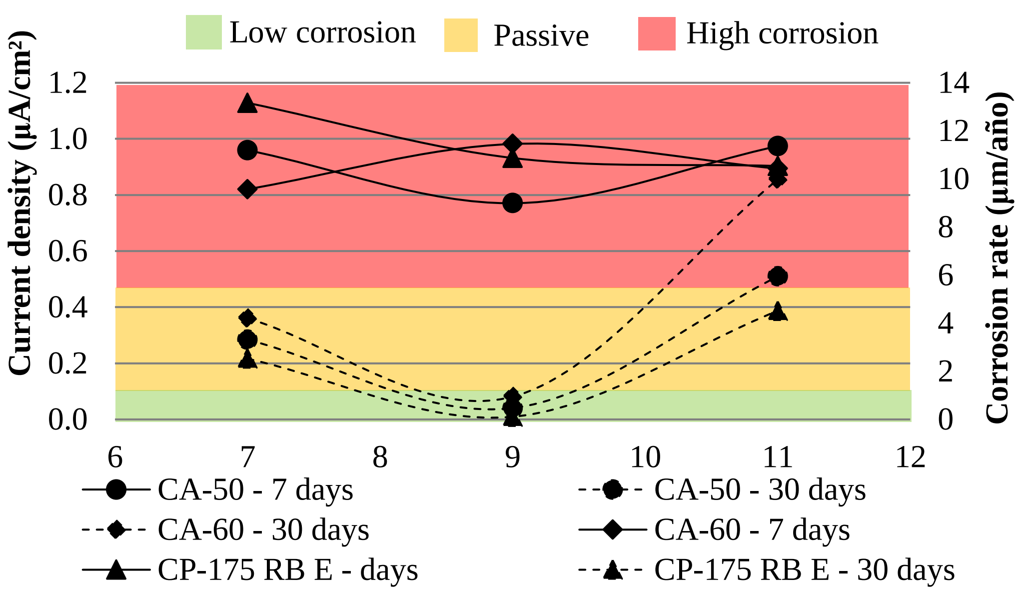

Figure 8. Current density and corrosion rate of the bars without protection (Reference elect rode: copper/copper sulphate- $Cu/CuSO_4$).

From the results shown in Figure 8, it could be observed that the pH had little influence on the results of unprotected bars, exposed for a period of 7 days, and the samples achieved a current density value in the range of 0.8 to 1.1 uA / cm², being classified as a corrosion state of moderate to high. During the 30 days, unprotected bars showed mixed results of corrosion rate, especially the reduction of corrosion to those who were in the pH = 9 solution - reaching passivity.

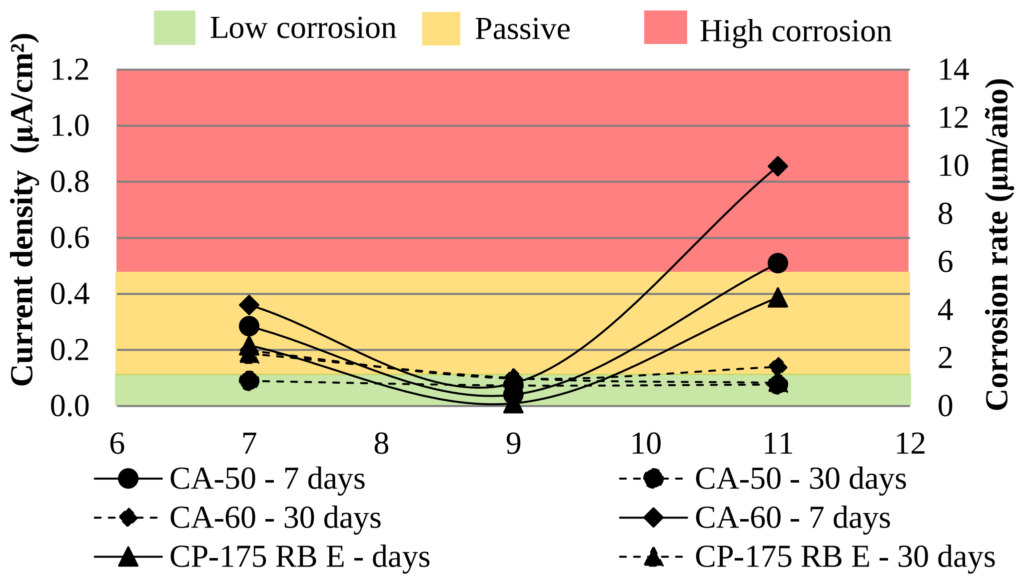

Figure 9. Current density and corrosion rate of the bars with protection (Reference electrode: copper-copper sulphate- $Cu/CuSO_4$).

By analyzing the bars with surface protection (Figure 9), it is noted that at 7 days was a variation in current density with increasing rates including protected bars, when pH = 11 with values up to 0.85 uA / cm². At 30 days the pH of the solutions was not influential on the results of the current density with near passivity results. However, it is possible to highlight the pH = 9 inhibited the corrosion process in all protected bars.

As the results of pH, the variation in the type of steel used is CP, CA 50 and CA 60, had little influence on the variability of results. However, it is observed that in both scenarios (30 days without protection and 7 days with protection) CP-175 was the least damaged by steel corrosion effect, followed by steel AC 50 and AC 60. This behavior can be explained by the fact that from 0.15 to 0.40% carbon steels in CA 50 and CA 60, while CP-175 presents content from 0.70 to 1.20% carbon (ARCELORMITTAL, 2016). Thus, by having a lower iron content in its chemical composition, the prestressing steel tends to have a lower rate of iron oxide formation.

Figure 10 shows a correlation between the results of corrosion potential and current density. It is found that there is a correlation between the two quantities corrosion monitoring, with R2 equal to 0.83.

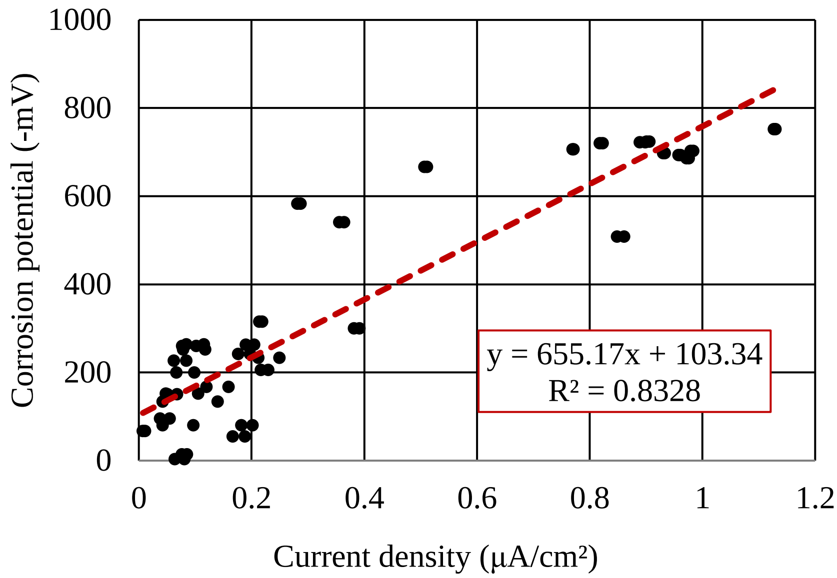

Figure 10. Correlation between measurements taken (Reference electrode: copper-copper sulphate-Cu/$CuSO_4$).

This result is expected since the increase in the corrosion potential induces an increase of the Tafel slope and hence the current density. The existence of a good correlation proves the effectiveness of the techniques applied.

With the objective to prove the difference between the results, was performed statistical analysis, by Tukey test, shown in Figure 11. In the test, the confidence intervals represent the interaction between two samples if there is the intersection between the end of the range top and bottom of the samples with zero vertical axis, it can be said that the differences between them are not significant.

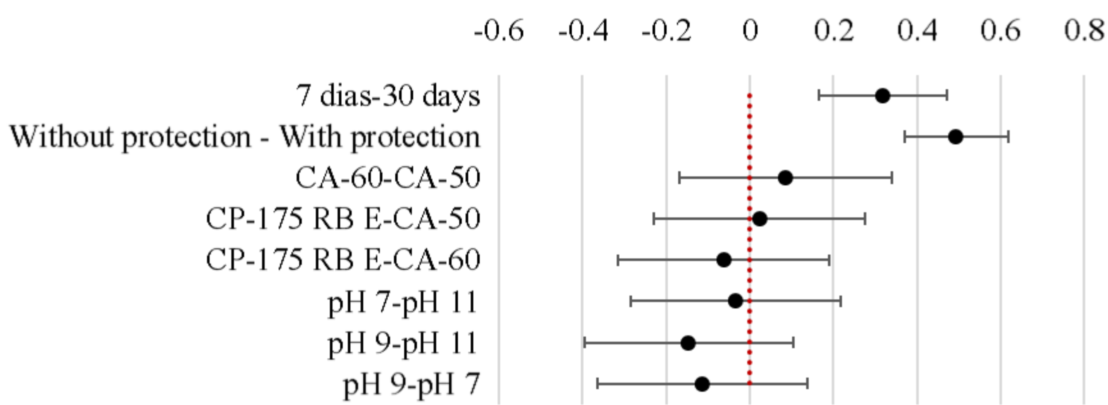

Figure 11. Tukey test for multiple comparison of averages.

As results show in Figure 11, differences occurred - with higher significance than 95% - of the results obtained after 7 and 30 days and between the bars with and without surface protection. Thus, protection was able to improve the electrochemical properties of the steel at different pHs, thus reducing the corrosion rate and current density.

Still according to Figure 11, it was observed that the type of steel used and the pH of the solutions did not exert statistically significant influence on the results. This behavior is explained by the observed deviation in the values of current density with different behaviors in each of the conditions studied.

However, although not statistically significant when analyzed, deviations in Tukey test caused by type of steel, it is noted that the steel that showed no difference in the results were the CP-175 and CA-50

steel. The pH of the solutions, it is noted that the pH = 9 was which generated a greater difference in the results (when observing the deviation of the pH analysis 9 - pH 11 and pH 9 - pH 7).
This type of analysis is essential to prove that the durability studies increasingly need to evaluate other variables that can influence the tests and environmental interactions in which materials are with steel and concrete.

5. CONCLUSION

In general, it confirmed the influence of variables on the corrosion process of steel bars: time and surface protection system. In this study we observed high corrosion rate values at early ages, with a reduction 30 days after the medium change - caused by the material resulting from the etching process (possible deposition surface on the bar and change of oxygen concentration in the solution). It was observed that bars subjected to surface protection demonstrated ability to maintain the corrosion rate even with low values ages.
The analysis of the classes of the steels found that there was a small difference between the values without influence statistically significant in the results of current density and each rate. The pH values, in turn, also did not statistically influence the results obtained.
Note that the linear polarization method for corrosion rate measurements proved to be efficient and accurate in its determinations, with good correlation between the measurements of corrosion potential and current density. Moreover, it can be stated the importance of conducting more studies focused on understanding the variables that influence the corrosion processes, thus enabling better prediction of service life of reinforced concrete structures.

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Fly ash effect on mechanical properties of concretes made with high absorbent crushed limeston aggregates

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ABSTRACT

Concrete made with high-absorbent crushed limestone aggregates from Yucatán, México are well known as a low quality concrete. The aim of this investigation is to enhance the mechanical properties of concrete with high absorbent crushed limestone aggregates and fly ash. The measured properties were: compressive strength and elastic modulus. The water/cement ratios were 0.5 and 0.7, fly ash was incorporated as partial substitution of cement with 20% and 40% and as a mineral additive in 10% and 20%. Results show that fly ash can be used in this kind of concretes as mineral additive due to compressive strength was similar to those reference samples. Finally, an equation for predicting mechanical properties is reported.

Keywords: fly ash, limestone aggregates, absorption, compressive strength, elastic modulus.

RESUMEN

El concreto elaborado con agregado calizo triturado de alta absorción de Yucatán, México, es considerado de baja calidad. El objetivo de la investigación es mejorar las propiedades mecánicas del concreto elaborado con este tipo de agregado incorporando ceniza volante (CV). Las propiedades medidas fueron: Resistencia a la compresión (RC) y módulo de elasticidad. Se utilizaron relaciones agua/cemento de 0.5 y 0.7, la CV se incorporó como sustitución parcial del cemento en un 20% y 40%, y como aditivo mineral en un 10% y 20%. Los resultados indican que la CV puede ser utilizada en concretos con ACTAA como agregado inerte fino ya que logra mantener una RC similar a la referencia. Se presentan ecuaciones para la predicción de propiedades mecánicas.

Palabras clave: ceniza volante, agregado calizo, absorción, resistencia a la compresión, módulo de elasticidad.

RESUMO

O concreto feito com esmagado limestone agregada alta absorção de Yucatan, no México, é considerado de baixa qualidade. O objectivo da investigação é o de melhorar as propriedades mecânicas do betão fabricado com este tipo de cinzas incorporando mosca agregado (CV). As propriedades medidas foram: resistência à compressão (RC) e módulo de elasticidade. Foram utilizadas razões água / cimento de 0,5 e 0,7, o CV foi incorporado como substituição parcial de cimento de 20% e 40%, e como um aditivo mineral a 10% e 20%. Os resultados indicam que o CV pode ser usado em concreto com ACTAA inerte adicionada tão fina quanto possível manter um RC de referência semelhantes. Equações para predizer propriedades mecânicas são apresentados.

Palavras chave: cinzas volantes, agregados de calcário, absorção, resistência à compressão, módulo de elasticidade.

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1. INTRODUTION

Commonly, aggregates in concrete are two-thirds of the total volume and can influence in high grades the workability, mechanical properties, durability and porosity. In addition, they had used to reduce costs and provide stability. Therefore, aggregates characterization is essential to design and predict concrete behavior.

Concrete from Yucatán Península is made with crushed limestone aggregates with higher absorption, porosity, fineness powders, fragility and lower density. These properties are proper of deficient aggregates compared with lower absorption aggregates (Moreno and Arjona, 2011). Hence, the aggregate phase has influence in the mechanical properties of the concrete as Compressive Strength (CS) and Elastic Modulus (EM) and trigger an increase the cement amount to achieve target mechanical properties. Given these conditions, Solís and Moreno (2012) investigated the maximum CS in concretes with High Absorptions Crushed Limestone Agreggates (HACLA) and a w/c ratio between 0.20 and 0.45. The volume of cement was from 460 to 1300 kg/m^3 without pozzolans. The maximum CS was around 500 kg/cm^2 at 28 days and 600 kg/cm^2 at later ages. There was not a significant increase in CS with higher volumes of cement such as 850 kg/m^3 because the aggregates strength was exceeded. Cement is the most expensive material both economically and environmentally in concrete, therefore supplementary cementitious materials could be considered as a requierement in any construction. Pozzolans are siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties (ASTM C 125). Therefore, pozzolans are used to Portland Cement substitutions or as mineral admixtures to achieve similar or higher mechanical properties in Portland Cement based concretes.

Pozzolans are not commonly used in Yucatan Peninsula due to the lack of volcanic activity near from the región and industry development without puzzolanic residues. However, Aportela and Pardo L. (2002) studied the technical feasibility to use natural fly ash from Popocatepetl volcano as a supplementary cementitious materials in concrete with HACLA, authors observed a decrease in Compressive Strenght when this fly ash was added to concrete.

At Nava, México region, a significant amount of Fly Ash (FA) is obtained from a carboelectric factory due to pulverized carbon burn. This FA is classified as an artificial type F due to the origin and oxide composition according to ASTM C 618. It has been reported concrete with high compressive strength due to high contents of FA from Nava region and with low content of Portland Cement (100-150 kg/m^3), and the use of superplasticizer is a key to achieve the flowability (Valdez P. et al. 2007). Pozzolanic Activity was not investigated in Valdez P. work; However, if compressive strength increases, elastic modulus should increase too, therefore, FA from Nava was consider as a material with potential in concrete industry at México.

Siddique R. (2003) investigated mechanical properties of concrete (CS and EM) with fly ash mineral additions in 10%, 20%, 30%, 40% y 50%; higher values were obtained from the mixtures with mineral additions compared to the reference, also it was conclude that FA class F can be use for structural porpuses.

Porositys registered in concrete with HACLA oscilate between 18% and 25% for different water/cement ratios, this value is higher than concretes made with another kind of aggregates. A decrease of the porosity in cement paste phase with FA from Nava region is propose as a solution to enhance mechanical properties on concrete with HACLA.

The main objective from this work is to determine the performance of FA to decrease porosity and enhance mechanical properties such as Compressive Strenght and Elastic Modulus in concrete with HACLA. Another focus is to optimize the amount of cement paste when is partially substituided by FA. However, from social perspective, the use of an industrial waste as fly ash also has an important impact in the region where is produced.

2. EXPERIMENTAL PROCEDURE

Materials used to cast concrete mixtures were characterize according to American Society of Testing Materials (ASTM) standards. Composite Portland Cement (CPC 30R) was used in the cast of specimens due to his often use at Yucatán Península and the materials for this project has to be the closest possible to the construction field. This cement satisfy the requirements from NMX C-414 ONNCCE norm.
Morphology from FA was performe with an Image Analysis through Scanning Electronic Microscope (SEM), oxide composition was calculate by X-ray fluorescence (FRX), Particle Size Distribution was performe by Laser diffraction; also, Pozzolanic Activity Index (PAI) and Density were measured.
Mixture proportions were calculate according to American Concrete Institute Code (ACI 211.1) with two modifications:

1. Absorption of coarse and fine aggregates was consider as the 70% of the calculated absorption according to ASTM C127/C128 standards because of saturation on aggregate samples were during 15 and 60 minutes instead of 24 houres after being dried in the furnace at 100°C for 24 hours (Hernández, 2013)
2. Fly ash addition as a supplementary cementitious material (Mixtures SCV-20 and SCV-40, table 1) and as mineral admixture (Mixtures ACV-10 and ACV-20). When fly ash was added, fine aggregate was replace in mixtures.

Ten mixtures were design with water/binder (w/b) ratio of 0.5 and 0.7. At Table 1, nomenclature of mixtures is given to practical reading porpuses. Each mixture was of 55 liters and the numbers of especimens are presented at table 2. The specimens were cast according to ASTM C 31 and were cure with limewater.

Table 1. Concrete mixtures nomenclature.

Name	Mixture number	Characteristics
MR	2	Reference
SCV-20	2	20% substitution of cement by FA
SCV-40	2	40% substitution of cement by FA
ACV-10	2	10% mineral addition of FA
ACV-20	2	20% mineral addition of FA

Table 2. Concrete specimens

Compressive strenght at 28 days	Compressive Strenght at 91 days	Elastic modulus	Porosity, density and absorption
4 specimens of 10 cm per 20 cm	4 specimens of 10 cm per 20 cm	4 specimens of 15 cm per 30 cm	4 specimens of 7.5 cm per 10 cm

Compressive Strenght, Elastic Modulus and porosity measurements were determine according to ASTM standards.

3. RESULTS

Physical characterization of fine and coarse aggregates such as unit weight, specific gravity, absorption, abrasive resistance and fineness modulus are given at table 3. Presented values are the average of 3 samples. Particle size distribution of fine and coarse aggregates are presented at figures 1 and 2. It´s observed that coarse aggregates do not satisfy ASTM C33 especifications in contradistinction to fine aggregates.

Tabla 3. Aggreggates properties

Type	Specific gravity (SSS)	Loose Unit weight (kg/m^3)	Compact Unit weight (kg/m^3)	Absorción (%)	Abrassion (%)	Fineness modulus
Grueso	2.32	1113.41	1234.40	8.1	32	------
Fino	2.42	1280.36	--------	6.8	-----------	2.72

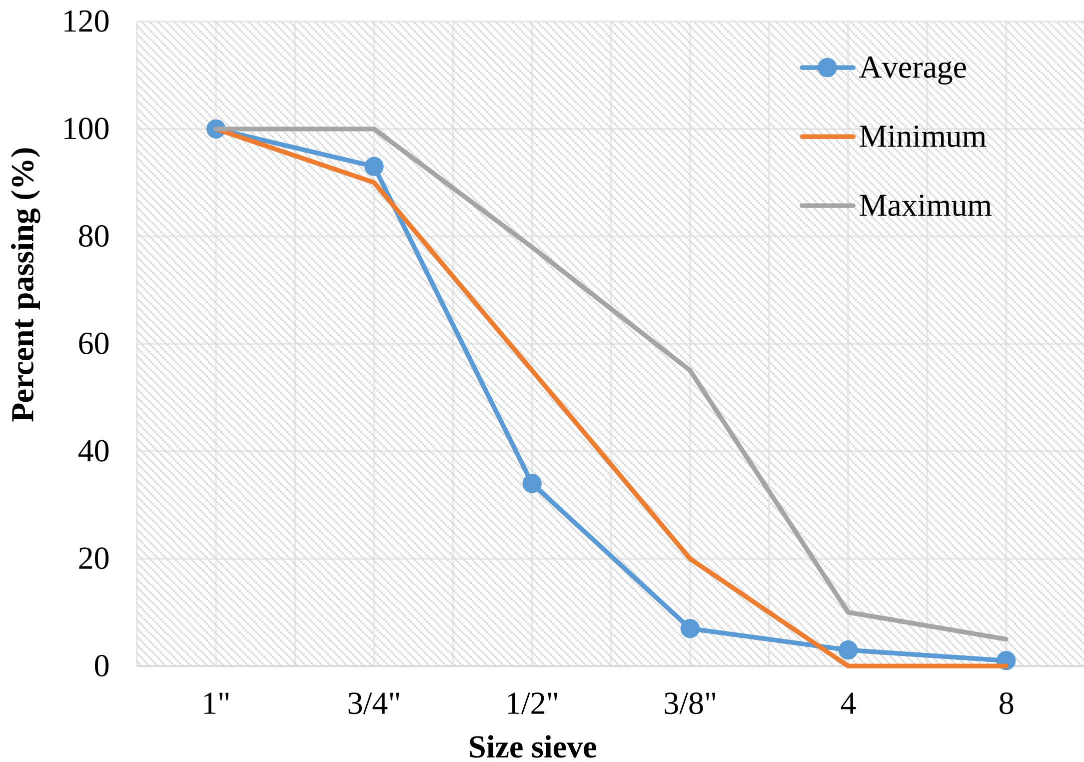

Figure 1. Particle size distributtion of coarse aggregates.

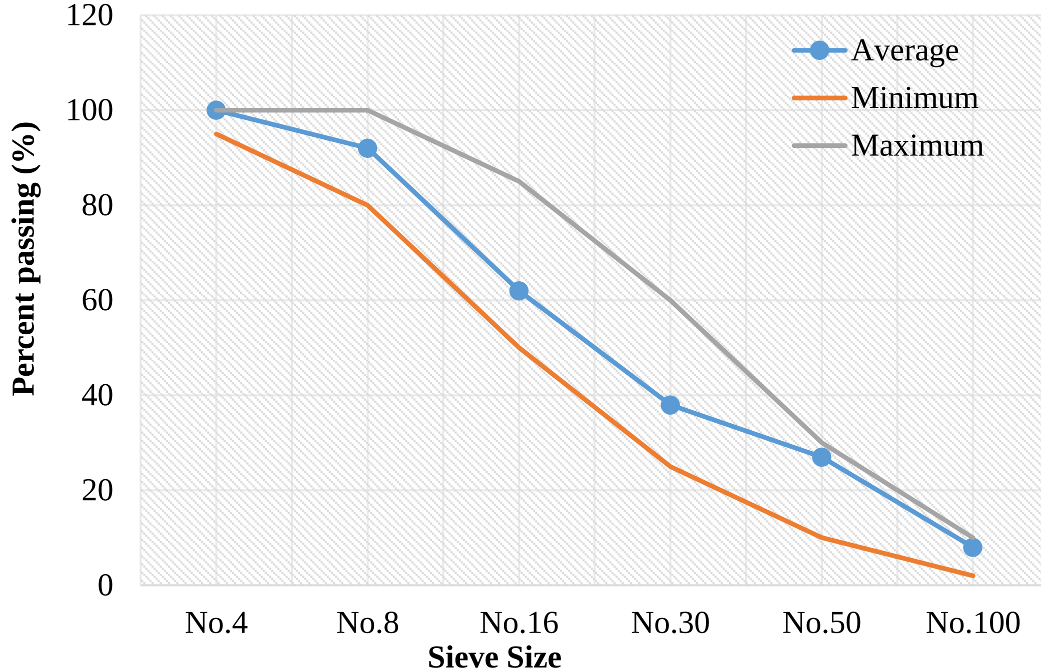

Figure 2. Particle size distributtion of fine aggregates

Fly ash should satisfy certain oxide composition, fineness and a Pozzolanic Activy Index to use within concrete according to ASTM C618. Total Oxide amount of aluminium, silica and iron should be as minimum 70% (table 4). Density of fly ash was 2.0 g/cm^3 according to ASTM C 311 and C188 method.

Table 4. Fly Ash oxide composition

Compound	**Na_2O**	**MgO**	**Al_2O_3**		**SiO_2**	**SO_3**	**K_2O**	**CaO**	**TiO_2**	**Fe_2O_3**
Percent (%)	3.315	1.667	33.105		56.511	0.344	0.518	0.698	0.357	1.486

Particle size distributtion of fly ash was determine through difrattion laser with a MICROTRAC equipment (figure 3). According to ASTM C 618, fly ash shouldn´t retain more than 35% of his total weight through sive no. 325 of 45 micrometers.

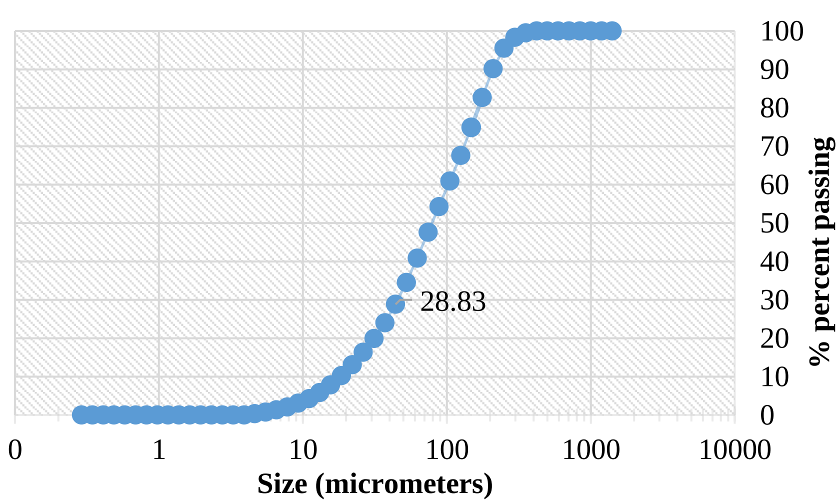

Figure 3. Particle size distributtion.

Finally, according to ASTM C 618, fly ash should obtain a PAI mínimum of 75% at 7 or 28 days to be considered as a pozzolan in concrete. Results are given at table 5.

Table 5. Pozzolanic activity index results.

Nom	Age	Compressive strenght (Kg/cm^2)	PAI (%)
MR-7	7 days	327.3	75%
MCV-7	7 days	244.6	
MR-28	28 days	401.5	82%
MCV-28	28 days	328.9	

Scanning Electronic Microscope images were taken at 1000 and 10000 x (Figure 4).

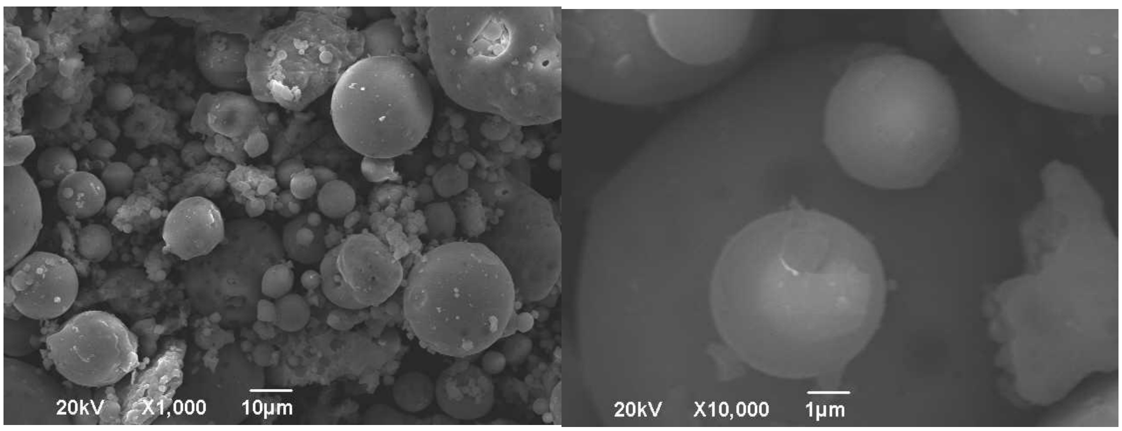

Figure 4. Fly ash images taken witn SEM

Mixtures design are given at table 6 with aggregates in a saturated condition. Results in CS tests at 28 and 91 days, EM at 28 and 91 days and porosity at 91 days are given at table 7, 8 and 9 correspondely.

Table 6. Mixtures design

	SCV-40	SCV-20	MR	ACV-10	ACV-20	SCV-40	SCV-20	MR	ACV-10	ACV-20
w/b	0.5	0.5	0.5	0.5	0.5	0.7	0.7	0.7	0.7	0.7
water (Kg/m^3)	200.9	202.9	205	204	202.9	202.1	203.5	205	204.3	203.5
Cement (kg/m^3)	241.1	324.7	410	407.9	405.9	173.2	232.6	292.9	291.8	290.8
FA (kg/m^3)	160.7	81.2	0	40.8	81.2	115.5	58.2	0	29.2	58.2
Coarse aggregates (kg/m^3)	823.7	832	840.4	836.1	832	828.4	834.3	840.4	837.4	834.3
Fine aggregates (kg/m^3)	646.4	665	683.9	643.1	602.6	745.7	759.7	773.9	744.4	715
Slump (mm)	30	50	30	50	40	60	100	30	160	140
Air (%)	4.2	3.8	3.9	4	3.9	4.3	4.1	4.2	4	4.1
Unit weight (kg/m^3)	2086	2124	2180	2145	2125	2071	2100	2143	2120	2114

Table 7. Compressive Strenght results

Mixture	w/b	Average strenght (kg/cm^2)	Standard deviation (kg/cm^2)	Variation coefficient (%)	Average strenght (kg/cm^2)	Standard deviation (kg/cm^2)	Variation coefficient (%)
		28 days			91 days		
SCV-40	0.5	232.9	8.1	3	272.9	17.1	6
SCV-20	0.5	300.0	17.0	6	328.4	31.3	10
MR	0.5	329.5	12.7	4	360.6	17.1	5
ACV-10	0.5	335.3	9.9	3	358.9	17.0	5
ACV-20	0.5	328.2	5.4	2	356.7	13.9	4
SCV-40	0.7	145.9	7	5	182.6	15.1	8
SCV-20	0.7	206.3	12.8	6	241.5	15.2	6
MR	0.7	275.1	7.2	3	295.6	10.6	4
ACV-10	0.7	241.1	5.5	2	285.0	7.0	2
ACV-20	0.7	228.2	3.0	1	283.1	2.3	1

Table 8. Elastic Modulus results.

Mixtures	w/b	f'c (kg/cm^2)	Average EM (kg/cm^2)	Standard deviation (kg/cm^2)	Variation coefficient (%)
SCV-40	0.5	232,9	200544,4	11136.7	6
SCV-20	0.5	300,0	218886,6	11208.6	5
MR	0.5	329,5	234237,5	32788.9	14
ACV-10	0.5	335,3	241605,9	12205.2	5
ACV-20	0.5	328,2	235716,8	3842.9	2
SCV-40	0.7	145,9	157068,7	3886.7	2
SCV-20	0.7	206,3	189455,2	4494.7	2
MR	0.7	275,1	215601,9	11315.6	5
ACV-10	0.7	241,1	210051,6	7107.6	3
ACV-20	0.7	228,2	201662,4	8718.5	4

Tabla 9. Porosity in concrete at 91 days results.

Mixture	w/b	Age (days)	Average porosity (%)	Standard deviation (%)	Variation coefficient (%)
SCV-40	0.5	91	24.3	0.56	2
SCV-20	0.5	91	22.0	0.13	1
MR	0.5	91	21.5	0.46	2
ACV-10	0.5	91	22.9	0.55	2
ACV-20	0.5	91	23.1	0.59	3
SCV-40	0.7	91	25.3	0.2	1
SCV-20	0.7	91	23.5	0.22	1
MR	0.7	91	21.8	0.68	3
ACV-10	0.7	91	23.9	0.30	1
ACV-20	0.7	91	23.1	0.49	2

4. DISCUSSION

Coarse aggregates did not satisfy with the particle size distribution recommended by ASTM C 33. It has unsufficient amount of 3/8" aggregate size due to a lack supervisión on the grinding proccess. Absorption value of coarse aggregates is high due to his porosity and low density. Fine aggregate satisfied with ASTM C 33 requierements. According with the Fineness Module, it is classified as a medium sand. However, the absorption and density are similar to coarse aggregate values. Fly ash satisfied the oxide composition with a 91.1% value, therefore is classified as a FA class C; also obtained a PAI of 75% at 7 days and 82% at 28 days according to ASTM C 618. Density of FA is lower than cement and aggregates; however, the measured value is within regular values as Neville mentioned at 1998. However, FA does not satisfied the fineness requirement to be considered as a Pozzolan within concrete because only 28.83% has the minimum particle size; however, grinding was not considered due to the main objective of use FA directly from the source and observe mechanical properties influence. When Fly ash is added as a mineral addition, a volume percent of fine aggregate is substituted. According to ASTM C 33, fly ash will increase the fineness amount in the concrete considering FA as a fine aggregate, however the fineness of FA proportioned an increase on packing within concrete. Image analysis from SEM (figure 4) confirm the spherical shape particle of FA.

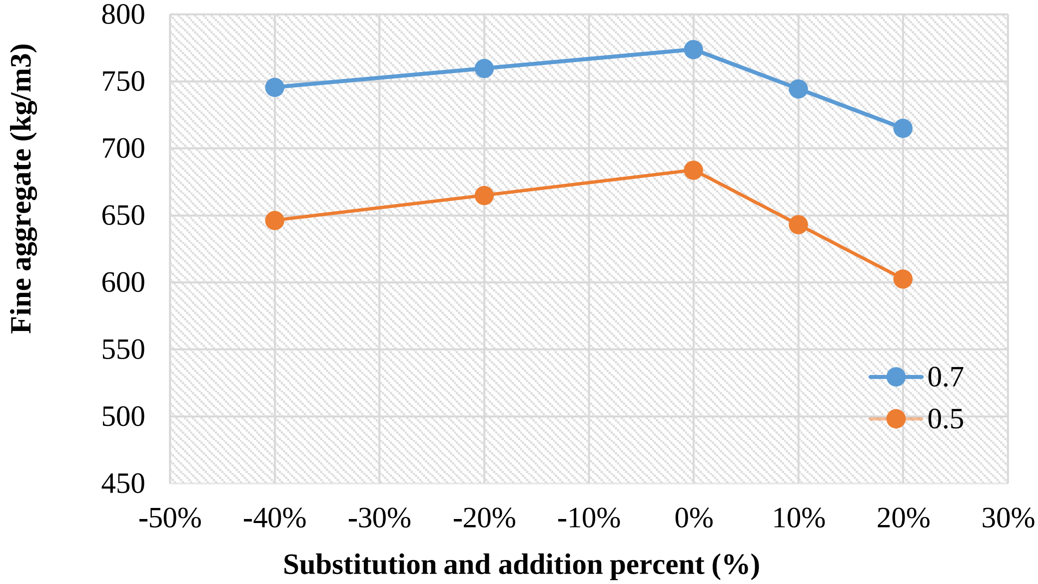

Figure 5. Fine aggregate variation

Volume of fine aggregate in mixtures decreased with fly ash addition both addition and substitution of portland cement (figure 5) due to a difference of densitys between fly ash and portland cement. Fly ash occupies more volume than portland cement when this was substituted in mixtures. Therefore, an increase on cementitious volume paste and a decrease on fine aggregate volume result in raw materials saving.

Compressive Strenght on concrete specimens with w/b of 0.5, mixture ACV-10 reached a CS higher than 334.4 kg/cm^2 (value recommended at ACI 211.1 at 28 days, no more mixtures reached the target CS. In concrete specimens of 91 years age, mixtures MR, ACV-10 and ACV-20 obtained higher CS than values recommended from ACI, however, mixtures ACV-10 and ACV-20 obtained less CS than the reference MR.

Compressive strength on all concrete specimens with w/b of 0.7 (except SCV-40) obtained a higher value than target value from ACI 211.1 of 200 kg/cm^2 at 28 and 91 days age. Fly ash did not improve the CS in any mixture either substitution or mineral addition compared with reference mixture, therefore, the increase on compressive strength from 28 days to 91 days is due to cement. A correlation between w/b ratio, substitution/adition percent of cement by fly ash and concrete age was made through a variance analysis (ANOVA) to predict compressive strength of concrete (table 10). The independent variables were concrete age, real w/c ratio, volume relation of FA and total volume of mixture and dependet variable was compressive strength as is shown in Table 10. Through a multiple regression in ANOVA programa equation 1 was obtained.

$$f'c = (0.54) * (age) - 291.21 * (^{w}/_{c}) + 312.12 * \left(^{Vol.F.A.}/_{m^3}\right) + 447.5 \qquad (1)$$

Where: f' c = CS in kg/cm^2, Age= Concrete age in days, w/c = real w/c ratio, Vol. F.A./m^3 = Relation of FA volume per m^3.

Table 10. Relation between concrete age, w/b ratio, real w/c, real FA volumen/total mixture volumen and compressive strenght

Mixture	w/b	Age (days)	w/c real	Vol F.A./total vol	CS (kg/cm^2)	Calculated CS (kg/cm^2)	Erro percent (%)
SCV-40	0.5	28	0,8	0.05	232.9	236.3	-1%
SCV-20	0.5	28	0,6	0.03	300.0	288.9	4%
MR	0.5	28	0,5	0	329.5	317.1	4%
ACV-10	0.5	28	0,5	0.01	335.3	321.2	4%
ACV-20	0.5	28	0,5	0.03	328.2	325.3	1%
SCV-40	0.5	91	0,8	0.05	272.9	270.6	1%
SCV-20	0.5	91	0,6	0.03	328.4	323.1	2%
MR	0.5	91	0,5	0	360.6	351.4	3%
ACV-10	0.5	91	0,5	0.01	358.9	355.5	1%
ACV-20	0.5	91	0,5	0.03	356.7	359.5	-1%
SCV-40	0.7	28	1,2	0.04	145.9	134.6	8%
SCV-20	0.7	28	0,9	0.02	206.3	213.7	-4%
MR	0.7	28	0,7	0	275.1	258.9	6%
ACV-10	0.7	28	0,7	0.01	241.1	261.8	-9%
ACV-20	0.7	28	0,7	0.02	228.2	264.7	-16%
SCV-40	0.7	91	1,2	0.04	182.6	168.9	8%
SCV-20	0.7	91	0,9	0.02	241.5	248.0	-3%
MR	0.7	91	0,7	0	295.6	293.2	1%
ACV-10	0.7	91	0,7	0.01	285.0	296.1	-4%
ACV-20	0.7	91	0.7	0.02	283.1	299.0	-5%

Considerations to use equation 1 are given in the following order:

- Concrete has to be made with CPC 30R and Class F Fly ash
- Curing regime of concrete should be with limewater for up to 28 days.
- High absorption Crushed limestone Aggregates with nominal diameter of ¾¨ should be used.
- Concrete design should be made according to ACI 211 recommendations with the changes explained in “experimental details chapter” and concrete design to achieve a target slump between 7.5 and 10cm.

Elastic Modulus of each mixture were calculated according to Complementary Technical Standards of Norms Construcion from Federal District (NTC RDF spanish acronym) equations for class 2 concrete and volumetric weight lower than 2200 kg/m^3 (equation 2), ACI 318 standard equations for concrete with unit weight between 1440 kg/ m^3 and 2480 kg/m^3 (equation 3) and equations from Facultad de Ingeniería of Universidad Autónoma de Yucatán (FIUADY) research made by Hernández at 2013, where a relation between aggregates density and square root of compressive strength was stablished to predict Elastic Modulus in concretes with HACLA (equation 4). Comparision of results are given in table 11.

$$E = 8000 * \sqrt{f`c} \qquad (2)$$

$$E = W_c^{1.5} x\ 0.14 \sqrt{f'c} \qquad (3)$$

$$E = 2273.69\ x\ GEAFxGEAG\ x\sqrt{f'c} \quad (4)$$

Where: E= EM en kg/cm^2, Wc= Unit weight of concrete in kg/m^3, f'c= CS in kg/cm^2, GEAG = Specific gravity of coarse aggregate (SSS), GEAF = Specific gravity of fine aggregate (SSS), f'c (kg/cm^2)= CS.

Table 11. Comparision between obtained data and models to determine EM

Mixture	w/b	f'c $^{1/2}$	EM obtained (kg/cm^2)	EM (NTC RCDF)	EM (Hernández 2013)	EM (ACI 318)
SCV-40	0.5	15,26	200544,4	122085,9	194809,4	201300,4
SCV-20	0.5	17,32	218886,6	138561,8	221099,6	237284,3
MR	0.5	18,15	234237,5	145219,3	231722,9	258740,1
ACV-10	0.5	18,31	241605,9	146485,2	233742,9	254684,9
ACV-20	0.5	18,12	235716,8	144936,9	231272,4	248545,4
SCV-40	0.7	12,08	157068,7	96627,9	154186,9	161137,5
SCV-20	0.7	14,36	189455,2	114902,4	183346,9	193506,7
MR	0.7	16,59	215601,9	132691,5	211732,6	230386,5
ACV-10	0.7	15,53	210051,6	124211,4	198201,2	212120,9
ACV-20	0.7	15,11	201662,4	120860,9	192854,8	205540,9
Difference between obtained EM and models to determine EM.				>39%	>2%	<5%

Correlation between obtained and calculated results with NTC-RCDF equations was made by calculating K function of Elastic Modulus obtained in experimental work with a lineal regression in Microsoft Excel. Consideration as f'c= 0 was taken to obtained a EM of 0 (figure 6). This is shown in equation 5.

$$E = 13079\sqrt{f'c} \quad (5)$$

Where: E= ME in kg/cm^2, f'c= RC in kg/cm^2. Obtained equation has a K of 60% up than NTC RDF standards. Consequently, the use of NTC RDF equations allow overstating concrete structures design, then, regional standards should be done according to aggregates type.

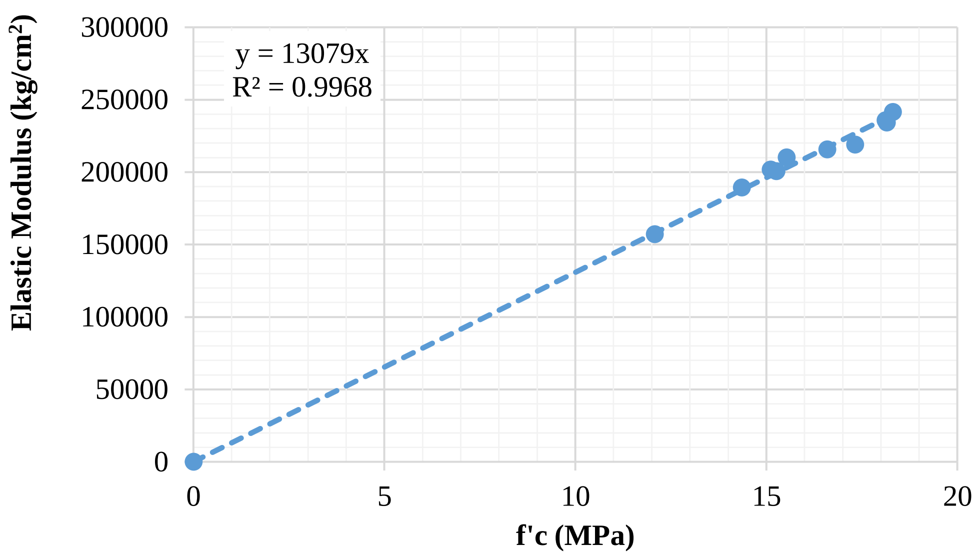

Figure 6. Lineal regression of results

Fly ash did not have a significant influence in Porosity, density and absorption results. However, it was observed a small increase of porosity when fly ash was added (Table 9).

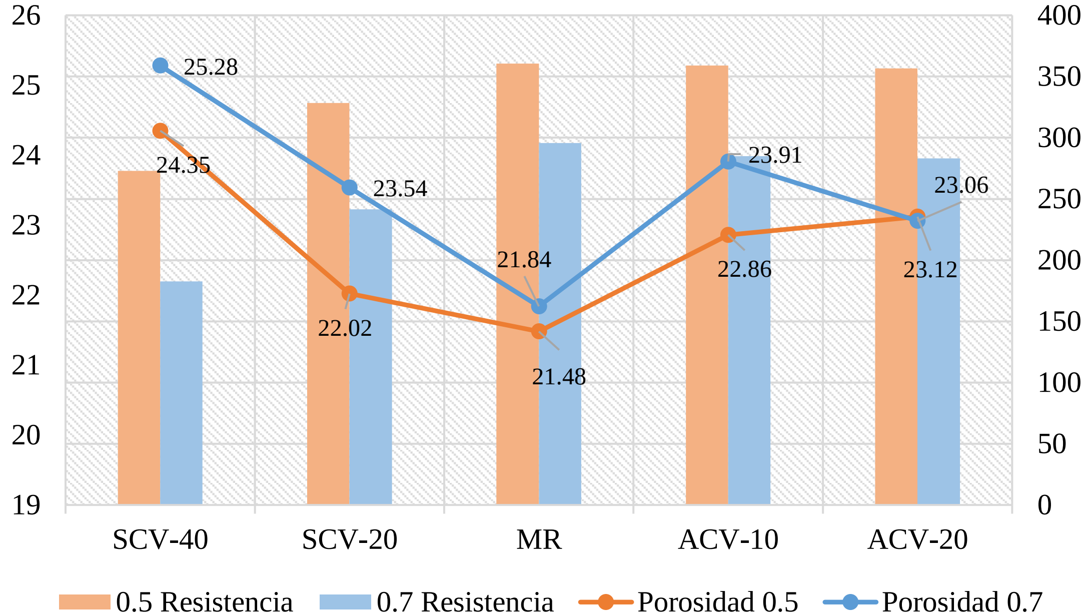

Figure 7. Porosity and compressive strenght

Porosity is a main factor in mechanical properties and durability in concrete, the higher porosity is lower mechanical properties are obtained, resulting in a lack of durability performance in aggressive environments (Mehta & Monteiro, 1998). It was observed a tendence on compressive strength decrease when porosity increases (figure 7) taking MR mixtures as a reference. However, porosity percents have a low variation range of +/- 4%. Only mixtures of cement substitution by Fly Ash obtained a significant decrease in compressive strength when porosity increases compared with MR mixtures. Experimental work from Solís and Moreno, 2011 concluded that porosity criateria in concrete with HACLA is not adecuaded to determine the quality of concrete. Relation between CS and Porosity results in this work seems to corroborate Solís & Moreno conclusions.

5. CONCLUSIONS

Class F Fly Ash from Nava, México is recommended to use in concrete with HACLA as an inert fine aggregate due to the following arguments:

- Even if compressive strength was not increased by fly ash this remained
- Fly ash concrete employment allows a usefulness to this material currently destined to environment affecting nearby urban zones.

According to mechanical properties results, the following conclusions were stablish:

- A lack of Pozzolanic Activity from Fly ash was obtained, even when certain requierements were satisfied; fly ash was not able to enhance quality in concrete with HACLA. However, fly ash can be used adjusting mixtures design with equation 1 and 5 to maintain target CS and EM, even without modifications to fly ash such as grinding.
- Equations 1 and 2 to determine CS and EM for concretes with HACLA were calculated to avoid overstating in concrete structures from Yucatán and optimize the raw materials.

Fineness of fly ash should be increase by grinding to enhance the pozzolanic activity and packing in concrete.

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Effect of buttering in mechanical properties of dissimilar metal weld joints for reinforcement bars in concrete structures

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ABSTRACT

In this work, the influence of Inconel 182 as buttering material in the mechanical properties of dissimilar metal welds between plain carbon steel and stainless steel bars welded using SMAW has been investigated by using microstructural analysis, Vickers microhardness testing, and tensile tests. Welding with SMAW process is commonly applied in field welding of concrete structures. Therefore, this process was selected for this work. The results indicated that even if the joints contain defects generated by the welder, the mechanical properties of dissimilar welded joint without buttering are higher than the properties of joints with buttering. This methodology is proposed for the rehabilitation of concrete structures with steel bars as reinforcement, which are located in maritime enviroments.
Keywords: buttering, dissimilar metal weld joints; mechanical properties; microstructure; defects.

RESUMEN

En este trabajo se ha investigado la influencia de utilizar Inconel 182 como material de "mantequillado" en las propiedades mecánicas de uniones disimiles acero al carbono-inoxidable manufacturadas por soldadura con electrodo recubierto. Para ello se han empleados las técnicas de análisis microestructural, ensayos de microdureza Vickers, y ensayo de tensión. Se ha empleado el proceso de soldadura con electrodo recubierto por que es el más común en la soldadura de campo para la construcción. Los resultados demuestran que aun cuando las uniones disimiles sin "mantequillado" presentan defectos, sus propiedades mecánicas son superiores a las de las uniones soldadas con "mantequillado". Esta metodología es propuesta para la rehabilitación de estructuras de concreto para ambiente maritimo con barras de acero como refuerzo.
Palabras clave: mantequillado, uniones soldadas con materiales disimiles, propiedades mecánicas, microestructura, defectos.

RESUMO

Este artigo investiga a influência da utilização de Inconel 182 como um terceiro material, nas propriedades mecânicas de diferentes aços inoxidáveis soldados a aços carbono com uso de eletrodo revestido. Para tanto foram usados os seguintes métodos: análise microestrutural, ensaios de microdureza Vickers e ensaios mecânicos de tensão. Tem sido empregado o processo de solda com eletrodo revestido, porque é o mais comum no campo de solda para a construção. Os resultados mostram que quando as uniões são realizadas sem esse terceiro material, as propriedades mecânicas são superiores às das juntas soldadas com auxílio do Inconel 182. Esta metodologia, ora criticada, tem sido proposta para a reabilitação de estruturas de concreto em ambiente marinho, com uso de barras de aço inox como reforços.
Palavras-chave: juntas soldadas com materiais diferentes; propriedades mecânicas; microestrutura; defeitos.

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1. INTRODUCTION

Concrete structures located in maritime environments are deteriorated with time by degradation of the reinforcement bars made of carbon steel; as it has been already reported by Hernández, and Mendoza (2006). In the same way, Tabatabai et al. reported that there are two main reasons for the deterioration of the steel bars of the concrete structures: carbonation caused by reduction in the alkalinity of the concrete and the destabilization of the iron oxide film due to chlorine ions (2009). Then, corrosion of the steel reinforcing bars represents the biggest problem for concrete structures.

Perez-Quiroz et al. have reported that the corrosion products generated during the plain carbon steel corrosion process cause tensile stresses within the concrete, which generates internal cracks and the loss of structural bonding between the reinforcement bar and the concrete (2008).

As example of the importance of corrosion resistance of the material for reinforcement bars in concrete structures, there is a pier in Yucatan, México, which was built between 1937 and 1941; whose concrete structures were made using 304 stainless steel bars as reinforcement. The result is that no rehabilitation has been necessary in the pier´s structures.

In contrast, Klueh and King have reported another pier in the same region of Mexico, whose reinforcements bars were of plain carbon steel, thoroughly damaged by corrosion of the reinforcement bars (1982). In the same way, Istrati, has reported AISI 304 austenitic stainless steel as a good candidate for welding construction for reinforcement bars.

Figure1. Comparison between the pier with structures made with stainless steel (right) and the pier with structures made with carbon steel (left) (Klueh, R. L., & King, J. F. 1982).

As it has been mentioned above, the study of different materials for substituting carbon steel of the reinforcement bars is a very important issue, since corrosion resistance materials increase the service life of reinforcement bars in concrete structures. However, the proposed materials must be joined to plain carbon steel in order to reduce maintenance cost and increase life service of the concrete structure. In order to reach this, topics such as dissimilar metal welding and the preservation of the mechanical properties of this weldment, must be investigated.

Dissimilar metal welds have been studied since 1935, but such studies are based on failure reports, research, interviews with different producers and users of such joints; with the aim of gather information about the performance of such welds. In this literature, it has been mentioned that the type of bevel of the joint´s preparation must be taken into account in order to get dissimilar metal welds with good properties (Lundin, 1982). In the same way, Doddy in 1992 as well as Ospina et al. in 2007 reported that in dissimilar welded joints between plain carbon steels and stainless steels, carbon diffusion from the plain carbon steel into the stainless steel takes place; moreover, the heat input of the welding promotes the precipitation of the chromium carbides or sigma phase in the grain boundaries of stainless steel.

These phenomena must be avoided to get good corrosion properties in the dissimilar metal welds and there have been efforts in order to propose methodologies to get the best dissimilar metal weld. For instance, Fuentes et al. have reported the properties of dissimilar weld joints between ASTM A537 and AISI 304L austenitic stainless steel using an ER-308L electrode as filler metal without buttering. In this case this weld was carried out using gas metal arc welding (GMAW). In their work, they reported, high hardness values on the fusion line between the weld and stainless steel (2011). However, GMAW is not commonly applied in field welding for concrete structures construction.

In the other hand, Murugan and Parmar have reported that ferritic steel welded to austenitic stainless steel plates using Inconel 625 and 725 as filler metal but they did not use buttering. The results showed no evidence of carbon migration into the stainless steel, but weldments with post welding heat treatment showed evidence of it (1997).

In order to avoid carbide diffusion into the stainless steel in a dissimilar weld, the process of buttering has been proposed as a solution for minimizing mechanical and metallurgical problems in this type of joints. For instance, Winarto et al. have reported that samples with 10 mm in thickness buttering showed higher mechanical properties than samples with 20 mm in thickness buttering. On the other hand, macroetching analysis showed that the Heat Affected Zone (HAZ) was bigger for samples with buttering than those with no buttering (2014). In this case, they used GMAW to weld plates with a “V” bevel preparation, and E7016 as filler metal.

From the above information, it can be said that there is not much information about dissimilar metal welds in bars using Shielded Metal Arc Welding (SMAW) or the effects of buttering using Inconel 182 in stainless steel as filler metal. Therefore, in this work, the influence of Inconel 182 as buttering in the mechanical properties of dissimilar metal welds between plain carbon steel and stainless steel bars welded using SMAW has been investigated. Considering that SMAW is commonly applied in field welding, this methodology can be proposed for the rehabilitation of concrete structures with steel bars as reinforcement.

2. EXPERIMENTAL PROCEDURE

First, it must be mentioned that for this work all materials, electrodes, and welding process were selected by the criteria of price and use in the welding field. So then, the SMAW process was selected in this work because it is commonly applied in field welding of the reinforcement bars for concrete structures; as well as the single bevel joint, has been selected because this type of joint is the most common joint preparation for reinforcement bars. The base materials were bars of ASTM A615 carbon steel and AISI 304 austenitic stainless steel.

The filler metal was AISI 309L stainless steel and buttering, was carried out using Inconel 182 in a 2 mm thickness layer welded to stainless steel. AISI 309L stainless steel was selected as filler metal because it is commonly applied in stainless steel welds, since it has a chemical composition of the base materials similar to AISI 304. Moreover, AISI 309L and Inconel 182 can be found in electrodes for SMAW with a diameter of 3.2 mm for each material. In fact, these electrodes are sometimes used as buttering using GMAW o GTAW process in weld joints between stainless steel and plain carbon steels.

Besides, Inconel 182 was selected for the buttering because its high nickel content and relative low price. Buttering was carried out in 5 stainless steel bars with an Inconel 182 electrode, in order to determine the effect of this material with the stainless steel, and at the same time in order to observe the effect of AISI 309L welded to plain carbon steel. The welding process was carried out by SMAW process with an electrode 309L as filler metal. The chemical composition of Inconel used is reported in Table 1.

Table 1. Chemical composition of Inconel 182 electrode

NiCrFe7 (182)	C	Si	Mn	P	S	Cr	Ni	Nb	Ti	Fe
Chemical composition (%)	<0.1	<1	5-9.5	<0.03	<0.015	13-17	>59	1-2.5	<1	<10

The preparation of the specimens was carried out according to the following procedure: 10 bars of AISI 304 stainless steel and 10 bars of A615 plain carbon steel with 19 mm in diameter and 100 mm in length were cut as base metal for the welding joints. The stainless steel bars were machined at 45° as preparation of the single bevel joint.
10 joints were prepared according to NMX-H-121-1988 and ANSI/AWS D1.4-M-2005. Buttering was applied on 5 stainless steel specimens with the 45° bevel groove. This process was carried out so in order to avoid carbide and sigma phase precipitation in the buttering region. One must to keep in mind that this Inconel 182 contains chromium, which can combine with the carbon that diffuses from the plain carbon steel. The welding machine was a Castolin Eutectic, Master NT2000 AC/DC; the welding parameters applied are shown in Table 2.

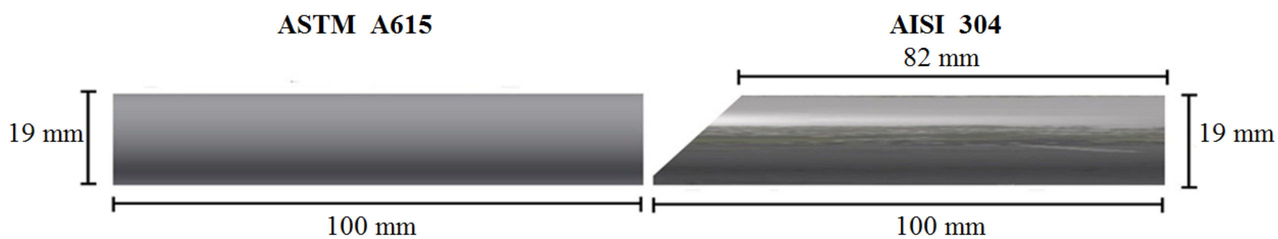

Figure 2. Schematic diagram of the single bevel dissimilar welded joints. The left bar was the ASTM A615 Steel and the right bar is AISI 304 stainless steel.

Table 2. Welding parameters for the welding process.

Bar diameter (mm)	19
Electrode Diameter (mm)	3.2
Voltage (V)	22
Current (A) DC-PI	90-95
Travel speed (mm / min)	45
Heat input (kJ / mm)	2.5

In order to determine the variations in hardness among the base metal, filler metal and heat affected zone, microhardness profile was measured in the longitudinal direction of each specimen. The measurements were averaged and then they were verified to meet ASTM E384. The microhardness testing was carried out using a High Quality microdurometer model MMT-1. The load used was 300 kgf; the indentation was observed and measured at 400X.
The tensile test was performed according to ASTM E8-M in an Instron FAST TRACK universal testing machine, model 8801. The samples were tested at 3 MPa/s and the deformation was measured using an extensometer.
Finally, these dissimilar welded joints were microstructural characterized by optical microscopy; the samples were prepared according to ASTM E3 and microstructure was revealed using Berahas etchant for the stainless steel and NITAL 2 for the plain carbon steel. The microstructural analysis was carried out using a NIKON 440 metallurgical microscope. The macrostructure of the specimens was revealed according to ASTM 340 and analyzed using a LEICA stereographic microscope.

3. RESULTS AND DISCUSSION

The chemical composition of ASTM A615 steel was determined by optical spectrometry and the results were applied to compute the carbon equivalent (Ceq) for the ASTM A615 steel bars. The calculated Ceq is 0.3683, which indicates good weldability of the plain carbon steel because it is lower than the 0.55. This value is stipulated in ASTM A706/A706M.
Besides, it was necessary to calculate the chromium equivalent (Creq) of the stainless steels, in order to determine the susceptibility of the stainless steel to sigma phase precipitation. In this case the Creq. is higher than 17; then this stainless steel is prone to sigma phase precipitation in the heat affected zone. This fact indicated that this steel should not be heat treated and the heat input of the welding process must be as low as possible. Table 3 shows the calculated Creq.

Table 3. Results of chromium equivalent.

Material	Creq (%)
AISI 304 stainless steel	19.22
AISI 309L filler metal	24.5

The macrostructures of the welded specimens were observed through stereographic microscope. The results of the specimens with buttering showed evidence of cracks in the filler metal but the cracks had its origin in the buttering layer of the specimens. An example of the cracked specimens is seen in figure 3B; while figure 3A shows the macrostructure of the specimens welded without buttering and no evidence of cracking is observed. This fact points out that dilution between the Inconel 182 of the AISI 309L of the filler metal induced cracking. This matches the results of Evans, who reported that nickel alloys joined to stainless steel are prone to hot cracking (1962).
In the same way, the chemical composition of the Inconel 182 and the AISI 309L indicated high nickel in both alloys; so then it is feasible that the nickel equivalent of the joint increases with the dilution of both Inconel 182 and stainless steel. In literature, it has been reported previously by authors as: Jang, Ospina, Fuentes, that a percentage of ferrite between 7 and 12 is recommendable for avoiding hot cracking in austenitic stainless steel (Jang, C., Lee, J. H., Jung, S. Y., Kim, J. S., & Jin, T. E. 2006; Ospina, R., Aguirre, H., & Parra, H. 2007; Fuentes, A. L. G., Centeno, L., García, R. D. S., & Del Rosario, A. V. 2011).
Macrographs showed evidence of lack of fusion located near the plain carbon steel in the single bevel joints (figure 3A). This is a serious defect because in weldments it functions as stress riser during service and this leads to failure of the welded component. However, this defect is commonly originated by the lack of skill of the welder; so even if the welder is very skilled this defect can be induced in some welds. Another cause of lack of fusion is a low welding current. In this case the welding current is in the recommended range for these materials (between 75A and 110A); so this cause can be discarded. In each case, it is important to mention that even though some samples had lack of fusions, they showed good tensile strength and this is very important for the structure performance. This fact is discussed in detail later on.

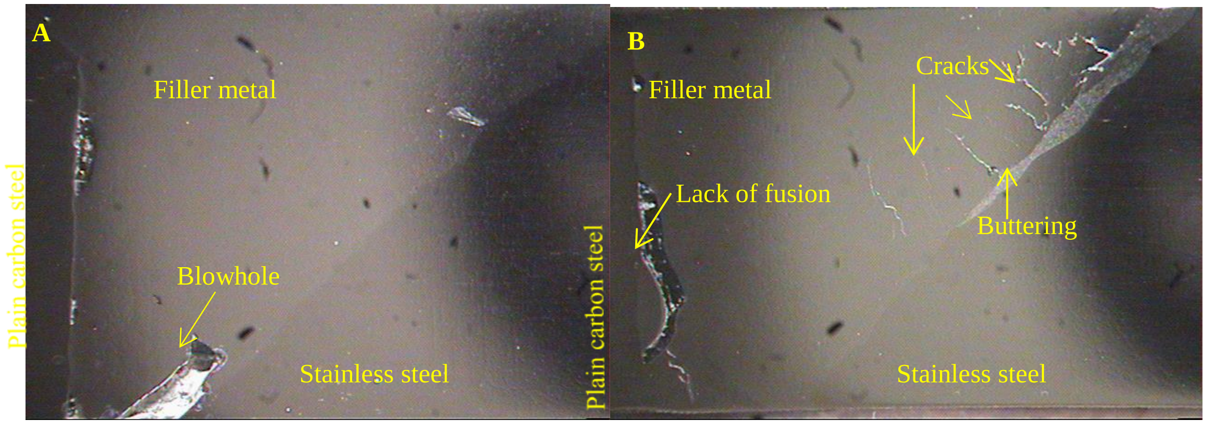

Figure 3A. Simple bevel without buttering. Figure 3B. Single Bevel with buttering.

3.1 Microstructures of welded joints

Figure 4 shows the microstructure of the plain carbon steel as base metal; there can be seen equiaxial bands of ferrite grains and bands of pearlite. In the case of figure 5, it shows the microstructure of austenitic stainless steel as base metal. There can be seen twinned grains but there is no evidence of sigma phase or metallic carbides precipitated in the microstructure.

In figure 6, the Heat affected Zone (HAZ) of the interface between the plain carbon steel and filler metal is shown. In the plain carbon steel there is no abnormal grain growth of the ferrite grains or cracking is observed in any specimen.

Moreover, no evidence of carbon diffusion from the carbon steel into the AISI 309L was found. This fact is very important because it agrees with the results of Murugan and Parmar and it indicates a no sensitization on the filler metal. Therefore it is feasible to weld low carbon steel to stainless steel using AISI 309L as filler metal without buttering and at the same time with no evidence of sensitization of the stainless steel.

Figure 4. Microstructure of the plain carbon steel at 100X.

Figure 5. Microstructure of the AISI 304 stainless steel at 100X.

Figure 7 shows the microstructure of the interphase between plain carbon and AISI 309L steels; there can be seen no evidence of sensitization in the grains boundaries of AISI 309L. In spite of applying SMAW welding, no cracking of carbide precipitation was found in the AISI 309. This is important if one considers that SMAW normally induces heat inputs higher than heat input of GTAW or GMAW.
In the other hand, the microstructure of the plain carbon steel consists of martensite; which is a typical HAZ microstructure of carbon steel weldments joint using SMAW. One has to notice that no evidence of cracking was found in any sample in the interphase between plain carbon and AISI 309L steel. Then it is possible to weld by SMAW plain carbon steels to austenitic stainless steels without filler metal using AISI 309L as filler metal.

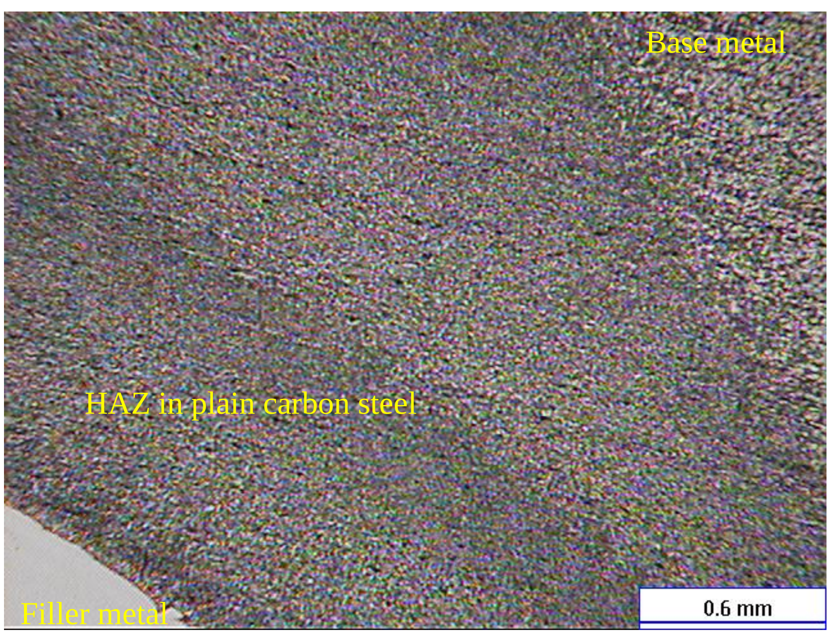

Figure 6. Microstructure at 50X of the weld between the plain carbon steel and the filler metal. No evidence of either, cracks or abnormal grain growth, is seen.

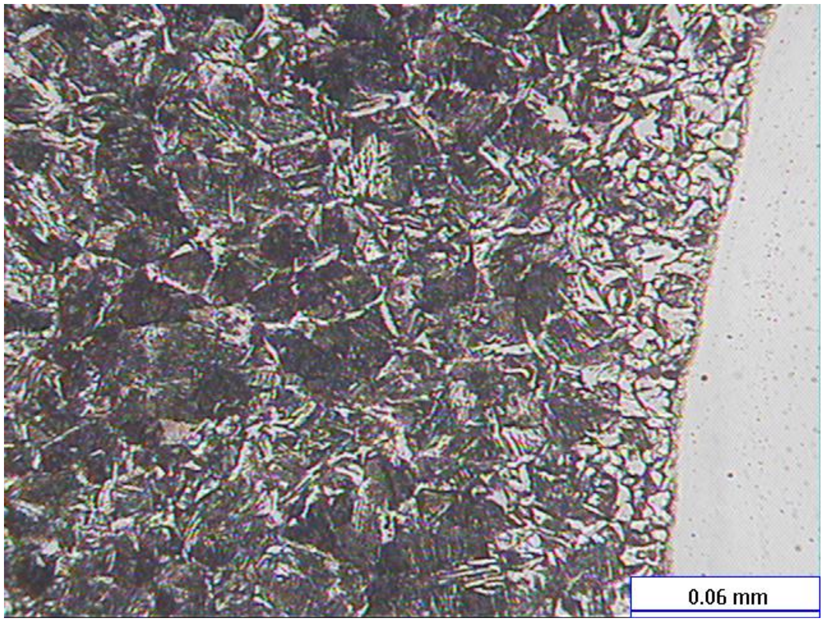

Figure 7. Microstructure at 500X of the weld between the plain carbon steel and the filler metal. There can be seen no evidence of precipitated carbides in the grain boundaries of AISI 309.

Figure 8 shows the microstructure of the interphase among the filler metal, the buttering, and the AISI 304 stainless steel. There can be observed cracks between the buttering and the filler metal. In the same micrograph can be seen that the crack´s origin is located in the interphase between the buttering and the filler metal, this fact support the above discussed possible origin of the cracks. Besides, in the micrograph of figure 8 is also seen evidence of shrinkages among the dendrites in the interphase between the buttering and the filler metal. Furthermore, it is clear that shrinkage continues growing into the dilution zone between the material of buttering and the filler metal. This fact agrees with the results of Evans.

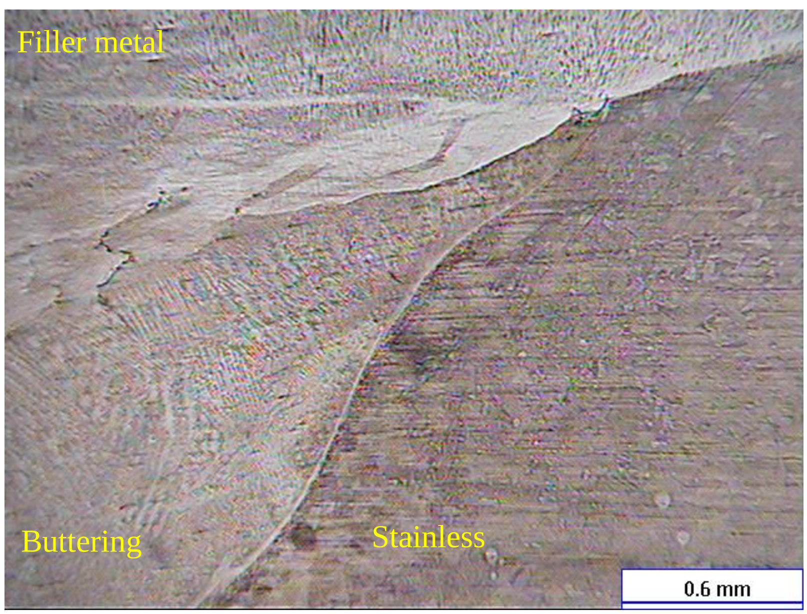

Figure 8. Microstructure of the welded joint between the buttering and the stainless steel. There can be seen cracks in the dilution region filler metal/buttering.

The above discussed microstructural evidences indicated that austenitic stainless steels welded with buttering using Inconel 182 are prone to cracking. This fact indicated that reinforcement bars for concrete structures must not be welded with buttering as long as the filler metal is properly chosen, this topic has been previously discussed in the literature (Jang, C., Lee, J., Kim, J. S., & Jin, T. E. 2008, Olden, V., Kvaale, P. E., Simensen, P. A., Aaldstedt, S., & Solberg, J. K. 2003, Shinozaki, K., Ke, L., & North, T. H. 1992. Murugan, N., & Parmar, R. S. 1997). These evidences indicate that the nickel percentage plays a very important role in the selection of the filler metal and in the buttering material.

3.2 Vickers microhardness results

Figures 9A and 9B show the microhardness profile obtained in the single bevel joint. There are seen differences between the specimen with buttering and the specimen without buttering. For instance, the average hardness of the filler metal is higher in the specimen without buttering than in the specimen with buttering. In the case of the plain carbon steels, the average microhardness in the specimen without buttering is higher than in the specimen with buttering. This fact is related to the martensite in the interphase between the plain carbon steel and the AISI 309L. In the same way, it can be mentioned that the lower average microhardness in the filler metal in the specimen with buttering is an evidence of the lack of carbon diffusion into the filler occurred. But in the case of stainless steel, the average microhardness is higher than in the same area of the specimen without buttering. This fact can be explained by dilution of nickel in the stainless steel, which causes solid solution hardening (Cunat, 2004).

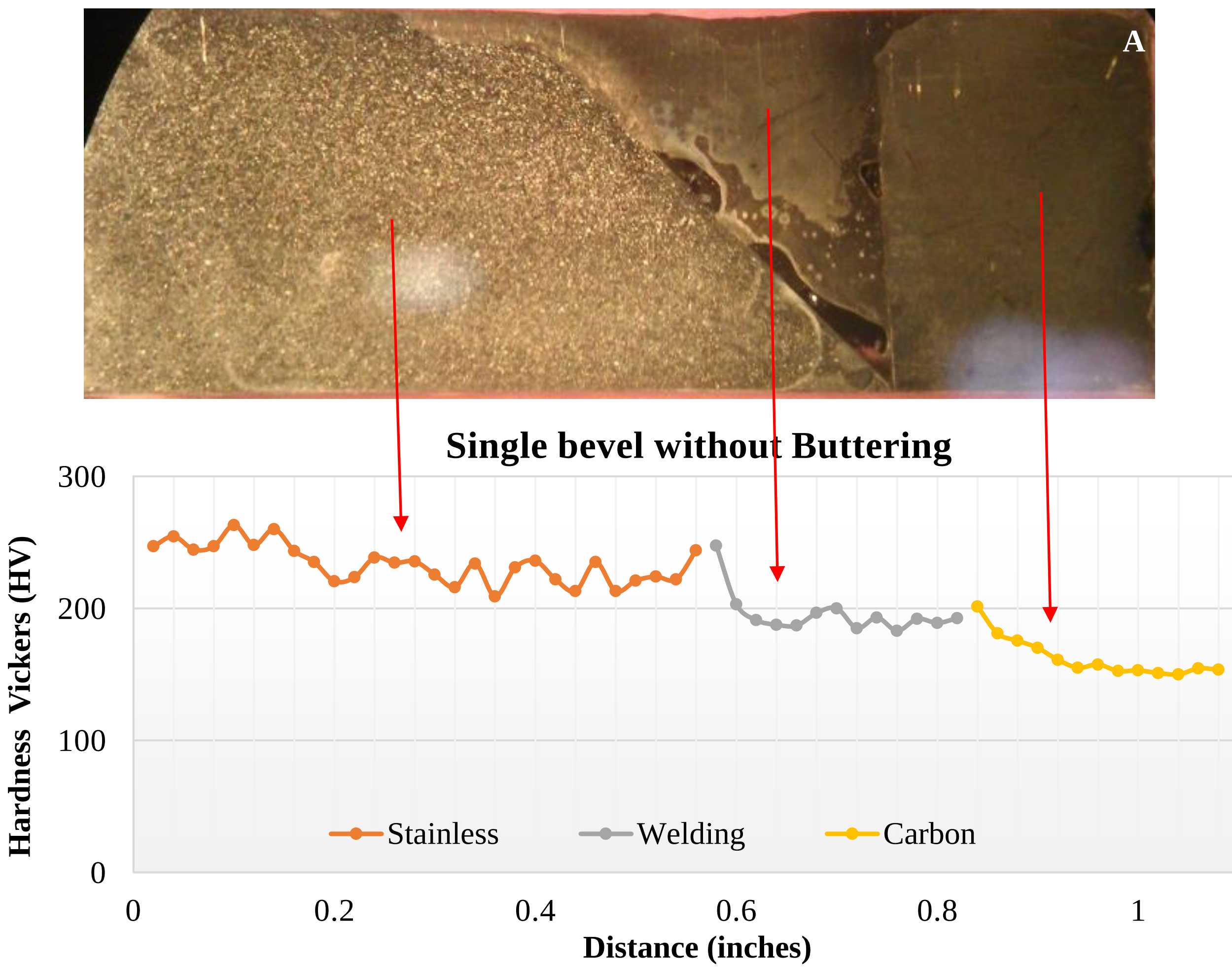

Figure 9A. Microhardness profile of the single bevel welded joint without buttering.

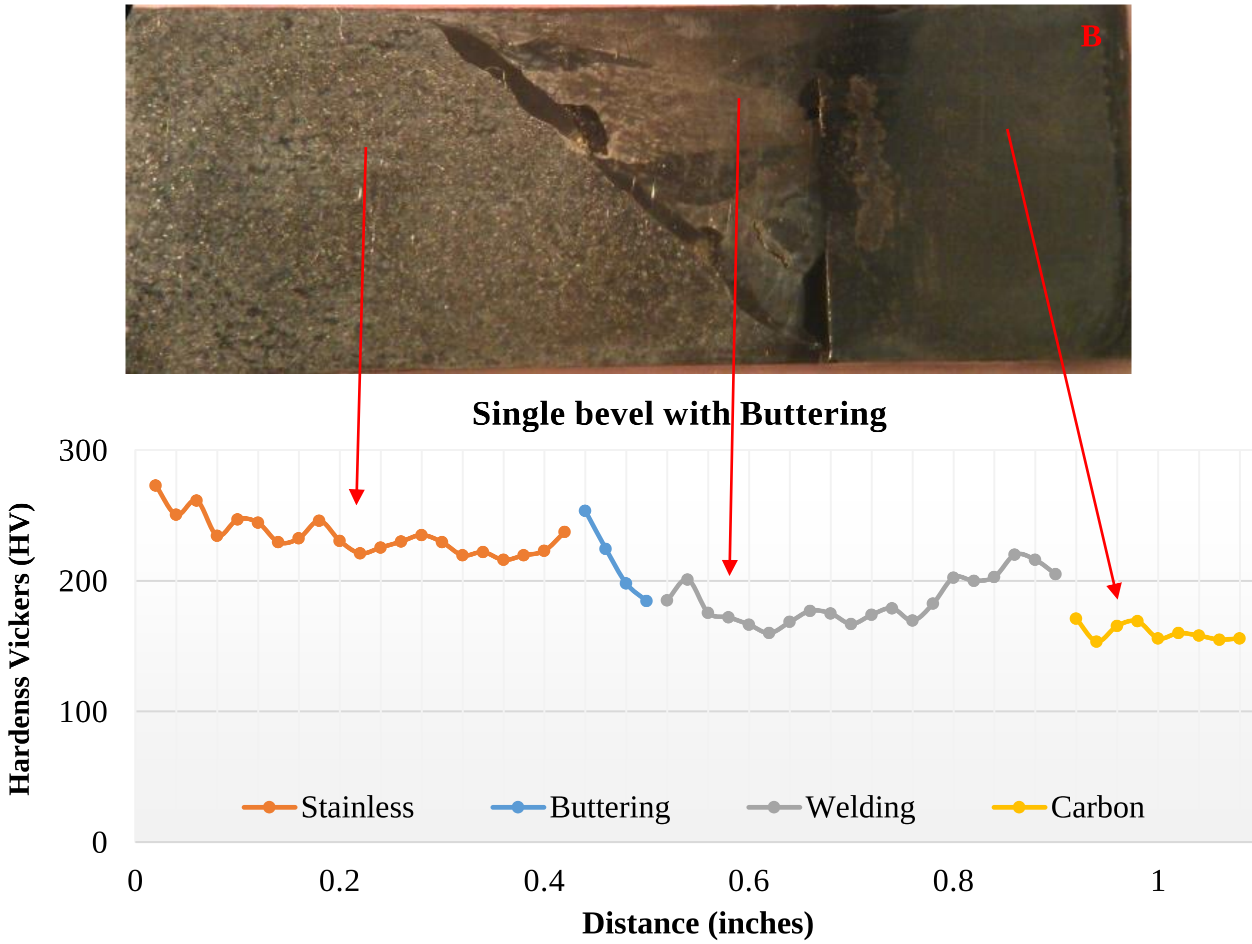

Figure 9B. Microhardness profile of the single bevel welded joint with buttering.

3.3. Tensile testing

In the engineering stress-strain curves of the single bevel specimens was observed that the specimens without buttering showed higher ultimate tensile strength (UTS) than the specimens with buttering (figure 10). This fact agrees with the results of the microhardness measurements, where it was seen that the specimens showed highest values of microhardness in the interphase of the filler metal and the carbon steel.

The yield strengths of the specimens with buttering are 50 MPa higher than the yield strength of the specimens without buttering, but the elongation of specimens with buttering is larger than in the case of the specimens without buttering. These facts can be explained by the macrostructures, which indicated that the specimens with buttering are more likely to contain defects such as lacks of fusion or cracks that acted as stress risers and they influenced the mechanical behavior during the tensile test. Nevertheless, it is important to mention that in spite of presence of defects such as lack of fusion of porosity, the specimens showed UTS higher than 200Mpa. The main influence of defects is in the elongation of the samples.

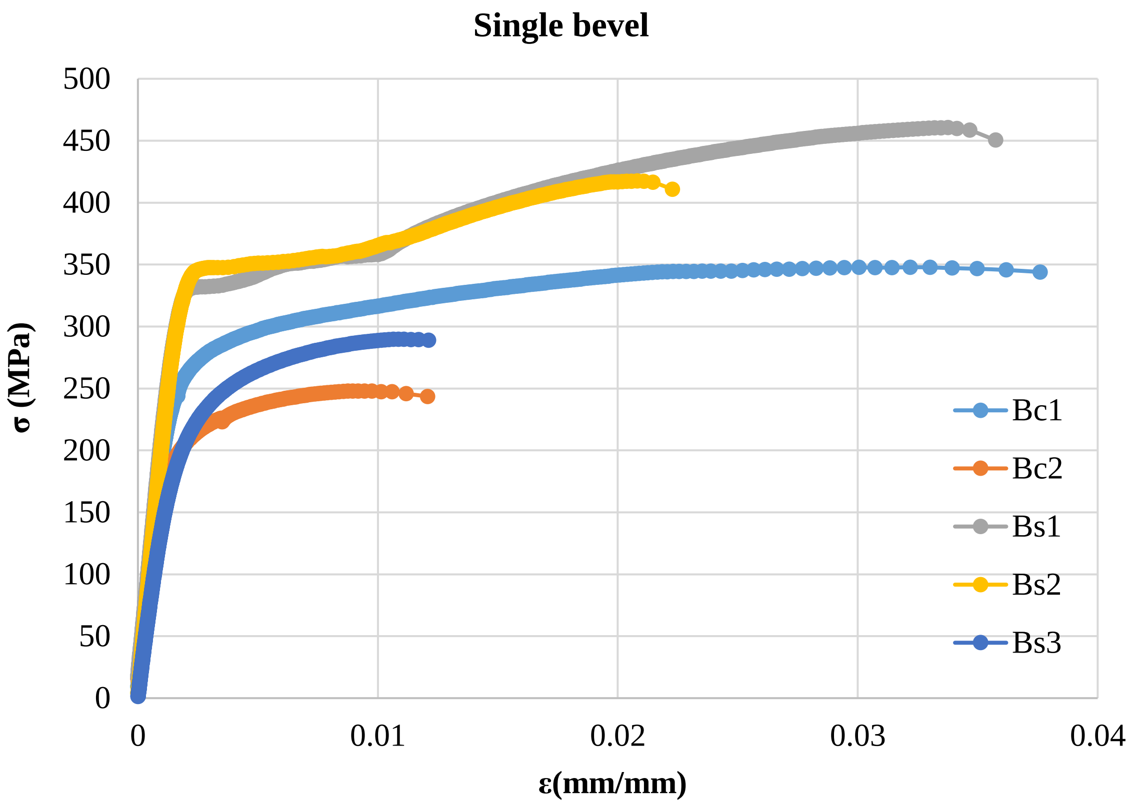

Figure 10. Stress-strain curve of single bevel specimens.Bc1, Bc2, were the samples welded with buttering; Bs1, Bs2, Bs3 were samples welded without buttering.

Regarding the fracture of the specimens, with exception of specimens that showed welding defects such as lack of fusion or blowholes, the fracture were located at the plain carbon steel side of the samples. Figure 11A and figure 11B show the fracture surface of the specimens that failed under lower fracture stress than the stress of the other specimens with the same preparation. The influence of these defects in the mechanical behavior of these specimens is proved by the fact that welding defects are revealed on the fracture surface of the specimens. For instance, figure 11A the fracture surface of the single bevel specimen with buttering. A lack of fusion on the fracture is the origin of the fracture. This is proved by the river marks indicated with black arrows. These results agree with the work of Jang et al. They showed that the fracture of dissimilar welded joints using Inconel as filler metal strongly depend on the weld (Jang, C., Lee, J., Kim, J. S., & Jin, T. E. 2008).

Figure 11A. Fracture of the specimen with single bevel specimen with buttering. Lack of fusion (red arrow) and blowholes (yellow arrow) are observed on the fracture.

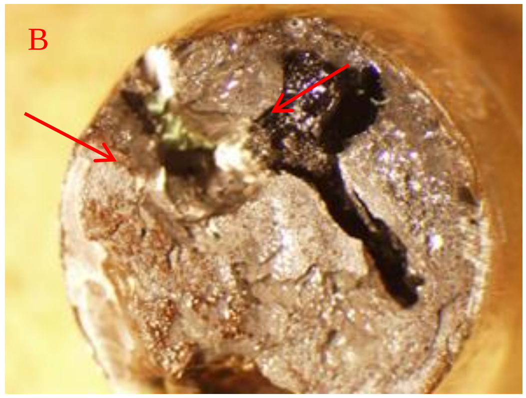

Figure 11B. Facture surface of the single bevel specimen without buttering. There can be seen lack of fusion on the fracture (red arrow).

4. CONCLUSIONS

The evidences discussed above lead to the following conclusions:
1.-Specimens with buttering are prone to show cracking in the interphases between the buttering and the filler metal.
2.-The welds between AISI 309L and plain carbon steel showed no evidence of cracking.
3.-Specimens without buttering showed higher average microhardness in the filler metal.
4.-Specimens without buttering showed better mechanical properties.
5.-The chemical composition of the materials for the buttering and the filler metal are very important for the integrity of the weldment.

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Analysis of non-conformity concrete: long time effects

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ABSTRACT
The aims of this paper is to contribute to the analysis of non-conformity concrete, focusing on long time effects. A review of compressive strength evolution, results variability and acceptance criteria was made. In addition, it is presented a case study of a nonconforming concrete used in composite structures (concrete-filled steel columns) that present 28 days strength below the specified. Considering long time effects, a nominal strength gain above the limits considered in technical standards was observed. This analysis, associated with a revision of the structural design and a carefully assessment, could help decision taking in case of non-conformity concretes.
Keywords: non-conforming; strength gain; structural safety.

RESUMO
O presente artigo tem como objetivo contribuir para a análise de concretos com não conformidades, com foco nos efeitos de longa duração. Foi realizado um levantamento dos intervenientes na análise de não conformidades: evolução da resistência à compressão, a variabilidade dos resultados e critérios de aceitação do concreto. De forma complementar, é apresentado o estudo de caso de concretos não conformes empregados em estrutura mista (pilares metálicos preenchidos) que apresentaram resistências abaixo do especificado aos 28 dias. Considerando os efeitos de longa duração, um ganho de resistência nominal acima dos limites normativos foi observado. Esta análise, aliada a uma revisão do projeto e a uma inspeção criteriosa, pode auxiliar na tomada de decisão em casos de concretos não conformes.
Palavras-chave: não conformidade; crescimento de resistência; segurança estrutural.

RESUMEN
Este artículo tiene como objetivo contribuir al análisis del concreto en casos de incumplimiento normativo, centrado en los efectos a largo plazo. Se llevó a cabo una encuesta entre los participantes en el análisis de no conformidades: evolución de la resistencia a la compresión, variabilidad de los resultados y criterios de aceptación. Complementariamente, se presenta un caso de estudio de un hormigón en incumplimiento utilizado en una estructura mixta (pilares metálicos rellenos) que mostró una a 28 días menor que la especificada. Teniendo en cuenta los efectos a largo plazo, se observó una ganancia de resistencia nominal por encima de los límites reglamentarios. Este análisis, junto con una revisión del proyecto y una inspección minuciosa puede ayudar en la toma de decisiones en casos de hormigón en incumplimiento.
Palabras clave: incumplimiento; ganancia de resistencia; seguridad estructural.

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1. INTRODUCTION

The high mechanical strength along with low production cost and easiness of casting various geometries, make concrete the most used material in constructions, standing out in technical and economic aspects. (Mehta; Monteiro, 2014). Consequently, as concrete consumption grows, is expected a growth of projected and designed builds that require evaluation of its performance regarding functions for which it was designed, combining efficiency technical aspects. A good choice of materials as well an investigation of effects of employed technologies, associated with a structural system improvement are important factors to ensure safety conditions.

In general, safety in structural design is introduced by safety partial factors that takes account inevitable imprecisions in load estimations or variability of mechanical properties of materials. Besides that, safety incorporates imperfections due to simultaneous actions that structure must support, but it also be included in these uncertainties the errors resulting from simplified design conception or capacity of redistribute action produced by eventual damages. When this aspects and coefficients are not addressed, neglected or verified, there is an increase of non-conformity cases and, thus, should be investigated. This can point out problems that put in doubt the structural design, possible repairs or total and/or partial condemnation of some elements.

Within this context, many studies and research has been made to understand non-conformity of structural concrete, addressing aspects of safety, confiability and risk analysis (Kausay; Simon, 2007; Pereira, 2008; Caspeele; Taerwe, 2011; Helene, 2011; Santiago, 2011; Santiago; Beck, 2011; Caspeele; Taerwe, 2014; Larrossa et al, 2014; Magalhães, 2014; Rao et al, 2014; Couto et al, 2015; Magalhães et al, 2015). In Brazil, the subject led the creation of a group of studies of the Brazilian Association of Structural Engineering and Consulting (ABECE), which resulted in the recommendation ABECE 001: Case studies in non-conformity of concrete. It is noteworthy that safety assessment of non-conformity structural concrete includes many stages and methods, which include extraction of concrete cores and design review with the obtained concrete compressive strength (Silva Filho; Helene, 2011).

This paper aims to review some of the main factors involved in the analysis of non-conformities in concrete. Aspects of variability of axial compressions test results, concrete acceptance criteria and compressive strength evolution in terms of long time effects are addressed. This review is complemented with a case study of a composite structure, with concrete-filled steel columns, which showed a lower compressive strength than the specified by the designer. With this analysis of long time effects on concrete compressive strength, is intended to contribute in decision making in the analys of non-conformities in concrete structures.

2. ANALYSIS OF NON CONFORMITY CONCRETE

2.1 Considerations on compressive strength gain over time

One possible approach to the assessment of structural safety consists in the analysis of long time behavior of concrete during time considering effects of strength evolution and long lasting load. In determining admissible compressive stress, $\sigma_{cd,}$ coefficients γ_c and β are used – design values based on characteristic values defined from probabilistic considerations for each limit state. γ_c coefficient represents differences between concrete from standard specimen and concrete from structural element as well uncertainties related to actions (Couto *et al*, 2015), while β is derived from the product of partial coefficients, i.e., by the multiplication of benefic effects of compressive strength evolution over time (β_1) by harmful effects of long lasting load (β_2) (Silva Filho; Helene, 2011).

Compressive strength evolution over time can be calculated by using mathematic models related to the compressive strength at 28 days. (Klemczak *et al*, 2015). It is well knowed that this evolution varies depending on the cement type, ambient temperature and curing conditions (CEB, 1990). Maintaining the ideal curing conditions and temperature at 20ºC, it is possible to estimate strength gain over time, using equations (1) and (2), proposed by fib *Model Code 2010* (CEB, 2012). This formulation is accepted by Brazilian Standard ABNT NBR 6118:2014 to ages under 28 days.

$$f_{cm}(t) = \beta_1(t) \times f_{cm} \tag{1}$$

$$\beta_1(t) = \exp\left\{ s \left[1 - \sqrt{\left(\frac{28}{t}\right)} \right] \right\} \tag{2}$$

$f_{cm}(t)$: Compressive strength at t days;
f_{cm}: Compressive strength at 28 days;
$\beta_1(t)$: Coefficient that depend on time (t);
t: Age at which is desired to obtain compressive strength;
s: Coefficient that depend on cement type: *s*=0.20: for high strength and rapid hardening cement type (case of CPV-ARI in Brazil); *s*=0.25: for ordinary and rapid hardening cement type (case of CPI and CPII in Brazil); and *s*=0.38: for slow hardening cement type (case of CPIII and CPIV in Brazil).

The loss of load capacity by long lasting loads was studied by Rüsch (1960). This decrease is constant and independent of studied concrete f_{ck}, besides that, is maximum of 25% (Silva Filho; Helene, 2011). The *fib Model Code 2010* (CEB FIP, 2012) proposes an equation (3) to determine coefficient β_2, wherein the reduction coefficient varies according to the loading age.

$$\beta_2 = \frac{f_{c,sus,j}}{f_{c,t0}} = 0.96 - 0.12 \times \sqrt[4]{\ln(72 \times (j - t_0)} \tag{3}$$

$f_{c,sus,j}$: Compressive strength of concrete under sustained load, at j days, in MPa;
$f_{c,t0}$: Potential compressive strength at time (age) t_0 just before application of long lasting load, MPa;
β_2: Harmful effects of long lasting load (t);
t_0: Load application age, in days, considered significant;
j: Any age of concrete after t_0, expressed in days.

It is estimated that the Brazilian Standard NBR 6118:2014 sets value of 1.16 to β_1 and 0.73 to β_2, considering load values at 28 days until 50 years, resulting in a β of 0.85 (Silva Filho; Helene, 2011). It is observed that this values are conservative, since it admits a strength gain of only 16% in a period of 50 years and a greater decrease than the maximum established by Rüsch (1960) (Helene, 2011). Thus, it is appropriate check the values of β_1 e β_2 considering formulation proposed by *fib Model Code 2010* (CEB, 2012) and considerations made by Rüsch (1960).

2.2 Considerations on variability of compressive strength test results

Another factor to be verified is the variability of compressive strength test results. Magalhães (2014) points out that all steps of concrete production leads to a dispersion of test results, which can be grouped in 3 different aspects: influence of materials, production methods and test procedures. Table 1 shows main factors that can affect compressive strength test results, as well the maximum variability of each factor.

Table 1. Factors that can affect compressive strength test results

Variation origin		Maximum variability
Materials	Variability of cement strength	±12%
	Variability of total amount of water	±15%
	Variability of aggregates	± 8%
Man-power	Variability of time and mixing procedure	-30%
Equipment	Lack of scale calibration	-15%
	Initial mixture, over and under charging, belts, etc.	-10%
Test procedure	Inaccurate acquisition	-10%
	Inappropriate concrete compaction	-50%
	Cure (considered at 28 days or more)	±10%
	Inappropriate concrete capping	-30% to concavity;-50% to convexity
	Rupture (loading rate)	± 5%

Source: Adapted from Helene and Terzian (1992).

It can be seen that several procedures involved in preparation, acquisition and test can directly affect test results and may reduce by up to 50% of concrete compressive strength. Indeed, this variability is true, as we can see in data from research and laboratorial tests. Santiago (2011) compiled technological control data of more than six thousand test specimens, coming from nine Brazilian states. The author identified non-conformity percentages of up to 28% for C40 concrete class. This values reaches 84% for C50 concrete class.

2.3 Considerations on concrete acceptance

Another aspect to be considered is concrete receiving and acceptance. Observing the main Brazilian national and international concrete standards, it appears diverging aspects regarding method and acceptance criteria. (Pacheco; Helene, 2013; Magalhães, 2014). The Brazilian standard, NBR 12655 (ABNT, 2015), presents two kinds of concrete sampling: total and partial. In partial concrete sampling, only some of the total batches is sampled. In total concrete sampling, all batches are sampled and the acceptance criterion is that none of the individual sample presents compressive strength below than the characteristic strength. Despite the high cost, this sampling method is widely used in Brazil (Pacheco; Helene, 2013).

The American standard, *ACI 318-11: Building Code Requirements for Structural Concrete*, establish three different criteria: the average of 3 consecutive test results must be equal or exceeds characteristic strength defined in design; no individual strength test is below than f_{ck}-3,5MPa (to concrete with f_{ck} below 35 MPa) and no individual strength test is below than 0,9* f_{ck} (to concrete with f_{ck} below 35 MPa) (Magalhães, 2014). Additionally, the standard does not provide total concrete sampling, establishing minimum criterion of only one sample per day to each 115 m^3 of concrete or each 465 m^2 of builded area. (Pacheco; Helene, 2013).

Another widely used standard, the British standard *BS EN 206:2013 - Concrete. Specification, performance, production and conformity*, presents different criterion according to the period of production: initial production or continuous production, when more than 15 results are avaible (Magalhães, 2014). The first criterion is related to the average of test results, that must be above or equal to f_{ck}+4,0MPa, to initial production, and above or equal to f_{ck}+1.48*s (standard deviation of results), to continuous production. The second criterion is related to individual test results, that, for both types of production, must be above than f_{ck}-4,0MPa (Pacheco; Helene, 2013; Magalhães, 2014).
Larrossa *et al* (2014) conducted a comparison of the above mentioned sampling criteria to 32 concrete batches. The authors pointed out that NBR 12655 (ABNT, 2015) presents most rigid criteria, followed by EN 206 and ACI 318-11. Indeed, comparing acceptance criteria adopted by international standards with the established by Brazilian standard, it can be seen that the acceptance criteria is more restrictive (Pacheco; Helene, 2013; Magalhães, 2014).

3. CASE STUDY

3.1 Methodological procedures

In conformity control realized in a building construction, by following procedures of NBR 12655 (ABNT, 2015), concrete compressive strength (f_c) of three batches of 8m^3 presented test results below than the f_{ck} of 40 MPa, specified by designer. As the building presents obstacles to extraction of concrete cores, since the structure is made of concrete-filled steel columns, a study of concrete compressive strength evolution was made, in order to helps in the safety assessment of structure. It is noteworthy that this analysis is complementary and must be performed together with other verifications, as the design review with the obtained concrete compressive strength and realization of non-destructive tests. Concrete mix proportions are presented in table Table 2.

Table 2. Concrete mix proportions

Material	Quantity	Unit
Cement CPV-ARI RS	341	kg
Pozzolan	114	kg
Fine sand	284	kg
Medium sand	426	kg
Coarse aggregate	1025	kg
Water	191	l
Polyfunctional admixture	3.41	kg
Superplasticizer	1.14	kg

Source: Concrete supplier

On specified dates, compressive tests were made at certified laboratory, following standards procedures. All specimens were grinded and tests were performed in a universal machine *EMIC – PC 200 CS.* Results are presented in Table 3.

Table 3. Compressive strength test results of non-conforming samples

Sample	Consistency (mm)	Test date	Age (days)	ø (mm)	fc (MPa)	Potential fc (MPa)
1	180	18/02/2015	7	100	23.9	23.9
		18/02/2015	7	100	23.3	
		11/03/2015	28	100	35.2	37.1
		11/03/2015	28	100	37.1	
2	230	18/02/2015	7	100	23	24.6
		18/02/2015	7	100	24.6	
		11/03/2015	28	100	37.4	37.4
		11/03/2015	28	100	34.6	
3	200	31/03/2015	7	100	23.3	23.9
		31/03/2015	7	100	23.9	
		21/04/2015	28	100	38.2	38.6
		21/04/2015	28	100	38.6	

Source: Adapted from tests reports

Regarding to conformity control used in building construction, it is important to mention that total concrete sampling was used (100%), where all of the batches are sampled. In this case, NBR 12655 (2015) states that the acceptance criterion is when all of the individual samples meet the f_{ck} specified by designer. As can be seen in Table 3, the potential strength, at 28 days, does not show strength above or equal than the 40 MPa specified.
Another pointed aspect is strength evolution after 28 days. In a study conducted by concrete supplier, in a year, the concrete presented a strength gain of 32.6% (cement CPV-ARI sulphate resistant with 22% of pozzolan addition), higher than the 16% considered by the ABNT NBR 6118:2014.

3.2 Results and analysis

From test results, it was performed an analysis of concrete compressive strength gain. Initially, β_1 values were calculated using s value of 0.20, since cement type is CPV-ARI. However, an addition of 33% of pozzolan was made (value relative to cement content), so it is appropriate calculate a β_1 value to a CPIV cement type, which contains 15% to 50% of pozzolan. Finally, a third β_1 value was calculated, for an intermediate s value. β_1 values for a 50 years, used in β calculation, are presented in Table 4.

Table 4. β_1 coefficient for different cement types

β1 values for 50 years	
CPV-ARI	1.21
CP “Intermediate”	1.33
CPIV	1.44
ABNT NBR 6118:2014	1.16

Compressive strength evolution over time for the three assumptions (CPV-ARI, CPIV e CP “intermediate”), considering lowest value of $f_{ck,est}$ (37,1 MPa) and a period of 365 days, can be verified in Figure 1.

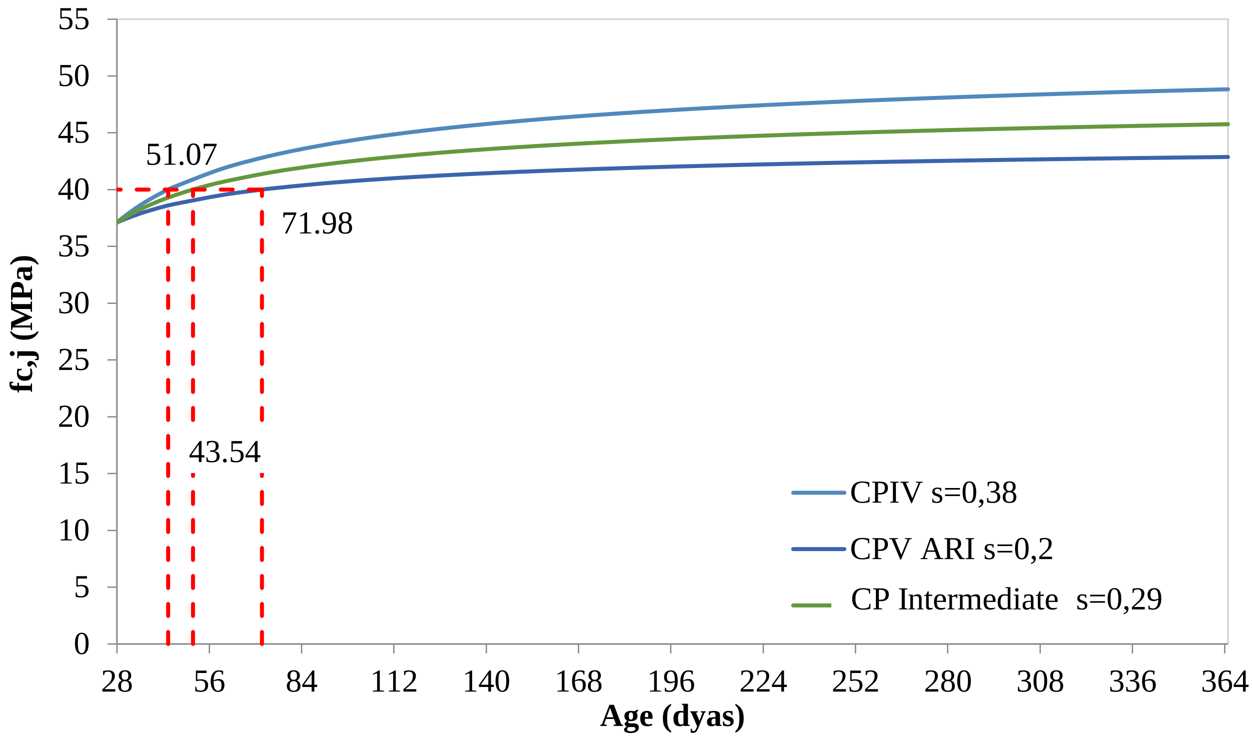

Figure 1. Strength evolution according to fib *Model Code 2010*

As can be seen in Figure 1, to achieve specified 40 MPa, it will take 45 days for β_1 to CPIV concrete, 51 days for β_1 to CP "intermediate" and 72 days for β_1 to CPV ARI. After defining β_1, it was calculated β coefficient, considering two different β_2 – 0.73, as defined by ABNT NBR 6118:2014, and 0.75, maximum value as defined by Rüsch (1960). Values are presented in Table 5.

Table 5. β coefficients for different types of cement and load

Condition	β1	β2	β
ABNT NBR 6118:2014	**1.16**	**0.73**	**0.847**
β_1 from CPV-ARI / β_2 from NBR 6118	1.21	0.73	0.885
β_1 from CP "Intermediate" / β_2 from NBR 6118	1.33	0.73	0.968
β_1 from CPIV / β_2 from NBR 6118	1.44	0.73	1.052
β_1 from CPV-ARI / β_2 maximum - Rüsch (1960)	1.21	0.75	0.909
β_1 from CP Inter. / β_2 maximum - Rüsch (1960)	1.33	0.75	0.995
β_1 from CPIV / β_2 maximum - Rüsch (1960)	1.44	0.75	1.081

For verification of safety in this study case, it was calculated the design compressive strength of concrete f_{cd}, according to Equation 4, using coefficients defined in Table 5. Results are presented in Table 6.

$$fcd = \beta * \frac{fck}{\gamma c} \quad (4)$$

Table 6. Concrete compressive stress

Condition	f_c (MPa)	γ_c	β	f_{cd} (MPa)
ABNT NBR 6118:2014 (reference value)	**40**	**1.4**	**0.847**	**24.2**
β_1 from CPV-ARI / β_2 from NBR 6118	37.1	1.4	0.885	*23.4*
β_1 from CP "Intermediate" / β_2 from NBR 6118			0.968	25.7
β_1 from CPIV / β_2 from NBR 6118			1.052	27.9
β_1 from CPV-ARI / β_2 maximum - Rüsch (1960)			0.909	*24.1*
β_1 from CP Inter. / β_2 maximum - Rüsch (1960)			0.995	26.4
β_1 from CPIV / β_2 maximum - Rüsch (1960)			1.081	28.6
β_1 from CPV-ARI / β_2 from NBR 6118	37.4	1.4	0.885	*23.6*
β_1 from CP "Intermediate" / β_2 from NBR 6118			0.968	25.9
β_1 from CPIV / β_2 from NBR 6118			1.052	28.1
β_1 from CPV-ARI / β_2 maximum - Rüsch (1960)			0.909	24.3
β_1 from CP Inter. / β_2 maximum - Rüsch (1960)			0.995	26.6
β_1 from CPIV / β_2 maximum - Rüsch (1960)			1.081	28.9
β_1 from CPV-ARI / β_2 from NBR 6118	38.6	1.4	0.885	24.4
β_1 from CP "Intermediate" / β_2 from NBR 6118			0.968	26.7
β_1 from CPIV / β_2 from NBR 6118			1.052	29.0
β_1 from CPV-ARI / β_2 maximum - Rüsch (1960)			0.909	25.1
β_1 from CP Inter. / β_2 maximum - Rüsch (1960)			0.995	27.4
β_1 from CPIV / β_2 maximum - Rüsch (1960)			1.081	29.8

The values obtained showed that only when considering exclusively cement CPV ARI the final stress obtained is below than the expected. For concrete with cement with pozzolan addition, as is the case of this study, design compressive concrete strength is above that required. It can be seen a conservative nature of ABNT NBR 6118:2014 – expected characteristic of a technical standard. However, there is a possibility of using consolidated knowledge and move forward in the study of compressive concrete strength gain after 28 days.

It is noteworthy that, according to ABNT NBR 6118:2014, if the compressive strength kept lower than design f_{ck}, a new structural design with the obtained value should be realized. Still remaining the unsafety, the use of structure should be limited, a reinforcement should be designed or even the total or partial demolition of non-conformity elements should be done.

4. CONSIDERATIONS

By analyzing long time effects on concrete compressive strength, as well the recommendations from international standards and others factors involved in technological control of concrete, it can be seen that the requirements established in ABNT NBR 6118:2014 are conservative, leading to a higher safety degree, as expected in technical standards.

However, some criteria established by this standard does not take into account important factors, particularly regarding to concrete strength gain over time, as noted in case study presented. The standard does not take into account actual behavior of the material, since it ignores pozzolan addition effects in this strength gain, besides considering a decrease in strength (*Rüsch effect*) higher than the maximum defined by Rüsch (1960) (Helene, 2011; Silva Filho, Helene, 2011). This factors can affect, directly, β coefficient, that influences structural design.

It is noteworthy that before using new coefficients, another stages of safety assessment must be executed, such as the design review and a rigorous inspection, checking accuracy of execution, geometry and material quality. This stages, in addition to the estimation of performance of concrete over time, can help in safety assessment and in decision-making in cases of non-conformity in structural concrete.

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Reinforcement corrosion rate and crack width relationship in concrete beams exposed to simulated marine environment

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ABSTRACT
This investigation presents an empirical correlation between the rebar corrosion rate and the corrosion-induced crack width propagation rate produced on beam's concrete cover, with or without load application to these beams. Reinforced concrete beams were evaluated, exposed to a natural corrosion process by spraying with 3.5 %w/w NaCl solution, to accelerate the rebar corrosion process, was performed with electrochemical tests. The beams corrosion-cracking evaluation was performed once every month, to determine the relation between crack width and the rebar corrosion loss. The results showed a direct relation between crack width propagation and rebar corrosion rate, showing wider cracks in the loaded beams.
Key words: corrosion; reinforced concrete; loaded beams; crack widths.

RESUMEN
Esta investigación presenta una relación empírica entre la velocidad de corrosión de la armadura y la velocidad de ensanchamiento de fisuras por corrosión del recubrimiento de concreto en vigas, con o sin aplicación de carga. Se evaluaron vigas de concreto armado, expuestas a un proceso de corrosión natural mediante el rociado con solución salina al 3,5 %p/p de NaCl, para acelerar el proceso corrosivo de la armadura, mediante ensayos electroquímicos. El ancho de fisuras se evaluó mensualmente para estimar la relación existente entre éste y la pérdida de sección de la armadura. Los resultados demuestran que existe una relación directa entre la propagación del ancho de fisuras y la velocidad de corrosión, observando fisuras de mayor ancho en vigas cargadas.
Palabras clave: corrosión; concreto armado; vigas cargadas; ancho de fisuras.

RESUMO
Esta pesquisa apresenta uma relação empírica entre a taxa de corrosão da armadura e a abertura de fissuras por efeito da corrosão da armadura em vigas de concreto, com ou sem aplicação de carga. Foram avaliadas vigas de concreto armado, expostas a um processo de corrosão natural por pulverização com solução salina a concentração de 3,5% de NaCl, para acelerar o processo de corrosão da armadura, mediante ensaios eletroquímicos. A abertura das fissuras foi avaliada mensalmente para estimar a relação entre ela e a perda de seção da armadura. Os resultados mostram que existe uma relação direta entre a propagação da abertura da fissura e a taxa de corrosão, observando a ocorrência de fissuras de maior abertura nas vigas sob carga.
Palavras-chave: corrosão; vigas de concreto armado sob carga; abertura de fissuras.

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1. INTRODUCTION

Corrosion damage of reinforced concrete structures, particularly in marine environments, is a serious problem worldwide. It incurs costly repairs and can threaten user safety in severe cases. This problem is particularly acute in reinforced concrete beams, because they experience tensile forces from flexure loading, making it almost impossible for them not to develop micro cracks. These constitute an entrance path for corrosive substances to arrive at the carbon-steel reinforcement bar and activate it. In well-designed beams, thin transverse cracking due to flexure loading is always present. These are normally invisible to the naked eye, and do not affect durability in most cases. As mechanical loads (applied directly to beams) increase, both crack number and width increase, commonly reaching widths of approximately 0.25 mm. If loads continue to increase, crack width will increase, although crack number changes little (ACI 224, 1992).

In the steel reinforcement model proposed by Tuutti (Tuutti, 1992), T_1 (initial corrosion) is defined as the period between structure fabrication and the start of corrosion, and T_2 (propagation) as the time between the start of corrosion and manifestation of external damage, when a structure attains an unacceptable level of deterioration in terms of safety, functionality and/or aesthetics. In addition to Tuutti's two stages (T_1 and T_2), a third stage (defined here as residual life), has been described that begins from degradation symptom appearance (e.g. cracks > 0.1 mm in width; concrete spalling; visible loss of steel section, etc.) and ends with structural collapse. During this final stage, load capacity progressively degrades in the corroding structure (Troconis de Rincón y col., 1997). In other words, the residual life can be considered as the time during which a structure must be repaired before collapsing.

The easiest stage to detect during visual inspection of a concrete structure is the residual life stage since wide cracks, surface rust stains, and concrete spalling are clearly visible to the eye. However, to evaluate if a structure is in stage T_1 or T_2 (service life) requires more complex, costly diagnoses. In addition to visual inspection, electrochemical monitoring must be performed to determine steel activation, corrosion rate estimation to indirectly document steel mass loss due to corrosion, extract test cylinders from the concrete element to measure chloride concentration at reinforcement depth, etc. Once corrosion rate and steel cross section loss are quantified, a prediction of structural damage can be generated.

Corrosion rate measurements indicate the amount of metal that has been transformed to oxide per reinforcement surface area and time. The amount of oxides generated is directly related to concrete cover cracking and adherence loss. Reduction in steel transverse cross section affects structure load capacity, and corrosion rate is therefore an indicator of decline in structure load capacity. Load capacity reductions in reinforced concrete elements affected by reinforcement corrosion are fundamentally due to four effects: 1) rebar cross section reduction; 2) rebar ductility reduction; 3) rebar/concrete adherence reduction; and 4) loss of effective concrete section due to concrete cracking. (Torres y Martínez, 2001)

Previous studies (Andrade y col., 1993; Cabrera, 1996; Rodríguez y col., 1996; Rodríguez y col., 1997; Tachibana y col., 1990; Torres y Sagües, 2000; Torres, 1999; Torres, Castro, Sagües, 1999; Torres et al, 2007) have evaluated corroded beams using accelerated corrosion tests and crack mapping survey to calculate the remaining load capacity (RLC) value for structural elements with generalized corrosion, based on easily measured damage such as corrosion-induced average surface crack width. The data in these studies (Torres y Martínez, 2003; Torres et al, 2007) indicate that during the residual life stage is when a structure's RLC begins to decline rapidly: corrosion radius loss is directly proportional to the crack width in the concrete surface. Therefore, this investigation will determine empiric correlations, which may allow the evaluation of how reinforcement corrosion may affect concrete surface crack initiation and propagation, in length as well as in width, to predict the structure's durability, and prevent higher deterioration impact that may increase repair costs and eventually its failure, for the particular case of loaded beams.

2. EXPERIMENTAL PROCEDURE

Twenty-four beams (1200 x 100 x 150 mm) were made with a 9.5 mm diameter steel bar with a 25 mm concrete cover. Concrete's capillary sorption was determined using the Sweden Standard also known as Fagerlund Method (Fagerlund, 1986). The procedure include the fabrication of cylindrical specimens (10x20 cm), which after a curing period of 30 days, they were dryed at 50ºC up to constant weight; afterwards they were placed in wet sponges in a wáter container and weighed for several days until reaching constant weight.
The middle section (250 mm long) of all twenty-four beams (Cl^- contaminated and non Cl^- contaminated), was sprayed with a 3.5% w/w NaCl solution twice weekly (on the face closest to the steel rebar) to further accelerate the steel rebar activation process via Cl^- ion diffusion from the exterior.
Half of each set of concrete beams (Cl^- contaminated and non Cl^- contaminated) was placed under constant flexure load for one year. This simulation of normal loads in real beams was done to evaluate the effect of flexure on corrosion rate and overall beam durability. Beams with this applied load are defined in this investigation as preloaded beams.

2.1 Concrete beams evaluation

Rebar half-cell corrosion potentials were measured twice weekly using a saturated $Cu/CuSO_4$ electrode at three points along the steel rebar, similar to a potential mapping procedure (ASTM C876, 2009). Corrosion rates were measured at three points along the steel rebar using the linear polarization method and a GECORR-6 field test device (rebar central zone and end zones).
To simulate the influence of flexure loading on corrosion of steel rebar in concrete structures, a constant center flexure load of 500 kg was applied to twelve of the beams in six preload systems, containing two beams each. Load was applied at three points (center and each end), as observed in Figure 1, making six preload systems with two beams each. Once every month, visual inspection was performed to the twenty-four beams (with or without flexure preload), to determine the appearance of surface cracks after. If surface cracks appeared at the beam's surface, a detailed crack survey was performed, including its location, length and width.

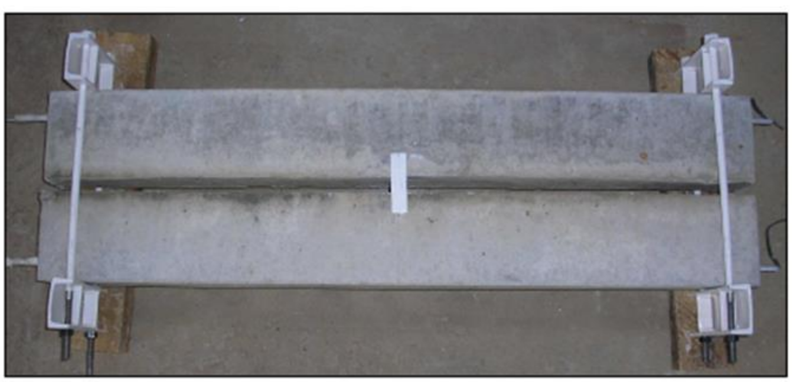

Figure 1. Load system applied to reinforced concrete beams

2.2. Results Analysis for correlation determination

The corrosion rate data were used to calculate rebar section loss with the Faraday Law conversion:

$$\Delta W_F = \frac{55.85\int I.dt}{n.F} \tag{1}$$

Where:
ΔW_F = mass loss (g)
n = iron valence (Fe^{+2})
F = Faraday constant= 96,500 coul/mol
Fe Atomic weight = 55.85 g/mol
$\int I.dt$ = ∫icorr (A)dt = Area below the icorr vs. time curve

This mass loss value was then used to estimate average radius loss from corrosion ($x_{AVERAGE}$) (Geocisa, 2000). These $x_{AVERAGE}$ values were correlated to concrete crack width over time.

$$x_{AVERAGE} = \frac{\Delta W_F .1000}{\rho.\pi.\phi.L} \tag{2}$$

Where:
ρ = Fe density (7.86 g/cm^3)
ϕ = Reinforcement bar diameter (9.5 mm)
L = Reinforcement bar length (1,000 mm)

3. RESULTS

The mixture design was defined using ACI 211.1 method (ACI 211.1, 1993), using a 0.60 water to cement (w/c) ratio concrete. The beams were made with a concrete rated as moderate performance in marine environments: capillary sorption = 1.50x10^{-4} m/s$^{0.5}$; effective porosity = 8.3-8.8%; average 28-day compressive strength = 330 kg/cm^2.

3.1. Electrochemical evaluation

Electrochemical measurements were taken during a three-year period (approx. 1,100 days), long enough to allow the steel rebar to activate (< -200 mV vs. $Cu/CuSO_4$), as observed in Figure 2. Preloading had no significant effect on half-cell potential values in the experimental period because the rebar was already active when the flexure load was applied.

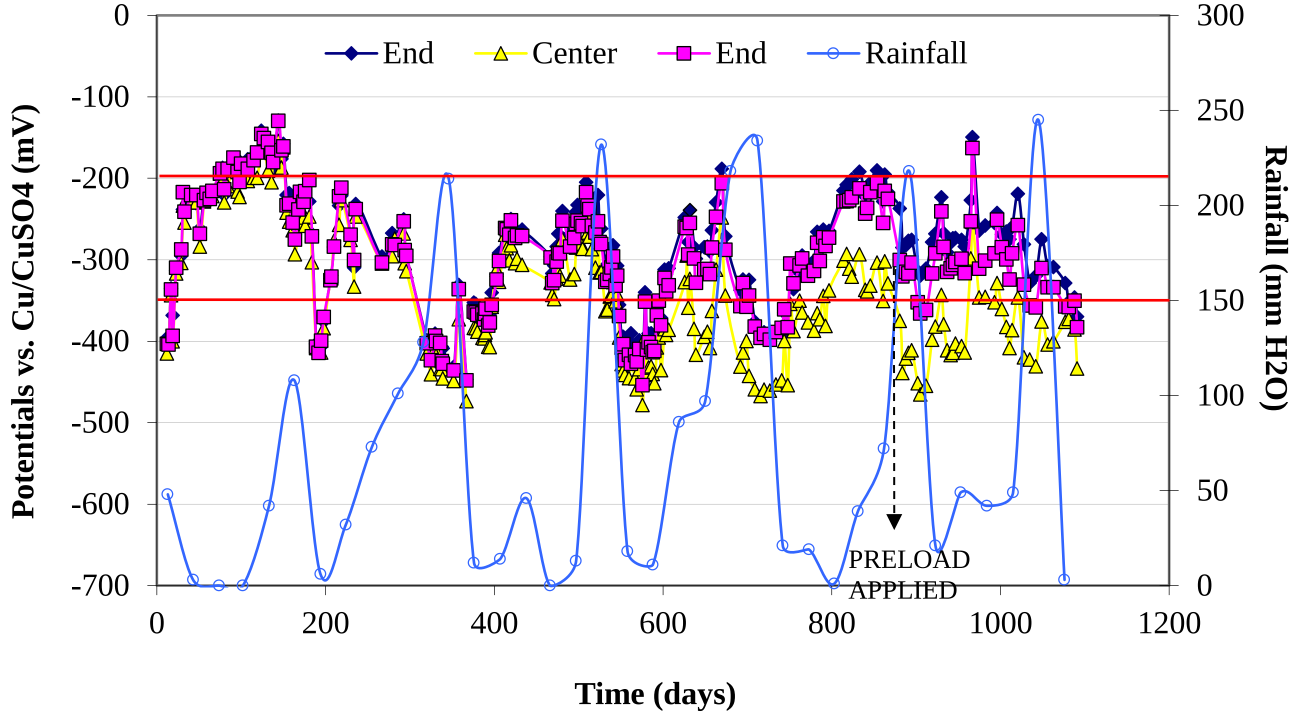

Figure 2. Potentials variation during experimental period of preloaded, Cl^- contaminated beams

Spraying with NaCl solution, rather than the preloading, caused the half-cell potentials in the beams' central portion to be more negative than the ends. Rainfall also affected steel rebar activity, which is to be expected since it lowers concrete resistivity. As occurred in the Cl^--contaminated beams, the non Cl^--contaminated beams had half-cell potentials vs. $Cu/CuSO_4$ electrode values indicating no significant effect from the applied preload (Figura 3).

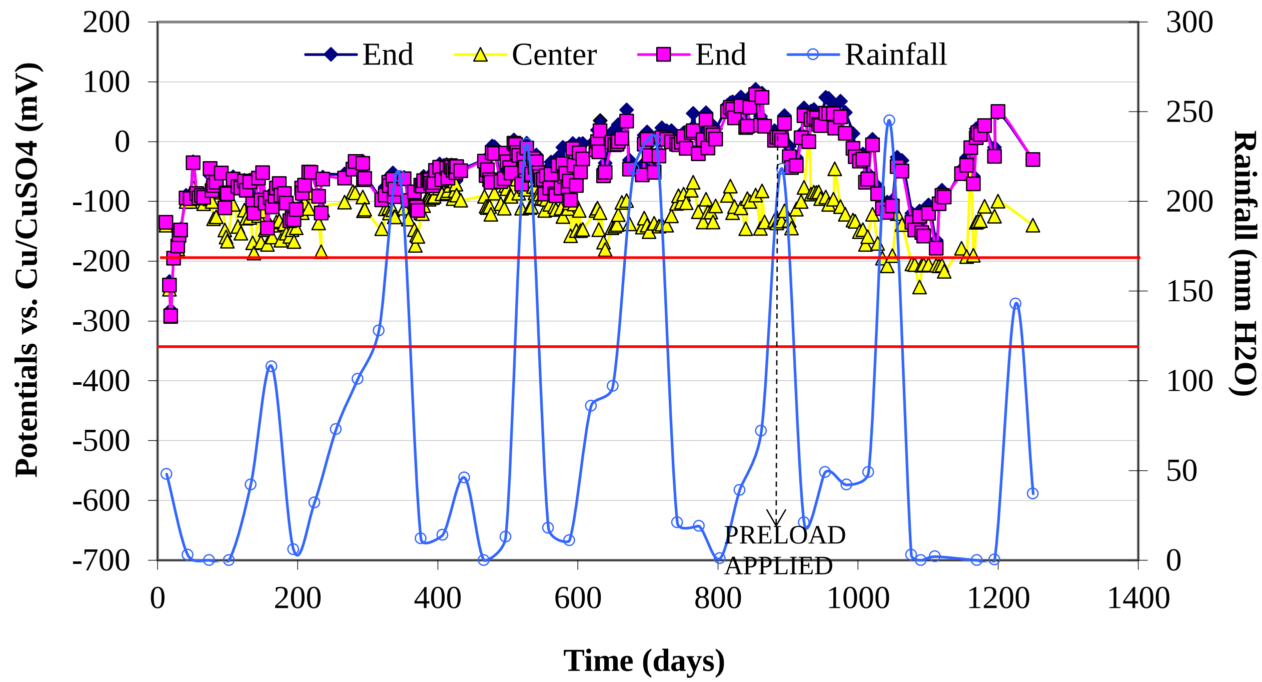

Figure 3. Potentials variation during experimental period of preloaded, non Cl^- contaminated beams

Figure 4 shows that rebar corrosion rate was not significantly affected by preloading, confirming half-cell potentials measurements (Figure 2). It was higher in the central portion of the bars' due to spraying with NaCl before and after preloading. The variation observed in corrosion rate was due to the effect of rainfall, which diminishes concrete resistivity.

The preloaded, non Cl^--contaminated beams that had been sprayed with NaCl in their central portion (Figure 5), initially exhibited some activity at all tested points. They then attained passivity and apparently remained so until the load was applied, when the half-cell potentials measurements indicated activity in the steel rebar, corroborating half-cell potential results obtained.

However, the same effect was observed in the non-preloaded beams. This suggests that preloading was not the cause of the increased corrosion activity, but rather the combined effect of the Cl^- ions and high rainfall during the experimental period. This is clearly visible during dry seasons, when corrosion rates decreased but values still indicated activity in the reinforcement (0.1 µA/cm^2), especially in the central zone.

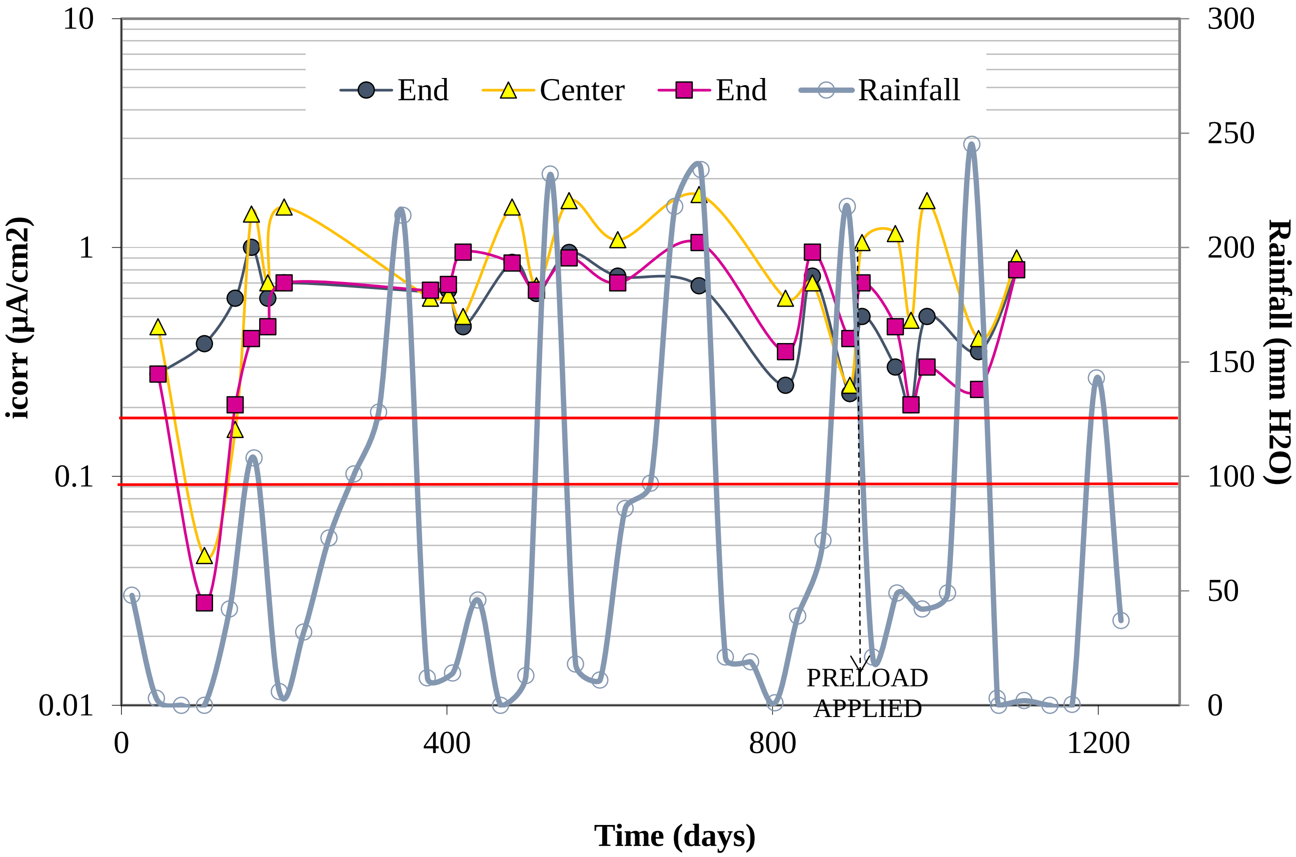

Figure 4. Corrosion rate during experimental period of preloaded, Cl^- contaminated beams

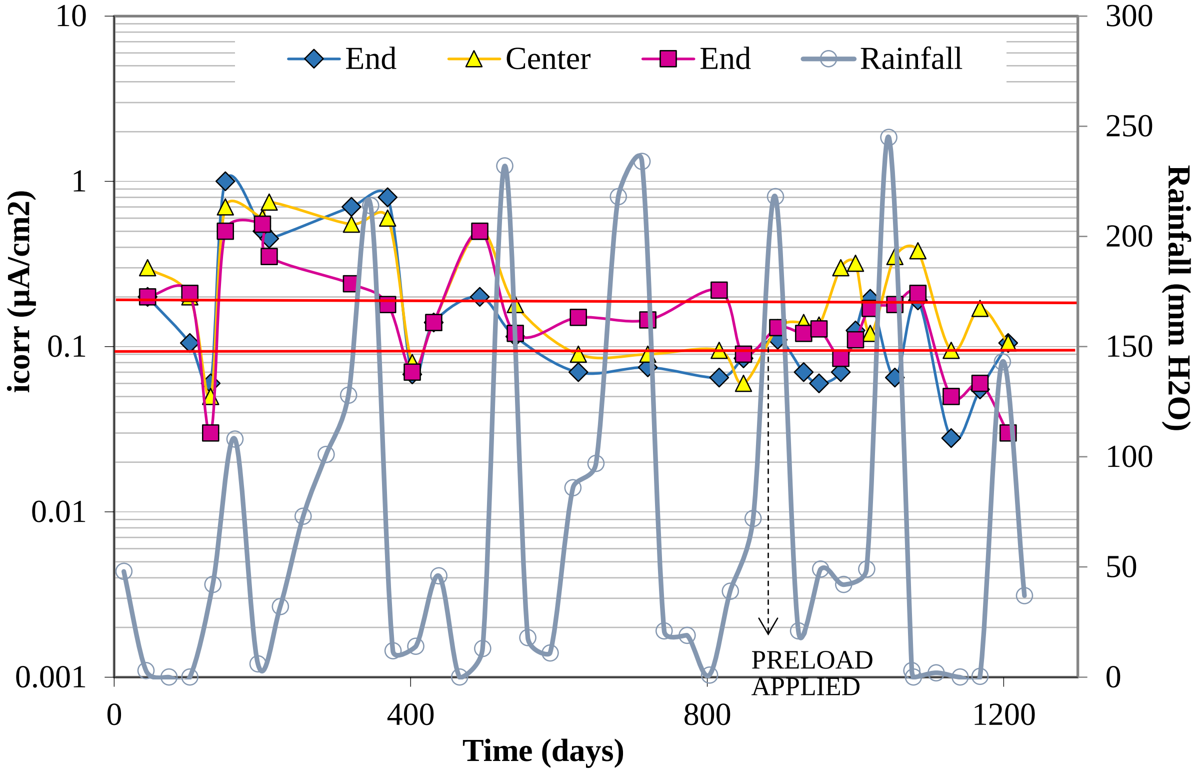

Figure 5. Corrosion rate during experimental period of preloaded, non Cl^- contaminated beams

3.2 Visual inspection and crack survey

When corrosion cracks appeared in the beams, monthly visual inspections and crack surveys were begun. Once the experimental period had ended, the beams were mechanically tested, until rupture.

3.2.1. Non Cl^- contaminated beam systems

Neither parallel no perpendicular cracking were apparent in the non-preloaded beams (Figure 6a), even with spraying of NaCl in the central 250 mm section. Corrosion rate values indicated activity (> 0.1 µA/cm^2), but it was insufficient to generate crack appearance on the concrete cover surface. As shown in the electrochemical parameter values, the preloaded beams (Figure 6b) exhibited only perpendicular cracks (< 0.15 mm) due to preloading. These did not enhance reinforcement corrosion during the experimental period.

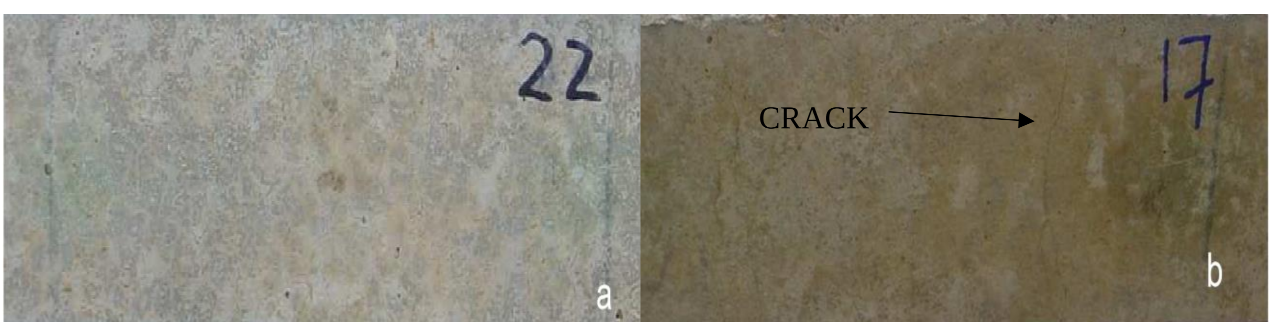

Figure 6. Beams without initial chloride contamination, rainsed with chlorides in the 25-cm central zone (a) non-preloaded (b) preloaded

3.2.2. Cl⁻ Contaminated beam systems

In non-preloaded beams (Figure 7a), it was observed in the central zone, where chloride rainsed was performed, cracks parallel to the rebar appeared, having 0.40 mm maximum width. In preloaded beams (Figure 7b), corrosión cracks reached 0.8 mm widths. This indicate that the initial chloride contamination helped corrosión crack initiation.

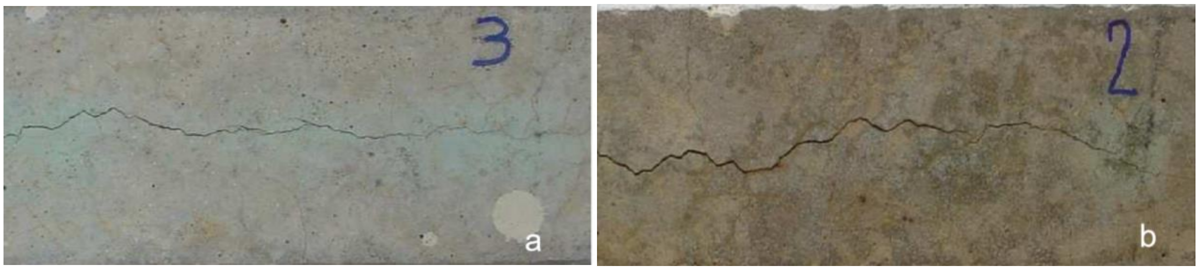

Figure 7. Beams with initial chloride contamination, rainsed with chlorides in the 25-cm central zone (a) non-preloaded (b) preloaded.

3.3. Crack opening rate vs. i_{CORR} correlation

Figure 8 shows average conditions of the evaluation performed. As observed in this figure, in central zone, average corrosion rate are similar for preloaded as well as non-preloaded beams (1.080 – 0.989 µA/cm^2, respectively). Same corrosión rate behavior was observed at the end zones of same beams (0.584 – 0.519 µA/cm^2, respectively) (Figures 4 and 5). Differences were present, however, between the central portion and ends of each beam treatment, due to spraying with NaCl solution in the former.

Beam Ttreatment	Average corrosion rate (µA/cm^2)	Crack opening rate (mm/yr)
Center Preload	1.080	0.752
Center Non-Preload	0.989	0.619
Extreme Preload	0.584	0.267
Extreme Non-Preload	0.519	0.225

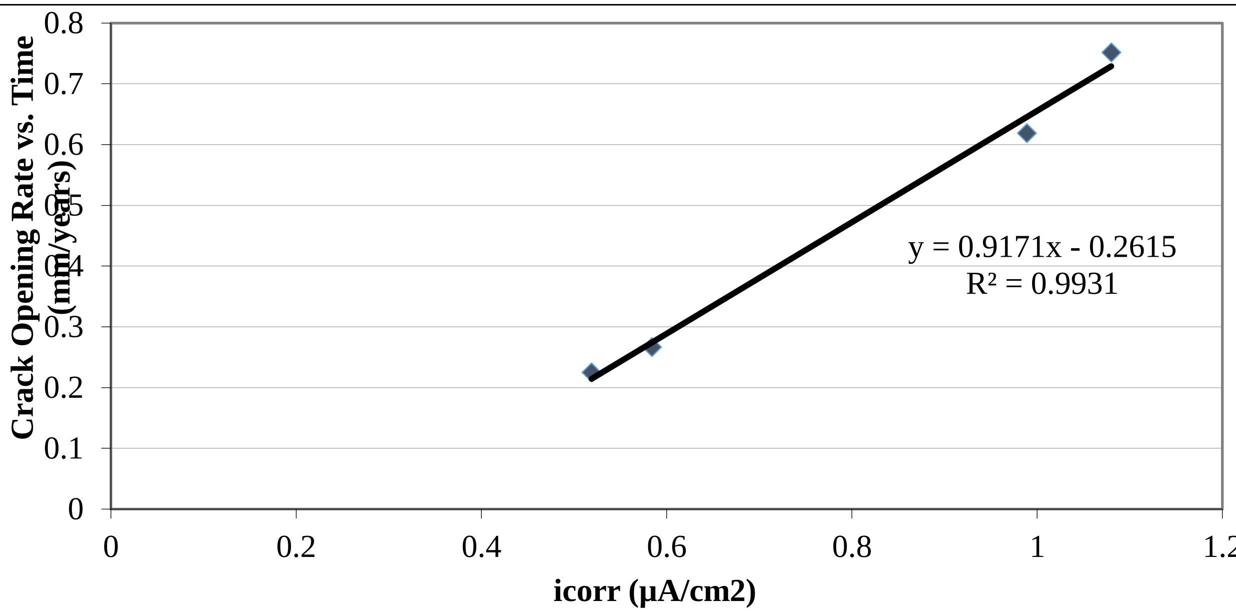

Figure 8. Average slope for crack width opening rate and reinforcement corrosión rate (Hernández, 2009)

A good correlation was observed between crack propagation rate and steel rebar corrosion rate (Figure 8). This phenomenon has been reported previously [Alonso y col., 1998] in a study using an accelerated corrosion method involving an anodic current applied to the rebar inside chloride contaminated concrete (10 and 100 μA/cm^2), and crack propagation rate was higher at the lower current intensity (low corrosion rate). In contrast to this previous study [Alonso y col., 1998], the present results indicate that with natural corrosion at low i_{corr} values (< 1.2 μA/cm^2) crack width propagation was directly proportional to the rebar apparent corrosion rate.

Figure 9 shows the correlation between máximum crack width and average rebar corrosión penetration of all tested beams, preloaded and non-preloaded, (natural corrosión, performed at CEC, Corrosion Studies Center, Venezuela), together with accelerated corrosion data for comparison purposes (Andrade y col., 1993; Cabrera, 1996; Rodríguez y col., 1996; Rodríguez y col., 1997; Tachibana y col., 1990; Torres y Sagües, 2000; Torres, 1999; Torres, Castro, Sagües, 1999; Torres et al, 2007). CEC data, preloaded and non-preloaded, shows for a determined crack width, less corrosión is needed when beams are preloaded as if the beam in non-preloaded (preloaded beams data trendline is shifted to the left of the non-preloaded beams data trendline). Results demonstrate the existence of a direct relationship between crack width and rebar corrosión rate, and observing wider crack son the preloaded beams. Figure 9 shows an interesting tendency where natural corrosion data presented a smaller slope in its tendency lines than the natural corrosión tendency line slope.

Under accelerated corrosion, cracks were produced with less steel loss, apparently because corrosion products rapidly accumulated near the reinforcement, generating high tensile stresses that caused cracking. Additionally, the anodic current may also have generated acidity at the steel/concrete interface, undoubtedly lowering adherence between the two materials (steel and concrete) and facilitating crack generation. In contrast, under the natural conditions studied here, corrosion was considerably slower, allowing corrosion products to migrate and thus requiring the accumulation of more products before cracking began.

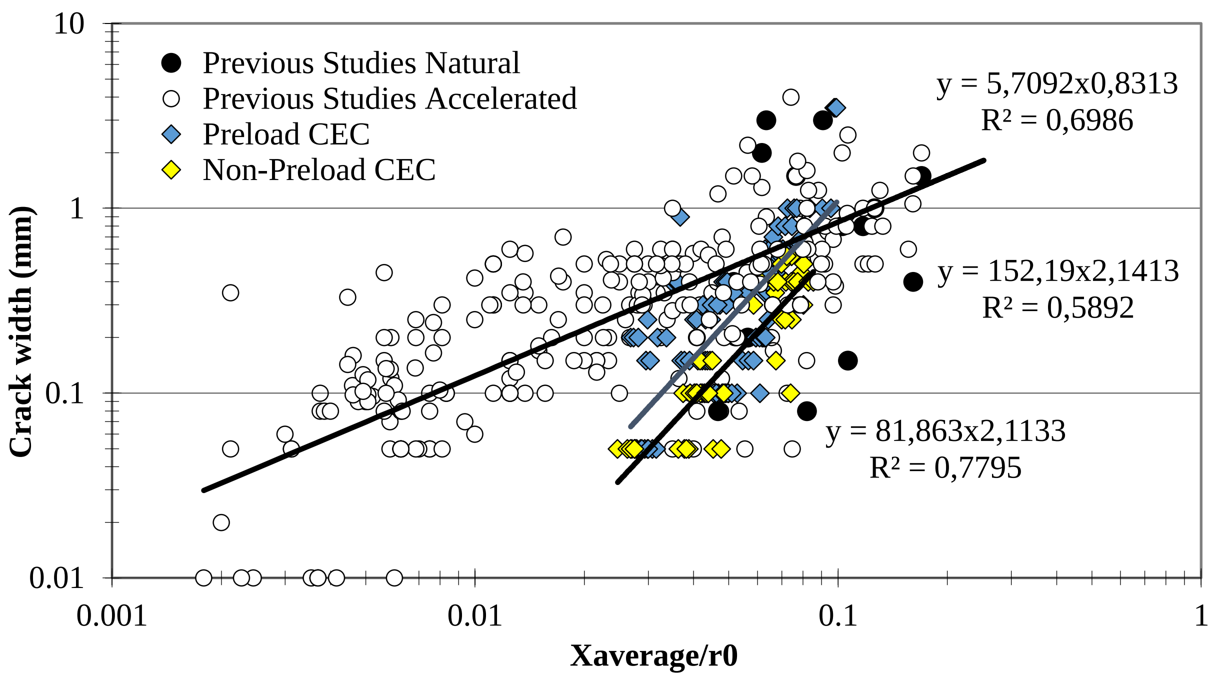

Figure 9. Maximum crack width vs. cross section loss ($X_{average}/r_0$) data from the CEC and previous studies

This may indicate that accelerated corrosion to simulate degradation in real structures therefore does not always produce results comparable to those of natural corrosion.
Analysis of chloride ion concentration at the reinforcement bar (Figure 10), showed that cracked beams had higher chlorides concentrations near the rebar, because the cracks allowed the NaCl solution, sprayed on the beams, to more easily penetrate the concrete and reach the rebar, increasing the concentrations up to higher values than the chloride threshold limit (4,000 ppm Cl^-), with respect to the non cracked beams.

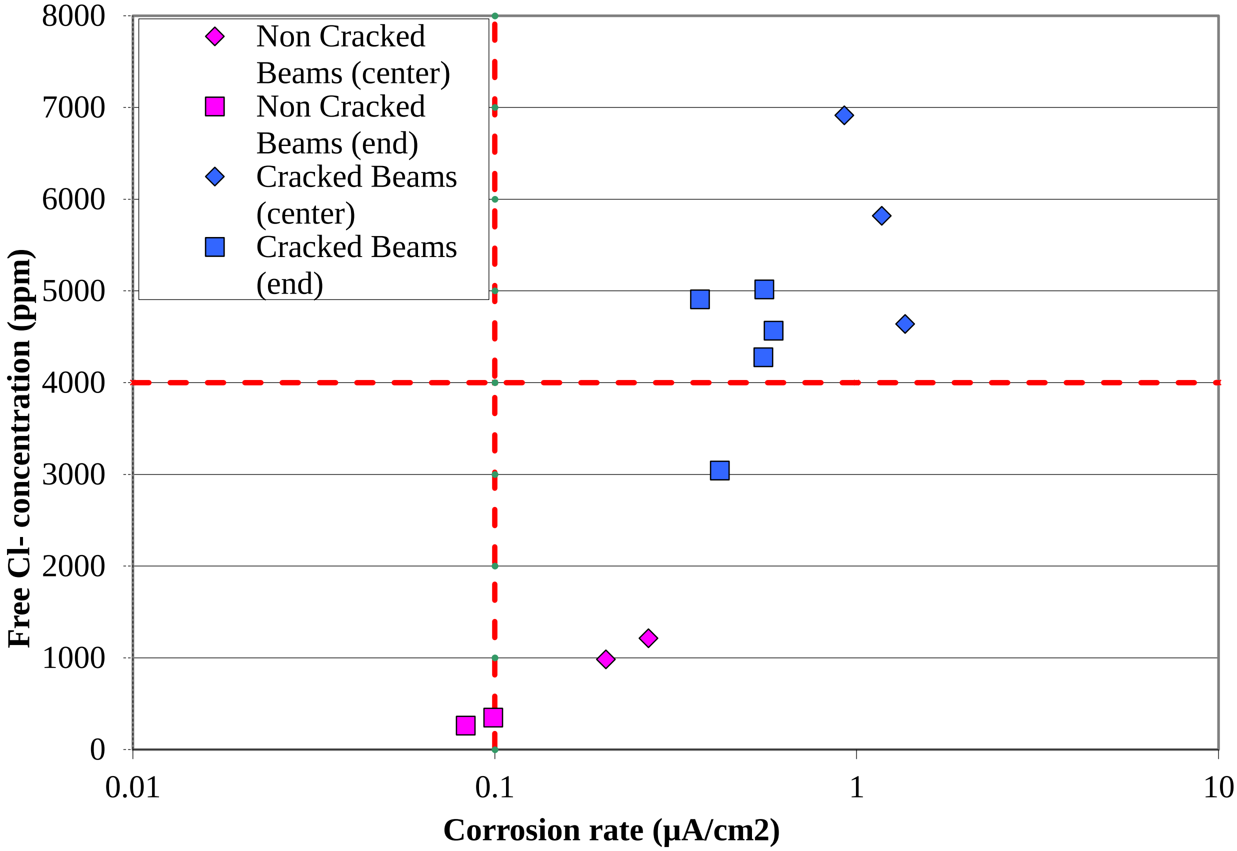

Figure 10. Chloride ion concentration at the rebar depth vs. corrosión rate, cracked and non cracked beams (Hernández, 2009)

This corroborates the results of a study evaluating concrete test cylinders from structurally cracked bridges in which chlorides were found to have penetrated the concrete more easily in cracked zones than in those without cracks (Sagües y col., 2001). The results indicate that free chlorides content was higher and corrosion rate greater in the center of the beams, corroborating how the preload increased chloride penetration (due to higher crack propagation rate) and the corrosión rate increased base don this increase of free chloride concentration.

4. CONCLUSIONS

1. Electrochemical analyses showed that crack widths < 0.15 mm do not enhance reinforcement corrosion during the evaluated time period.
2. Preloading accelerates the reinforcement corrosion process by allowing greater entry of aggressive compounds.

3. Accelerated corrosion of an element causes crack appearance with less steel loss. Natural corrosion, in contrast, requires more corrosion product accumulation before cracking begins.
4. A direct relationship exists between crack width and corrosion-induced rebar cross section diameter loss, with wider cracks in flexure loaded beams.
5. In a natural corrosión process, to obtain a similar crack with value than in an accelerated corrosión process, less corrosión cross section loss in needed when specimens are non-preloaded. There is also an increase in corrosion crack propagation when natural corrosion beams are prelodad than when are non-preloaded.

5. AKNOWLEDGEMENTS

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Evaluation of an active immobilizing opportunity: A case study for a micro-enterprise information technology branch applied to construction

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ABSTRACT

The purpose of this article is to analyze the immobilizing opportunity to corporate real estate, considering the expectations of users of real estate corporations. In order to achieve our purpose, we have chosen as our method the case study of a situation where a micro branch of information technology has been applied to construction. Thus, the purchase opportunities and its alternatives were evaluated: a short-term lease, long-term lease and built-to-suit. The method of decision-making was the hierarchical analysis, forit contemplates both qualitative and quantitative decision criteria, which are relevant to the choice between the alternatives. The result of the analysis is the recommendation of possible choices between the shown alternatives, starting from the one that best meets the criteria prioritized by stakeholders.
Keywords: immobilization, demobilization, real estate, analytic hierarchy process.

RESUMO

O objetivo deste artigo é analisar a oportunidade de imobilização de imóveis corporativos, considerando as expectativas das corporações usuárias dos imóveis. Para isto, optou-se pela realização de estudo de caso com uma microempresa do ramo de tecnologia de informação aplicada à construção civil. Assim, foram avaliadas as oportunidades e as alternativas: compra do imóvel, a locação de curto prazo, a locação de longo prazo e o built-to-suit. O método de tomada de decisão utilizado foi o da análise hierárquica, pois esta contempla critérios de decisão qualitativos e quantitativos relevantes à escolha entre as diferentes alternativas. O resultado final da análise é a recomendação da alternativa de escolha, que melhor atende aos critérios priorizados pelas partes interessadas.
Palavras-chave: imobilização; desmobilização; imóveis corporativos; análise hierárquica.

RESUMEN

El propósito de este artículo es analizar la posibilidad de inmovilización de inmuebles corporativos, teniendo en cuenta las expectativas de los usuarios corporativos de los inmuebles. Para ello, optamos por realizar el estudio de caso con una micro empresa en el sector de tecnologías de la información la construcción civil. Por lo tanto, fueron evalúdas las oportunidades y las alternativas: compra de bienes inmuebles, alquiler de corto plazo, alquiler de largo plazo y la construcción a la medida. El método de toma de decisiones utilizado fue de analisis jerárquico, ya que esta incluye criterios de decisión cualitativa y cuantitativa relevantes para la elección entre las distintas alternativas. El resultado final del análisis es la recomendación de la alternativa de elección, que mejor atienda principales por las partes interesadas.
Palabras clave: inmovilización; la desmovilización; inmuebles corporativos; análisis jerárquico.

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1. INTRODUCTION

According to Gregory (2010), the expansion of corporate business in the first decades of the twentieth century (post-Industrial Revolution), has promoted the need for corporate spaces. In the 1960s, many organizations focused their real estate[1] activities on the construction of new buildings for their own use. As the focus of corporations was to promote their growth, the acquisition and construction of new buildings had become part of their main activities, and so, receiving a greater quantity of resources. Thus, the demand for rented spaces has had a great growth, boostingthe professionalism of the real estate markets.

Prior to any decision regarding the sale, purchase or asset demobilization property of a corporation, corporations need to know the importance of the real estate for the transaction, since it isan asset with important impacts for their financial aspects, market and organization (O'Mara, 2000). As Pottinger et al. (2002) said, the level of flexibility and kinds of spaces for the same corporation are not uniform. Thus, the strategic and operational needs of each space used in the operation, interfere on the decision by immobilization, or not, of the real estate enterprise.

In research conducted by Jones Lang La Salle[2], it were contacted relevant information about the trend toward investment in corporate real estate. In the year thissurvey was published (2005), only 15% of respondents were corporations that owned more than 50% of the space used in their operations and 43% did not own any real estate assets. It wass also observed that companies that had between 10% to 50% of corporate spaces indicated interest in reducing their real estate assets, as shown in Figure 1.

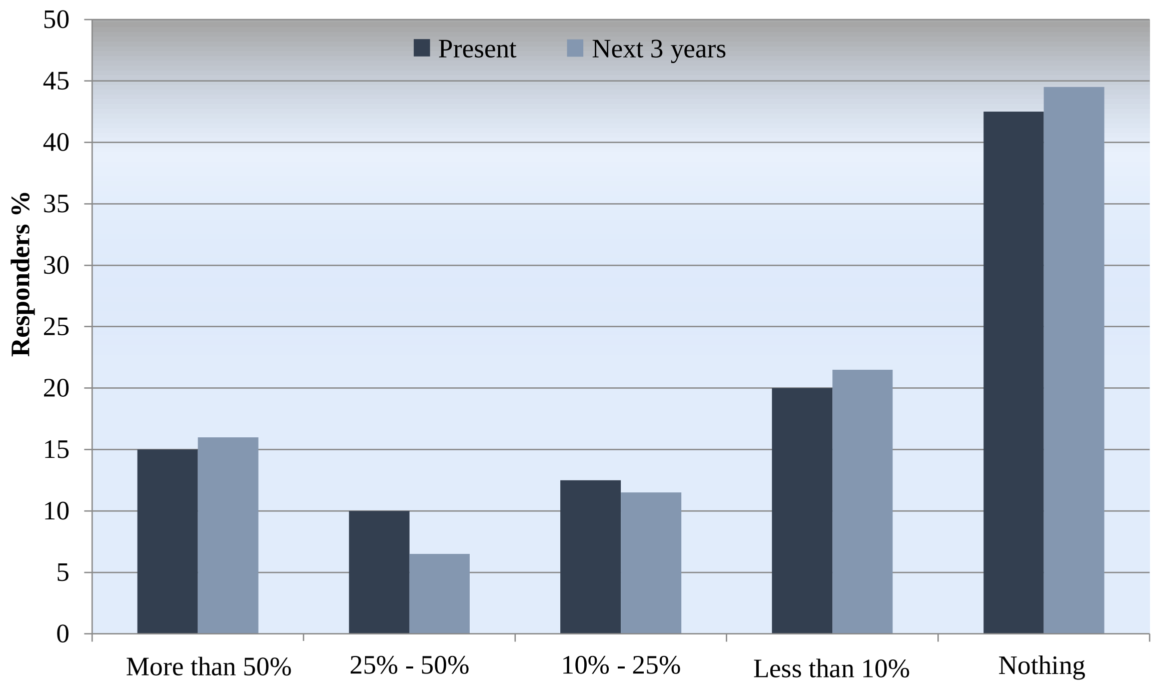

Figure 1. Percentages of corporations in relation to ownership percentages of corporate spaces. Source: Jones Lang La Salle, 2005.

[1] According to the Real Estate Center (NRE) of the Polytechnic School of USP, Real estate is the sector of goods and real estate.

[2] The Jones Lang La Salle is a company that offers professional real estate services and real estate investment management for investors, homeowners and tenants. Currently has a gross revenue estimated at US $ 6.0 billion and with operations in 80 countries.

Another survey, developed in 2013 by the same author, indicates that 66% of executives in charge of the surveyed management companies in Brazil were focused on expanding their portfolios in the next three years. At the same time, 15% of international companies were outsourcing all the activities related to their real estate portfolios, while only 6% of brazilian companies were doing the same.
Currently, Brazilian companies are reorganizing their capital structure, changing from the owners' position to tenants of physical spaces, in order to prevent corporations of immobilizing their capital in real estate assets, and so promoting the investment only in their business activity (GREGÓRIO, 2010).
Brazilian corporations tend to demobilize real estate assets and a to have a higher resistance to the immobilization of assets in the acquisition of new properties. This increases the importance of developing a research on this topic. It is worth mentioning that Brazil has some peculiarities concerning the implied forms of contract transaction, as well as necessary guarantees, posible risks, and other factors. These particularities have impacts on qualitative and quantitative aspects of the alternatives between mobilize or not in the real estate corporate.
The purpose of this paper is to analyse the possibilites of immobilisation and demobilization of the corporate real estate, considering the expectations of corporations that make user of it. For this, we have chosen to study the case of a micro-enterprise whose focus was on applying information technology to construction, and which was incubated at the Federal University of Paraná. The reason for choosing this enterprise was that it had great potential for expansion of it's business. Their active immobilizing opportunities were then assessed, and amongst these, the following ones have been considered: buying the property (SP) or not, as well as the options of not immobilizing it: the simple lease short-term (LCP), the simple lease long term (LLP) and the built-to- suit (BTS).

2. METHODOLOGY

The decision-making method used in this paper is the hierarchical analysis, which includes relevant qualitative and quantitative decision criteria for the choice between options that operate in very different ways in order to achieve the necessary attributes and criteria to be considered the most suitable one. And it is what ensures the alignment of real estate resource to the operational needs and strategic objectives of corporations.
According to Medeiros (2014) the hierarchical analysis process converts the evaluated criteria in numerical values that can be processed and compared considering the entire extent of the problem. This method is more useful to teams that are involved in complex problems that need the comparasion of different alternatives and whose resolution will imply on long-term impacts (Bhushan & Rai, 2004).
While the unmeasurable qualitative criteria focus mainly at meeting the operational requirements for the use of commercial spaces, the quantitative result indicators support the decision from the economic and financial points of view, which are important to meet the corporation's investment strategies and policies.
This technique assesses the most suitable alternative of accommodation for the operational activities of each studied corporation, considering the different corporate spaces needed, and being in accordance to the operational and strategic needs of each corporation.
The final result of this analysis is the recommendation of a hierarchy between the different options, starting from the one that best meets the criteria prioritized by the stakeholders, as shown in Figure 2.

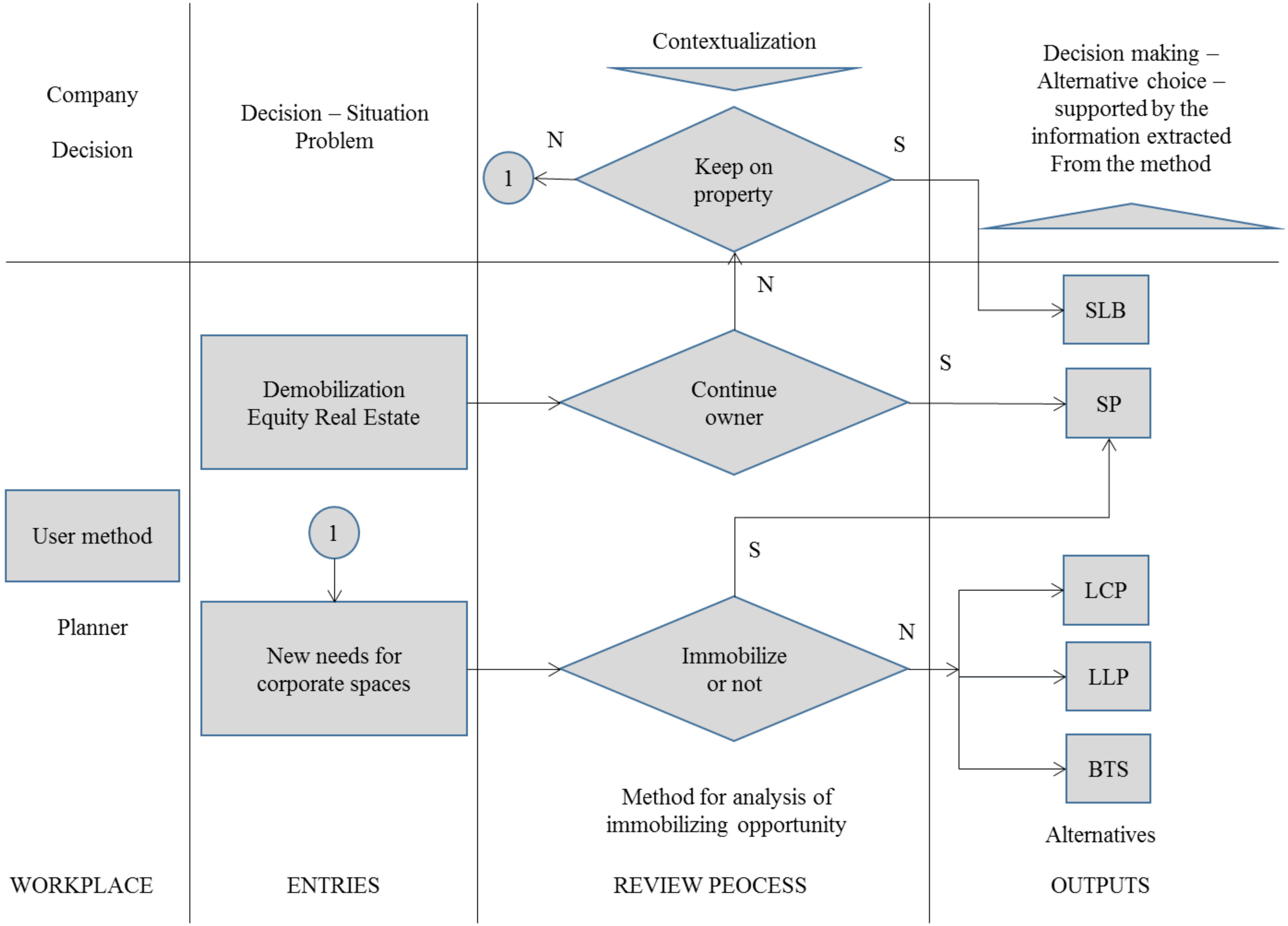

Figure 2. Article goal. Source: Adapted by the author (Gregório, 2010).

2.1 Methodology for the method configuration

Gregório (2010) states that qualitative references along with the quantitative references structured in a decision support tool configure the MAOI (Method for Analysis of Fixed Assets Corporate Real Estate Opportunity), as shown in Figure 3.
This method allows the balance between both qualitative criteria, which is related to the use of space, and quantitative data, which concerns investment strategies and policies of corporations when choosing the most appropriate solution for achieving strategic objectives and attending operational needs.

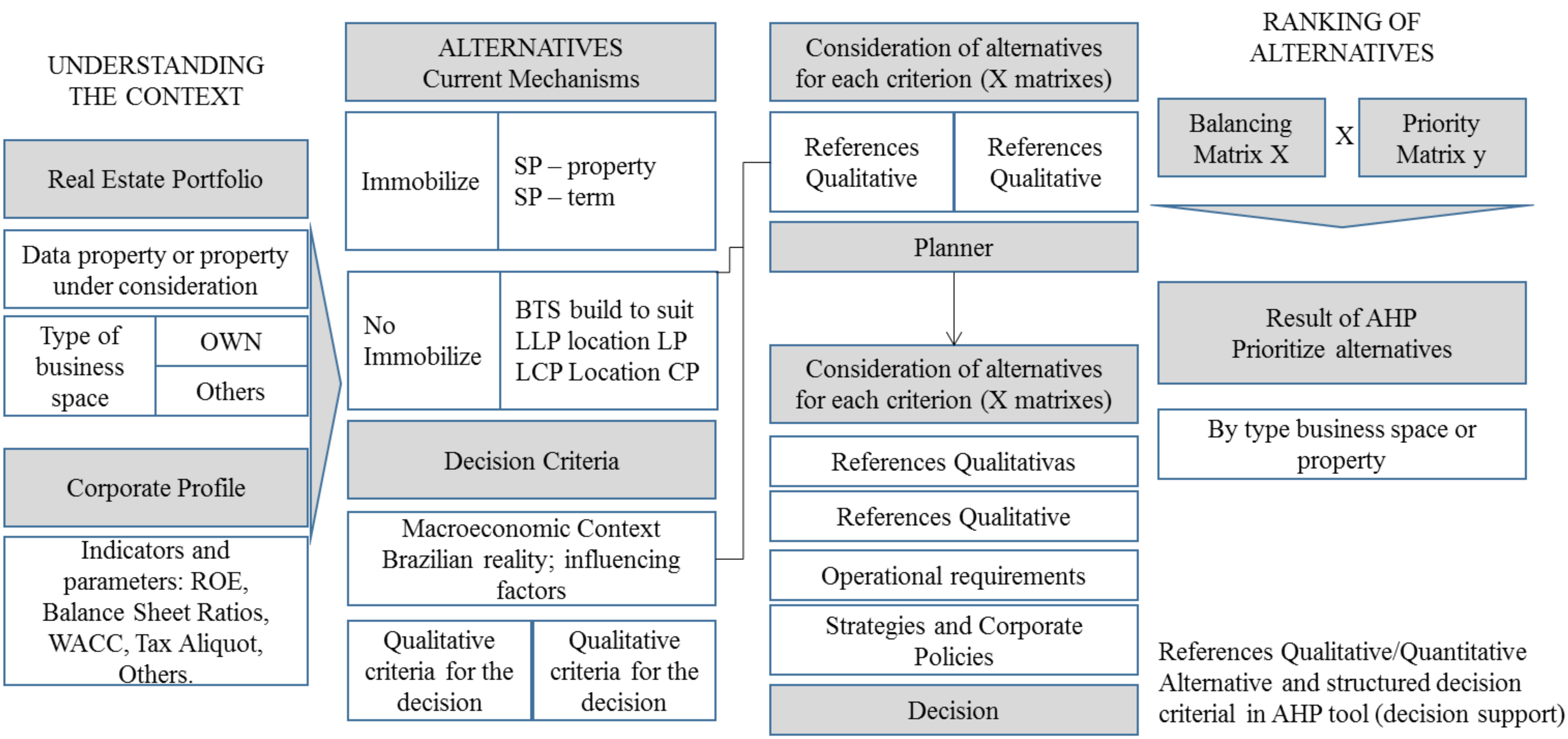

Figure 3. Method Routines for Immobilization Opportunity Analysis in Corporate Real Estate (MAOI). Source: Adapted by the author (Gregório, 2010).

The decision between immobilize, or not, the real estate enterprise, is taken based on a multi-criteria analysis process. This article has used the decision support tool known as Analytic Hierarchy Process, which provides different alternatives for each criterion, allowing the best choice according to the strategies of each company.

2.2 Quantitative analysis - measurable decision criteria

The quantitative analysis provides indicators that are relevant to the decision between immobilizate in the real estate enterprise or not. Each corporation has it's own strategy to immobilize financial resources. According to Gregório (2010), the following tests must be performed:
i. Losses and earnings of opportunities in the main business;
ii. Impacts on financial ratios of the corporation, such as the liquidity and debt ratio;
iii. Reduction of the tax impacts for each of the alternatives.

2.3 Qualitative analysis - prioritization of criteria by stakeholders

The tool used to structure the qualitative and quantitative references of MAOI was the Analytic Hierarchy Process (AHP - Analytic Hierarchy Process). This method has been developed by Saaty (1980) with the function of structuring decisions hierarchically. The models must include all important measurable factors (both quantitative and qualitative), which may be tangible or intangible and able to be compared and considered.

The main function of hierarchical analysis is to increase objectivity and reduce the subjectivity of the decision process. By the division of the decision into smaller parts, and through the comparison and correlation between criteria, it is possible to take a better choice according to the prioritization of criteria given by each corporation.

The advantage of each tool is to enable stakeholders to assign relative weights to the criteria and compare them with each other by following the scale developed by Saaty. An array of comparison for n elements is shown as follows: A = [aij] where aij = 1 / aji. Thus, it is possible to develop a comparison matrix, as shown in Table 1. All the criteria are compared and correlated following the Saaty scale, shown in Table 2. This scale can also be seen in ASTM 1765 -2011 - Standard Practice for Applying Analytical Hierarchy Process (AHP) to Multiattribute Decision Analysis of Investments Related to Buildings and Building Systems.

Table 1. Matrix of alternatives.

COMPARISON OF THE MATRIX [alternative]				
Alternative	A_1	A_2	...	A_n
A_1	1	a_{12}		a_{1n}
A_2	$a_{21}=1/a_{12}$	1		a_{2n}
...			1	
A_n	$a_{n1}=1/a_{1n}$	$a_{n2}=1/a_{2n}$		1
∑ heft (T)	$1+a_{21}...+a_{n1}$			$a_{n1}+a_{2n}+...+1$

Source: Gregório (2010).

Table 2. Scale Note

VALUES SCALE FOR COMPARISON	
Note to be assigned	Importance awarded
1	Both compared alternatives equally meet the attribute
3	Alternative X **seems to better** serve the attribute than Y
5	Alternative X **serves better the** attribute than Y
7	Alternative X **serves much better** to attribute than Y
9	Alternative X **server exceptionally better** to attribute than Y
2, 4, 6, 8	Intermediate scales to note assignment

Source: Adapted by the author (Saaty, 1991)

According to Saaty (2008), if all results are perfect in all of the comparisons, it is stablished the following condition: Aik = aij, ajk, for all i, j, k. The high number of comparisons may lead stakeholders to give an specific alternative importance gradients in a different way from as the others. The same author has also presented a consistency index (CI) for the evaluation of the comparison matrix between the different factors. The consistency index is represented in Equation 1:

$$IC = \frac{(\lambda_{máx} - n)}{(n-1)} \qquad (1)$$

Where:

λ_{max} = eigenvector obtained from the multiplication of two matrices - the first one being formed from the eigenvector (relative weight) and the second by sum of the assigned values in the comparison matrix.

n = order of a square matrix

Saaty has also introduced randomness index, which consists of a random consistency index (RI) generated for random arrays of different dimensions, as described in Table 3.

Table 3. Random consistency index values (depending on the array order)

n	1	2	3	4	5	6	7	8	9	10	11
CA	0,00	0,00	0,58	0,90	1,12	1,24	1,32	1,41	1,45	1,49	1,51

Source: National Laboratory Oakridge

With the random consistency index (RI) and the consistency index (CI) obtained by Equation 2 reason of consistency (RC):

$$RC = \frac{IC}{IR} \qquad (2)$$

According to Saaty (2008), a lower consistency reason or equal to 0.1 may be acceptable for analysis.

2.4 Case study

The selected case for study was an information technology applied to construction company, which was incubated within the Federal University of Parana. This company has an innovative profile in the information technology area, being nationally highlighted as a reference in the area BIM - Building Information Modeling.

According to the rules of ANPROTEC (National Association of Promoting Entities of Innovative Enterprises), a company can stay in the incubator for the maximum period of six months, if it is at the Pre-Incubation Program, and three years if it is in incubation. The studied company started its operation within the university in 2013, having two more years to continue their activities within the university. Thus, its facilities could not be sold, so that the hypothesis Sale Leaseback, which consists of the sale of the corporation's property, followed by a long-term contract with the investor, was dismissed for the study.

Figure 4 shows the structure of hierarchical analysis and alternative criteria based on the thesis written by Gregório (2010). The structure has an hierarchical level which consists of eight criteria organized in four qualitative and four quantitative aspects.

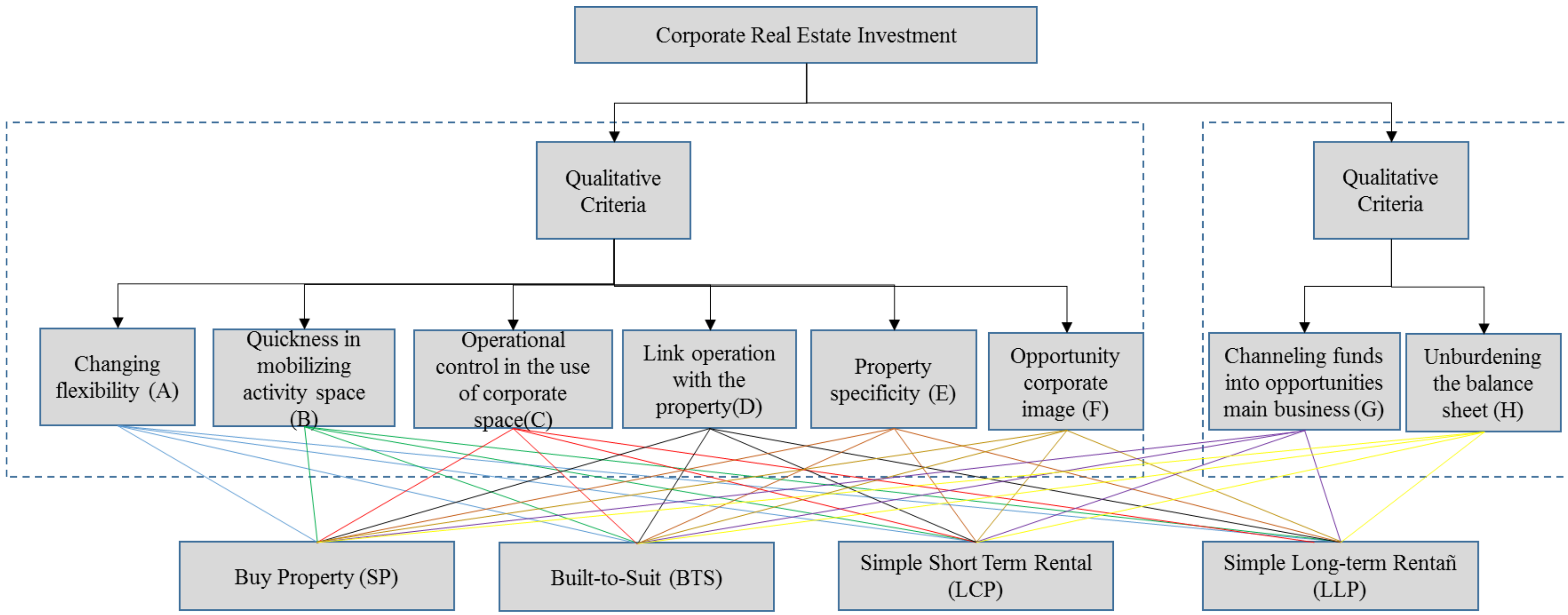

Figure 4. Hierarchical structure analysis

Table 4 shows a brief comment on each of the selected criteria, in order to help the understanding of the criteria that was used by the incubated enterprise’s owner when completing the questionnaire.

Table 4. Decision criteria

Decision criteria		
Criteria related to the use of the corporate space (qualitative criteria)	**Change flexibility (A)**	Importance of ease (time and cost) of the corporation to change it's corporate spaces as a way to meet changes in it's operational requirements, such as the required amount of area, location and / or operating demand of space.
	Quickness in mobilizing activity space (B)	Importance of quick start of operational activity in space (to meet increased demand, production growth, etc.)
	Operational control in the use of corporate space (C)	Importance of freedom on making interventions during the use (infrastructure, layout, external envelope) in order to meet the changes in operational demand (considering spaces that technically allow these interventions).
	Link operation with the property (D)	Strategic importance of the property according to the following aspects: the activity performed in it has direct dependence; the location is strategic (logistics, geographic and market aspects) and / or fixed assets investment in local facilities are significant.
	Property specificity (E)	Importance on meeting all of the defendant's specifics to the operation performed in the property, such as it's particular location, functionality and / or space architecture (which hinder their availability ready for rental market).
	Opportunity corporate image (F)	Importance of the translation of the corporate image to the operation carried out in space. This translation can be in the location's choosing process by the enterprise, on in its internal environment (when it is ready and available for rental), and on it's external environment, through elements that link the property to the corporation (facade architecture, exterior finish and logo).
Criteria related to investment strategies and corporate policy (quantitative criteria)	**Channeling funds into opportunities main business (G)**	The funds channeling can be a prioritized attribute when strategic opportunities can be foreseen in the core business.
	Unburdening the balance sheet (H)	Importance of improvement in corporate balance sheet ratios, such as liquidity ratios, debt ratios, among others.

Source: Gregório, 2010

In this paper, we used the data collection protocol presented by Gregório (2010), which enabled the corporation's data collection by the means of an interview, whose results are summarized in Table 5, as well as a questionnaire in which the planner has compared several preestablished criteria, assigning them grades according to values presented by Saaty in Table 2.

Table 5. Context description

Context Description: Decision of immobilization or not in the real estate to a new corporate space to be used	
Industry focus	Construction
Company stage	Incubation (product development)
Type of business space	Only the company's headquarters, room available UFPR / CESEC - Civil Engineering Research Centre
Uncertainties regarding the use of space	Best location for customers and employees
Specificity required in space	Open space with proper lighting and accessibility
Property market availability	Difficult because large areas are found in new buildings with high cost
Property	Single user
Activity held	Administrative, production and customer area
Predominant interventions during use	Internet networks and multimedia equipment
Property importance	Company strength, better structure for employees
Corporation rating	Reference in the use of BIM technology in Brazil
Translation of the desired image	Company innovative, competent and committed
Start of activities in the new space	Must occur within one year
Condition the option for immobilization	Bank financing
Balance situation	Consolidation
Relative income of activated working capital	-
Preferred legal instrument for the alternatives of not immobilizing	Lease long-term

From the description of the context and the results obtained from the questionnaire, it was possible to stablish degrees of importance for every decision criterion, which were later compared and weighted by the comparing array of both quantitative and qualitative alternatives.

3. ANALYSIS OF RESULTS

From the answers obtained in interviews with professionals from the real estate sector, as described by Gregório (2010), it was possible to calculate their weights, based on the relevance of each alternative for every decision criterion, as shown in Table 6.

Table 6. Comparison matrix for decision criteria - Change flexibility.

Change flexibility	Property purchase	*Built-to-Suit*	Lease short term	Long-term lease	Σ line	Variable helf
Property Purchase	**1,00**	2,00	0,13	0,20	3,33	8,65%
Built-to-Suit	0,50	**1,00**	0,11	0,17	1,78	4,63%
Lease short term	8,00	9,00	**1,00**	3,00	21,00	54,64%
Long-term lease	5,00	6,00	0,33	**1,00**	12,33	32,09%
Σ matrix helf	14,50	18,00	1,57	4,37	38,44	1,00

Source: Adapted by the author (Gregório, 2010).

However, given that the presentation of all decision criteria individually would result in an excess of information, in this paper we have chosen to summarize these answers in form of a table (Table 7), which shows, from the values that were obtained according to the procedures exemplified in Table 6, a summary of the weight of the studied variables and also the individual weight of each decision criterion.

Table 7. Full table with comparison matrix

	Variable helf			
	Alternatives			
Criteria	**(1)**	**(2)**	**(3)**	**(4)**
Change flexibility (A)	8,65%	4,63%	54,64%	32,09%
Activity mobilization of how quickly the space (B)	22,94%	6,31%	38,24%	32,50%
Control the use of space (C)	49,67%	32,24%	3,93%	14,16%
Link operation with the property (D)	43,43%	30,40%	4,22%	21,95%
Property specificity (E)	34,31%	43,20%	3,56%	18,93%
Translation of corporate image (F)	32,39%	32,39%	6,88%	28,34%

Source: Adapted by the author (Gregório, 2010).

The importance of each of the decision criteria is shown in in Table 8, while in Table 9 it is shown the comparison matrix between the quality criteria. In Table 10 the quantitative criteria are compared. All of them are scored according to the rating scale developed by Saaty.
It is worth to emphasize that the criteria adopted as quantitative refer to economic performance, while the qualitative criteria address the technical performance of the building.

Table 8. Importance of decision criteria developed by the parties in accordance with the values established by Saaty (1991)

Criteria	9	7	5	3	1	3	5	7	9	
Change flexibility					x					Activity mobilization of how quickly the space
Change flexibility				x						Control the use of space
Change flexibility				x						Link operation with the property
Change flexibility				x						Property specificity
Change flexibility					x					Translation of corporate image
Activity mobilization of how quickly the space			x							Control the use of space
Activity mobilization of how quickly the space					x					Link operation with the property
Activity mobilization of how quickly the space					x					Property specificity
Activity mobilization of how quickly the space					x					Translation of corporate image
Control the use of space					x					Link operation with the property
Control the use of space					x					Property specificity
Control the use of space						x				Translation of corporate image
Link operation with the property					x					Property specificity
Link operation with the property						x				Translation of corporate image
Property specificity					x					Translation of corporate image

Table 9. Comparison matrix between the qualitative criteria

	(A)	(B)	(C)	(D)	(E)	(F)	Σ line	Variable helf
(A)	1,00	1,00	0,33	0,33	0,33	1,00	4,00	0,90
(B)	1,00	1,00	0,20	1,00	1,00	1,00	5,20	0,11
(C)	3,00	5,00	1,00	1,00	1,00	3,00	14,00	0,30
(D)	3,00	1,00	1,00	1,00	1,00	3,00	11,00	0,23
(E)	3,00	1,00	1,00	1,00	1,00	1,00	8,00	0,17
(F)	1,00	1,00	0,33	0,33	1,00	1,00	4,67	0,10

Table 10. Comparison matrix between the quantitative criteria

	(G)	(H)	Σ line	Variable helf
(G)	1,00	3,00	4,00	0,75
(H)	0,33	1,00	1,33	0,25

These data were collected and tested following the techniques reported by Saaty (2008). After the calculation of the decision matrix eigenvalue ($\lambda_{máx}$) we have calculated the consistency index of the decision matrix using Equation 1.

- Consistency Index for qualitative criteria:

$$IC = \frac{(\lambda_{máx} - N)}{N - 1} = 0{,}07489$$

- Consistency Index for the quantitative criteria:

$$IC = \frac{(\lambda_{máx} - N)}{N - 1} = 0{,}00$$

Costa (2006) proposes the use of a random consistency index (IR), as seen in Table 3, for a reciprocal matrix of order of n, without negative nor randomly generated elements.
By using the data obtained from the consistency index and the random consistency index in equation 2, we have obtained the consistency ratio (RC).

- Consistency Ratio for the qualitative criteria:

$$RC = \frac{IC}{IR} = \frac{0{,}007489}{1{,}24} = 0{,}06039$$

- Consistency Index for the quantitative criteria:

$$RC = \frac{IC}{IR} = \frac{0{,}00}{0{,}00} = 0{,}00$$

According to Saaty (2008), the value of consistency reason lies within the recommended amount, i.e., RC <0.1. When comparing the results between the calculated consistency of reason, and the reason of consistency established by Saaty, it can be observed that the values that were provided by the owner of the enterprise incubated within the Federal University of Paraná have enough consistency.
To check the alternative that best meets the company studied, we have correlated the set out criteria with possible alternatives. Next, the criteria's weights were applied to each of the correlated values, in order to find the alternative that best suited the company's needs, that is, the one with the highest score. Table 11 shows their relative performance as the qualitative criteria.

Table 11. Relative Performance as for Qualitative criteria

Criteria	**Property purchase (CI)**	***Built-to-Suit* (BTS)**	**Lease short term (LCP)**	**Lease long-term (LLP)**	**Variable helf (PV) (%)**
(A)	0,16	0,08	1,00	0,59	8,72
(B)	0,60	0,17	1,00	0,85	11,34
(C)	1,00	0,65	0,08	0,29	30,52
(D)	1,00	0,70	0,10	0,51	21,80
(E)	0,79	1,00	0,08	0,44	17,44
(F)	1,00	1,00	0,21	0,88	10,17

(Continued Table 10)

Criteria	**(CI x PV)**	**(BTS x PV)**	**(LCP x PV)**	**(LLP x PV)**
(A)	1,38	0,74	8,72	5,12
(B)	6,80	1,87	11,34	9,64
(C)	30,52	19,81	2,42	8,70
(D)	21,80	15,26	2,12	11,02
(E)	13,85	17,44	1,44	7,64
(F)	10,17	10,17	2,16	8,90
Performance Index (∑column)	**84,53**	**65,30**	**28,20**	**51,03**

While Table 12 shows the relative performance relating to the quantitative criteria.

Table 12. Relative Performance as the quantitative criteria

Criteria	**Property purchase (CI)**	***Built-to-Suit* (BTS)**	**Lease short term (LCP)**	**Lease long-term (LLP)**	**Variable helf (PV) (%)**
(G)	0,25	1,00	1,00	1,00	0,75
(H)	0,21	1,00	0,88	1,00	0,25
Criteria	(CI x PV)	(BTS x PV)	(LCP x PV)	(LLP x PV)	
(G)	18,75	75,00	75,00	75,00	
(H)	5,16	25,00	21,88	25,00	
Performance Index	**23,91**	**100,00**	**96,88**	**100,00**	

Thus, for better viewing, Table 13 shows in decreasing order, the results of economic attractiveness obtained for each of the evaluated alternatives in the case study.

Table 13. Attractiveness economic

Alternatives	**Economic performance index (25%)**	**Technica performance index l (75%)**	**Rating (100%)**
Built-to-Suit	100	65,30	73,97
Compra do imóvel	23,91	84,53	69,37
Lease short term	100	51,03	63,28
Lease short term	96,88	28,20	45,37

As we could verify by using the method of hierarchical analysis, the most viable alternative to the selected case is the Built-to-Suit. However, during the interview for contextualization of the company, it has been said that the preferred alternative would be the long term lease. This alternative obtained the third highest rate in view, as it would sucessfully fulfill many of the aspirations of the incubated company.

4. CONCLUSION

This article was written based on the use of hierarchical analysis method for the decision on imobilize or not their assets in real estate. The chosen method provides parameters for better assessment of alternatives, as it is based on quantitative and unmeasurable qualitative criteria. The qualitative process correlates all the criteria, giving them grades according to the scale previously proposed by Saaty.
The final result of this analysis provided numerical indices that allow to define, in a systematic way, the best alternative for the case study, given that the company incubated within the Federal University of Paraná has, according to ANPROTEC three more years to remain installed at the University.
For this case, it was seen that the alternative with the best performance was th Buit-to-suit, which serves more broadly the company's needs, although in the interview it was said that that the preferred option was the lease long term. This option obtained the third highest rate in view, attending many of the aspirations of the incubated company, only not more effectively.
Despite the company’s preference, chosing Built-to-suit (BTS), may not be, tough, the best alternative, since the BTS operation implies the construction of a custom property for the specific use by the company, being linked to a long term lease. It is for the company to find investors to assume the construction of this property. Usually, this operation is used as part of the investment portfolio only for large leased areas (a property with large or several smaller properties) for the same client.
Nonetheless, both alternatives – best option (Buit-to-suit) and preference of the company (long-term lease) – are similar on what regards the decision not to immobilize the capital in the purchase of a property.
It should be noticed that the used method was intrinsically based on interviews with the company’s representants, so that the achieved results are essentially constitued on subjective data, as the interview tends to be a subjective method. Thus, extrinsic changes, such as the respondent's stress level during the evaluation of responses, may influence directly on the consistency of the results.
It is also noteworthy that this work had the purpose of evaluating the best solution for a single company, so that the result obtained does not necessarily represent a statistical universe of the same size companies, practice area, etc. The used method, however, can be applied to any related study.

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