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ORIGINAL ARTICLE

Structural retrofitting method for evaluating RC Jacketing in columns with amplification of moments under seismic loads

Método de reforçamento estrutural para avaliação do encamisamento em pilares com amplificação de momentos sob cargas sísmicas

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Abstract: Different reasons can make certain structures need reinforcement to achieve specific levels of safety and performance. The occurrence of events of significant magnitudes, such as earthquakes, are examples of this. Retrofitting vulnerable structures becomes a practice to mitigate the destructive effects of earthquakes, and the RC Jacketing becomes an alternative. The present work studies this type of reinforcement, proposing and applying an assessment methodology under vulnerable construction built in a high seismic risk zone. The diagnostic of the current situation was determined, and the structural suitability was evaluated using RC Jacketing. With the computational software S-Model, created in this research, the effectiveness of the proposed reinforcement was verified, based on the results of the analysis carried out with commercial software. The preload was considered, and the strength of the reinforced column section was verified against all load combinations. The S-Model program was able to verify the non-simultaneous occurrence of the amplification of moments with the most critical loading. Both conditions demand more significant stresses on the element, and therefore the calculation of the steel areas of the column may be underestimated if they are not considered.

Keywords: jacketing, amplification of moments, seismic vulnerability, structural retrofitting, software.

Resumo: Existem diferentes razões que podem fazer com que certas estruturas precisem de reforço para atingir níveis específicos de segurança e desempenho. A ocorrência de eventos de grande magnitude como os sismos são um exemplo disso. Para mitigar os efeitos destrutivos dos terremotos a adequação de estruturas vulneráveis torna-se uma prática e o encamisamento em concreto armado converte-se em uma alternativa. O presente trabalho estuda esse tipo de reforço, propondo e aplicando uma metodologia de avaliação sob uma edificação vulnerável, construída em uma zona de alto risco sísmico. O diagnóstico da situação atual foi determinado e a adequação estrutural foi avaliada usando concreto de encamisamento. Com o programa computacional S-Model, criado nesta pesquisa, verificou-se a efetividade do reforço proposto, a partir dos resultados da análise feita com um programa comercial. A pré-carga foi considerada e se verificou a resistência da seção do pilar reforçado ante todas as combinações de carga. O programa S-Model foi capaz de verificar a não ocorrência simultânea da amplificação de momentos com o carregamento mais crítico. Ambas as condições demandam maiores esforços solicitantes sobre o elemento, e por tanto o cálculo das áreas de aço do pilar podem resultar subestimadas se não são levados em conta.

Palavras-chave: encamisamento, amplificação de momentos, vulnerabilidade sísmica, reforçamento estrutural, software.

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1 INTRODUCTION

Certain structures require structural reinforcement to achieve specific levels of safety and performance. According to Rodríguez et al. [1] such structural reinforcement is necessary for three main reasons: (i) change of use of the building, in cases where efforts are increased in the structure, and a higher level of performance is required; (ii) construction quality problems (e.g., low concrete strength, insufficient placement of reinforcement, etc.) in this case, the safety of the structure is reduced; and (iii) changes in regulations caused by extreme events (e.g., earthquake, wind, etc.). The last item mainly affects old constructions built in seismic zones, which can become vulnerable over their lifetime when catastrophic events happen.

Amid this reality, studies in earthquake engineering are developed and updated, which are usually accompanied by new technical parameters that require changes to be made to current standards. As a measure to mitigate this vulnerability, projects and the consequent works of structural adjustment must be carried out, a subject that demands the formulation of the feasible alternative for reinforcement and the relevant validation, Yepez Aguirre [2].

Adaptation methods and techniques for reinforced concrete structures have shown rapid evolution, generally based on experience and craftsmanship practices, due to their characteristics or the peculiarities of each case. Furthermore, there is little information in the current regulations regarding the methodology to be followed to analyze the mechanical behavior of the reinforced element in a more realistic way. Even though these elements have poor structural behavior, they actively support the acting loads, which must be considered both in computational modeling and the reinforcement construction process.

Within this scenario, structural calculators, who mostly use commercial computer software, are left with few guidance tools to face these types of problems; carrying out the verification of the reinforcement proposal in a more rational way becomes an almost impractical task. The S-Model program developed by Yepez [2] can be used when one intends to perform reinforcement of columns using RC Jacketing. It was created as a computational calculation tool that complements the dimensioning done using commercial programs. The main advantages are consideration of the preload and verification of the strength of the column reinforced section against all load combinations, including those that may be more unfavorable due to the simultaneous action of amplification of moments with the critical load. These conditions demand greater efforts on the cross-section of the element, and therefore the calculation of the column steel areas can be underestimated if they are not considered. Therefore, it is of great importance that the calculator performs this verification when the proposal of reinforcement of the column includes RC Jacketing.

Increasing the cross-section or RC Jacketing is one of the most used techniques to adapt reinforced concrete columns, as it leads to an increase in the axial strength, bending, rigidity, and ductility of the original column. It also has comparative advantages over other reinforcement methods, among which the low price of materials, the lower need for qualified labor, and its good structural performance stand out, all supported by a large number of experimental tests, including Ersoy et al. [3], Julio et al. [4], and Krainskyi et al. [5].

This research proposes a step-by-step procedure to evaluate RC Jacketing as a seismic retrofitting technique for concrete structures, including the verification of the column cross-section using S-Model. An old building was chosen as a case study to apply the methodology, presenting visible pathological manifestations and seismically unfavorable constructive characteristics. The initial stages include a three-dimensional (3D) analysis of the building to determine the diagnosis of the structural system. These results will indicate the need to perform a new 3D analysis, in this case, including the reinforcement alternative. The procedure ends with the isolated verification of the element using the computer software S-Model.

1.1 Seismicity in the study area

Earthquake events of the last sixty years worldwide revealed numerous weaknesses of reinforced concrete (RC) structures built in the 1950s–1970s period or previously [6-8]. In this paper, the study zone comprises the region of the Venezuelan Andes, in the west of the country, which is a high seismic hazard. This threat is mainly caused by the seismic activity associated with the Boconó fault, which is of the current dextral type, and by the inverse faults located on both flanks of the Andes, Audemar and Audemar [9].

The most important historic earthquakes in this region occurred in the years: (i) 1610 with a magnitude of 7.5; (ii) 1894 with a magnitude of 7.3 called "The Great Andes Earthquake"; and (iii) 1932 of magnitude 6.8. The case study was built in the Trujillo State, which belongs to the Andean zone, specifically in Isnotú city. The earthquakes that caused damage in this state happened in the years 1674, classified as very harmful, and in the years 1775, 1894, and 1950, these last three classified as severe, Hernández and Schmitz [10].

Return periods of 135-460 years for magnitudes 8, 45-70 years for magnitudes 7, and 7-15 years for magnitudes 6. These were estimated using, as a database, the events reported in the western region from the year 1590 to the present, Estevez and Laffaille [11].

1.2 Structural vulnerability

Living in a region smothered by the seismic threat makes the earthquake-resistant construction of new buildings or the adaptation of existing ones a prescript to neutralize or reduce the destructive effects of earthquakes. The magnitude of these effects will depend on the structural condition of the edification, that is, on its vulnerability. Determining the vulnerability condition of a building can be done by qualitative and quantitative means. In the latter case, numerical simulation was used, from which it was possible to assess whether the structural elements adequately resist the seismic loads.

The building's response to a seismic event is mainly associated with the intrinsic conditions of its construction, the architectural configuration, and the structural and environmental conditions surrounding it. This proclivity to be harmed measures its vulnerability. Common structural problems can be found in buildings, making them vulnerable, for example: (i) improperly arranged armatures in structural elements; (ii) insufficient transverse reinforcement to guarantee confinement and avoid shear failures, mainly in short columns; (iii) frames with strong beam/weak column configuration; and (iv) architecture with short columns. Another essential aspect to consider is related to the irregularity of the building, both horizontally (plan) and vertically (elevation).

A plan irregularity is considered if there is a great eccentricity in any of its main directions, high risk of torsion, non-orthogonal system, or flexible diaphragm. In the vertical direction, structural irregularity must be considered if we have any of the following aspects: flexible pavement, irregular distribution of masses on adjacent pavements, increase in masses with elevation, variations in the geometry of the structural system, excessive slenderness, discontinuity in the plane of the system resistant to lateral loads, absence of connection between the vertical elements, or the effect of a short column in the frames.

1.3 Seismic retrofitting

Tsonos and Kalogeropoulos [12] indicate that the devastating social and economic impact caused by catastrophic collapses triggered the reformation and refinement of the building codes while also clearly demonstrating the immense need for retrofitting of the existing RC structures. The latter are framed structures with low strength and ductility as well as poor deformation capacity. Structures built before the publication of the current Venezuelan normative codes, those that presented changes in use implying greater requests, and those that were built in disregard of the norms, are vulnerable and need to be adapted.

The Comisión Venezolana de Normas Industriales establishes the procedures for the development of projects for earthquake-resistant buildings, COVENIN, 1756-01 [13] and considers that buildings with evident signs of deterioration in the support structure are feasible for evaluation, adaptation, or repair in its structural system. The building's earthquake-resistant capacity curve defines the threshold that separates good from bad performance sees Figure 1.

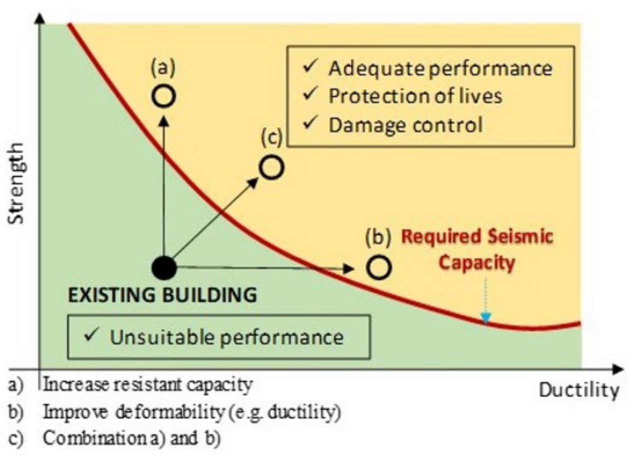

Figure 1. Earthquake-resistant capacity curve.

The balance point between ductility and strength must be within the safety zone, above the curve. Whatever the reinforcement treatment to be followed for the structure, it is necessary to define beforehand several parameters, indicated below.

(i) Building use: use is the object of its operation and is classified by groups (A, B1, B2, C), each of which is defined as an importance factor.

(ii) Types of structure: It is a classification based on the components of the earthquake-resistant system, and they are classified by Roman numerals (I, II, III, IV). Except for type IV, all others must have diaphragms with the necessary rigidity and strength to effectively distribute the seismic actions.

(iii) Levels of detailing (ND): These levels consider the energy dissipation capacity of the structure in the inelastic regime and depend on the effectiveness of the reinforcement detailing to guarantee the adequate ductility of the element. They are classified as ND1, ND2, and ND3, from less to greater rigor in detailing. For an existing building, when the scope is to verify and if there is little information about the structure, ND will depend mainly on the age and type of structure.

(iv) Reduction factor: Reduction factors (R) derive their name from the fact that they are a coefficient that reduces elastic seismic forces for long period modes of vibration. However, for short periods, the reduction is smaller, although it remains associated with the reduction factor R. For an existing building, the factor R depends on the structure's capacity to dissipate energy in the inelastic range, and therefore on the level of detail of the reinforcements contained in the element.

In addition to the standard indicated above, those for the calculation of actions, COVENIN, 2002-88 [14] and those for reinforced concrete structures, COVENIN, 1753-06 [15] are used.

1.4 Verification using S-Model

In practice, designers carry out several arbitrations to numerically simulate the behavior of columns with RC Jacketing. With S-Model software is possible to verify the Jacketing-type reinforcement on columns from the results obtained using commercial software. The model considers parameters not available in the software commonly used by calculators, especially regarding: (i) performance of loads on the column in both the original and strengthen configurations; (ii) consideration of the individual mechanical characteristics of both configurations; and (iii) consideration of slenderness and deferred deformations in the ultimate strength of the column.

The S-Model follows the recommendations of the American Concrete Institute, ACI 318-14 [16] to consider both preload and moment amplification caused by slenderness effects. Below, there is a detailed description of the parameters related to the simplified model and its functioning.

1.4.1 ACI Moment Amplification Method for Frames with Fixed Nodes

Figure 2 shows a column with simple curvature, submitted to both the action of an axial compression load (P) and the action of a moment (M_e) at the ends, Nilson [17]. P_C represents the critical load due to column buckling or Euler's critical load. The variables y_o and y represent the column deflection originated by M_o and (M_o +P), respectively. The variables Δ_o and Δ represent the deflection in the middle of the column originated by M_o and (M_o +P).

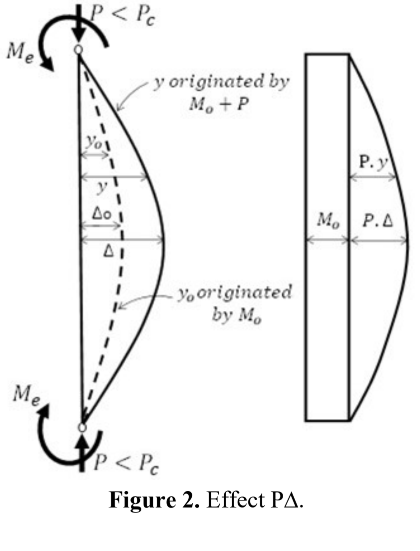

Figure 2. Effect PΔ.

The moment amplification identified by the factor δ represents the increment of the first-order moment (M_o) due to axial force P (see Equation 1). Figure 2 also illustrates the total moment in the column, which is equivalent to the sum of the portion of the first-order effects (M_o) that act in the presence of P and the portion of the additional moment produced by P times the lateral deflection (y), this last term more commonly called the P-delta (P.Δ) effect, see Equation 2.

$$\ddot{a} = \frac{1}{\left(1-\frac{P}{P_C}\right)} \tag{1}$$

$$M = Mo + P \cdot y \tag{2}$$

The lateral deflection (y) of elastic columns can be calculated from the column deflections without axial loading, Timoshenko and Gere [18] also; at the point of the maximum moment the deflection takes the value of Δ, so the above equation can be rewritten as indicated in Equation 3. According to Johnson [19] Equation 3 can be rewritten as a function of a coefficient ψ. This last coefficient depends on the type of loading, and it varies approximately between ±0.20 in most practical cases, see Equation 4.

$$M_{máx} = Mo + P \cdot \Delta = Mo + P \cdot \Delta o \cdot \frac{1}{\left(1-\frac{P}{P_C}\right)} \tag{3}$$

$$M_{máx} = Mo \frac{\left(1+\emptyset\frac{P}{P_C}\right)}{\left(1-\frac{P}{P_C}\right)} \tag{4}$$

Considering that the P/P_C ratio is always significantly less than 1, we observe that the second term in the numerator of Equation 4 is much smaller than unity, and this term can be neglected; with this, we finally obtain a simplified expression for the calculation of maximum moment in Equation 5. Substituting in Equation 5 the moment amplification factor from Equation 1, we obtain Equation 6, and the moment can be calculated. Johnson [19] shows that the moment amplification (δ) depends on the relative magnitude of the moments at the ends of the column (M_1 and M_2) where $M_1<M_2$.

$$M_{máx} = Mo \cdot \frac{1}{\left(1-\frac{P}{P_C}\right)} \tag{5}$$

$$M_{máx} = Mo \cdot \ddot{a} \tag{6}$$

The ACI, 318-14 [16] indicates that slender reinforced concrete columns reach the limit of their strength when the combination of axial load (P) and bending moment (M) in the section subjected to maximum effort produces the failure. In this case, P and M convert to P_N and M_N, shown in Figure 3. $P_{N\ robust}$ represents the nominal load of a short column for a given eccentricity (e_o), and is in point A. On the line segment oA, the rate M/P is constant due to the absence of defections in the column. $P_{N\ slender}$ represents the nominal load of a slender column for the same eccentricity (e_o), and is in point B. We have two behaviors on line segment oB; initially the rate M/P is constant due to the absence of defections in the column, after this rate is not linear due to slenderness effects. The first-order moment (M_o) is the product between $P_{N\ slender}$ and eccentricity (e_o), and $M_{máx}$ is Mo amplified by δ.

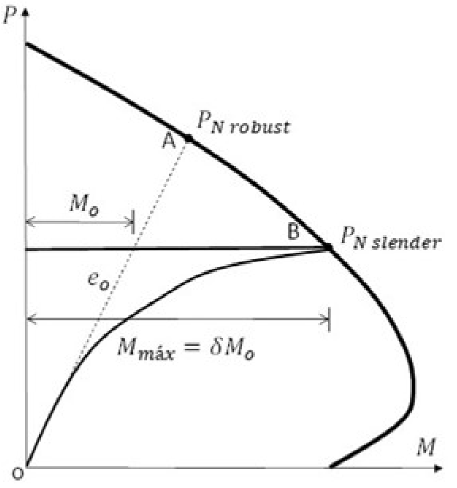

Figure 3. Effect of slenderness on carrying capacity.

The interaction curve describes the strength capacity of both a robust column with negligible moment amplification at point A and a slender column with significant moment amplification as a function of the load increase at point B. This relationship can be considered by including the factor C_m, see Equation 7, whose determination is defined by Equation 8. The quotient M_1/M_2 will be positive whenever the moments at the ends of the column produce simple curvature and will be negative double curvature is produced.

$$M_{máx} = Mo\frac{C_m}{\left(1-\frac{P}{P_c}\right)} \tag{7}$$

$$C_m = 0.6 + 0.4\frac{M_1}{M_2} \geq 0.4 \tag{8}$$

The ACI 318-14 [16] determines the moment amplification for fixed-node frames, which acts simultaneously with the increased axial load (P_u), using Equation 9. In this case, the moment amplification factor is defined by Equation 10. The value of 0.75 corresponds to the coefficient of reduction of the strength, considered to estimate a conservative value of P_C.

$$M_{máx} = ä \cdot M_2 \tag{9}$$

$$ä = \frac{C_m}{\left(1-\frac{P_u}{0.75P_c}\right)} \geq 1 \tag{10}$$

1.4.2 The S-Model Software applied to RC Jacketing columns

In the case of RC Jacketing retrofitting columns, subjected to the action of an eccentric normal compression load, the second-order effects due to slenderness and deferred deformations are greater when the Jacketing is performed under the action of the load on the original column (preload), see Figure 4.

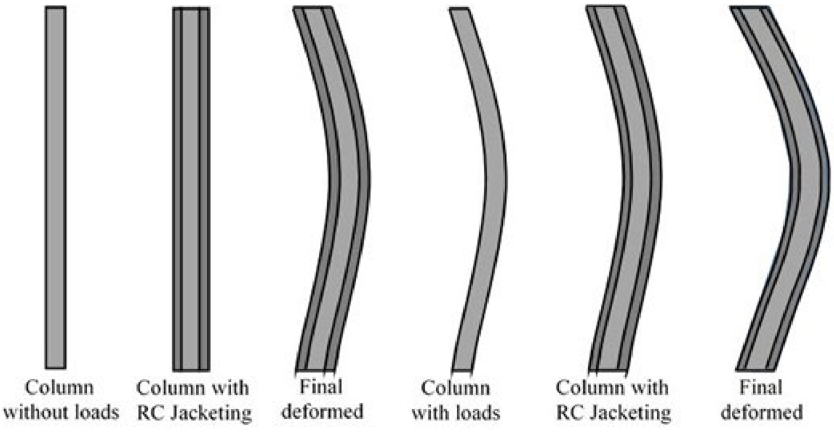

Figure 4. Columns deformed with and without preload.

The general equations indicated in the previous paragraphs were used to establish the formulations included in the software. The subscripts sj and cj are equivalent to without and with RC Jacketing, respectively. The addition of second-order effects, in the condition indicated in the previous paragraph, is considered in the S-Model software, which calculates the deformations in two steps. The first step corresponds to the calculation of deformations on the column without RC Jacketing (Δ_{sj}) see Equation 11, which is obtained by isolating Δ from Equation 3.

$$\Delta_{sj}= \frac{\left(M_{m\acute{A}Xsj}-M_{osj}\right)}{P_{sj}} \tag{11}$$

The maximum moment is $M_{m\acute{a}xsj}$, and the first-order moment is M_{osj}. Afterward, the expression of the maximum moment of Equation 7 is substituted in Equation 11, making simplifications we obtain Equation 12, which defines the maximum lateral deformation of the column in the initial stage of the RC Jacketing construction. The second step corresponds to the calculation of the lateral deformation of the column with Jacketing at the moment of failure (Δ_{cj}), see Equation 12, which is obtained by isolating Δ from Equation 3. Isolating M_o from Equation 7, replacing this expression in Equation 13, and making the respective adjustments, we obtain Equation 14.

$$\Delta_{\mathrm{sj}}= \frac{M_{osj}}{P_{sj}}\left(\frac{Cm_{sj}P_{Csj}-P_{Csj}+P_{sj}}{P_{Csj}-P_{sj}}\right) \tag{12}$$

$$\Delta_{cj}= \frac{\left(M_{m\acute{a}xcj}-M_{ocj}\right)}{P_{cj}} \tag{13}$$

$$\Delta_{cj}= M_{m\acute{a}xcj}\left(\frac{Cm_{cj}P_{Ccj}-P_{Ccj}+P_{cj}}{Cm_{cj}P_{Ccj}P_{cj}}\right) \tag{14}$$

In this phase, the value of the load applied to an eccentricity e_o what causes the column to fail (P_{cj}), and the value of the maximum moment ($M_{m\acute{a}xcj}$) is determined. Both values correspond to point B indicated above in Figure 3. A portion of the deflection Δ_{cj} corresponds to the deflection Δ_{sj}, calculated for the original column due to the moment $M_{m\acute{a}xsj}$; therefore, to calculate the deflection in the column reinforced with Jacketing, this moment must be subtracted and to the deflection obtained add Δ_{sj}. Finally, we have an expression in Equation 15 to determine the total deflection in the column, considering the second-order effects in both stages, before and after, the construction of the Jacketing reinforcement.

$$\Delta_{cj}= \left(M_{m\acute{a}xcj} - M_{m\acute{a}xsj}\right)\left(\frac{Cm_{cj}P_{Ccj}-P_{Ccj}+P_{cj}}{Cm_{cj}P_{Ccj}P_{cj}}\right) + \Delta_{sj} \tag{15}$$

2 METHOD

The procedure recommended in this work consists of:

(i) Step 1: Case Study. It refers to the collection of information and the description of the case study, characterizing the building and its surroundings. Generally, when the building to be studied is ancient, the available technical information is incomplete or non-existent. Therefore, the information must be obtained by other means, carrying out work in situ.

(ii) Step 2: Previous Studies. It mainly refers to realizing the planialtimetric survey of the building and its surroundings, the survey of the symptoms of failures in the structure, and experimental tests in the existing structure. These tests can be destructive and non-destructive to verify the quality of the materials specified in the project. Activities such as in situ explorations of the foundation system, scarification of the cover to verify the type, diameter, and state of the reinforcement, determination of the carbonation depth, topographical leveling, geotechnical studies, among others, may also be included.

(iii) Step 3: Structural Diagnosis. It refers to creating a 3D digital twin to perform the numerical analysis of the behavior of the structural system using commercial software. The information obtained in the previous steps constitutes part of the data entry (INPUT) of the analysis; thus, the structural behavior of the building can be simulated more realistically. The capacity of the structure to resist requesting loads in the Ultimate Limit State (ULS) and the verification of horizontal displacements in the Service Limit State (SLS) can be determined. With this, it is possible to know the vulnerability of the building and the need for adaptation.

(iv) Step 4: Structural Adaptation. In this step, the different reinforcement alternatives are discussed to evaluate and select the most appropriate one finally. Then, on the 3D digital twin of the previous step, the chosen reinforcement is included, and a second analysis is made to verify the effectiveness of the adopted system. The structure is considered adequate when both ULS and SLS indicated in the codes are verified.

(v) Step 5: Verification. It refers to the analysis of columns as isolated elements, including preload and subsequent the verification of strength for all requesting combinations. The S-Model software is used to perform this verification and evaluate the performance of the reinforcement layer against the acting requests.

Figure 5 shows a methodology scheme and then details each of the steps indicated above, with application in the case study.

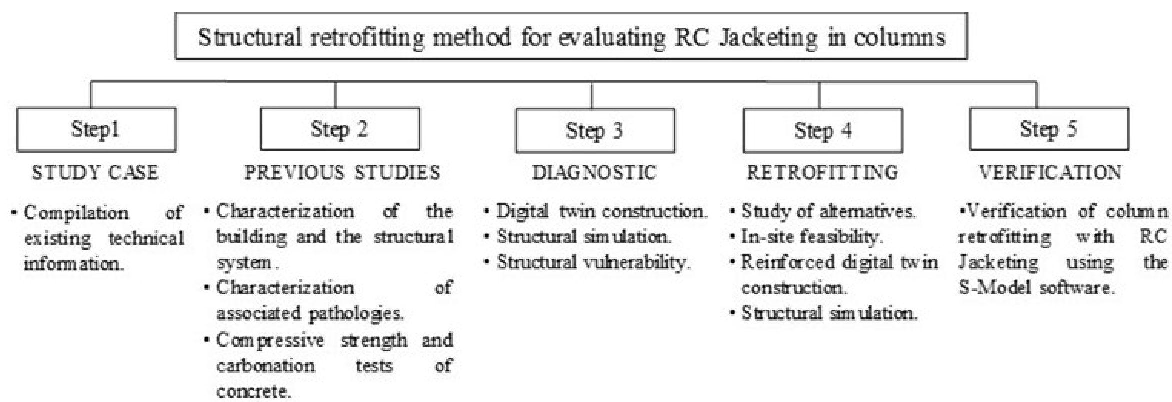

Figure 5. Method.

2.1 Step 1: Case Study

The “Escuela Samuel Darío Maldonado” (ESDM) was built in an area of Venezuela that has a high seismic risk. The project was carried out by the Ministry of Public Works (MOP) in the fifties, using less demanding standards than today, see Figure 6.

Figure 6. Main facade of the building.

The building is in the western Isnotú city, Trujillo state, on the Andes Mountains. It has a structural archetype analogous to a school building located in another region but under the same tectonic fault, destroyed in the last devastating earthquake that happened in 1997 in that same country. The collapsed building was the "Liceo Raimundo Martinez Centeno" (LRMC) located in the eastern city of Cariaco, see Figure 7.

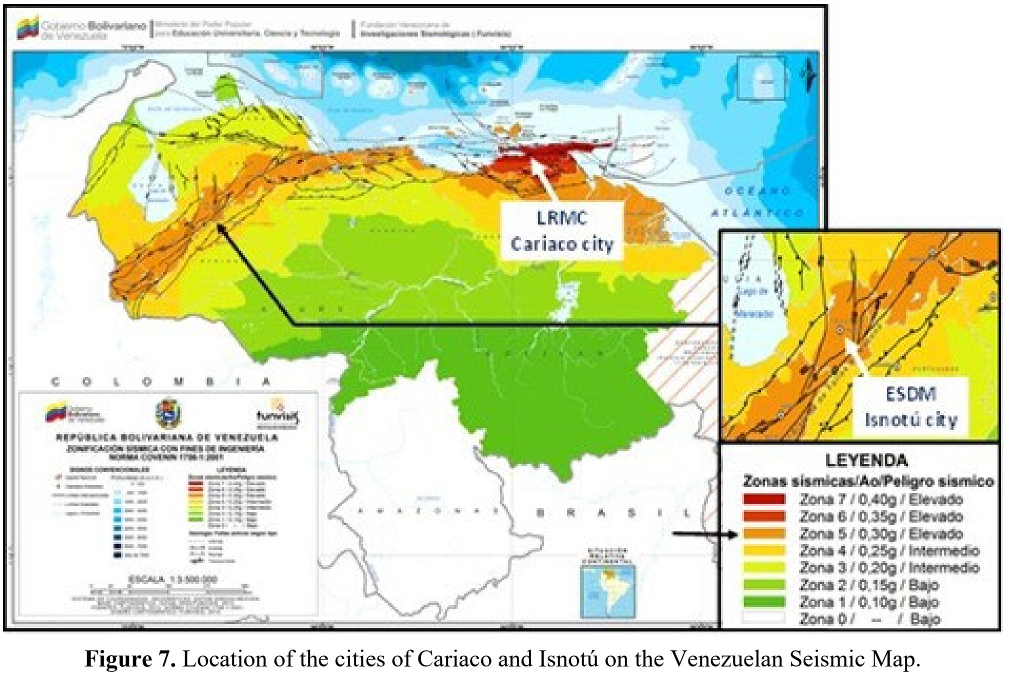

Figure 7. Location of the cities of Cariaco and Isnotú on the Venezuelan Seismic Map.

Both schools have a construction typology called Old Type Construction, which characterized educational buildings from that age. Its construction was scattered throughout the national territory; examples of this are the schools mentioned above. The predominant type of failure consisted of a break in all the columns of the first level, see Figure 8, by the formation of plastic hinges. A building like the LRMC was analyzed by Rojas Agüero [20] using the software Portal de Porticos, available at that time at the National Center for Scientific Calculation (CECALCULA) of the University of Los Andes (ULA-Venezuela).

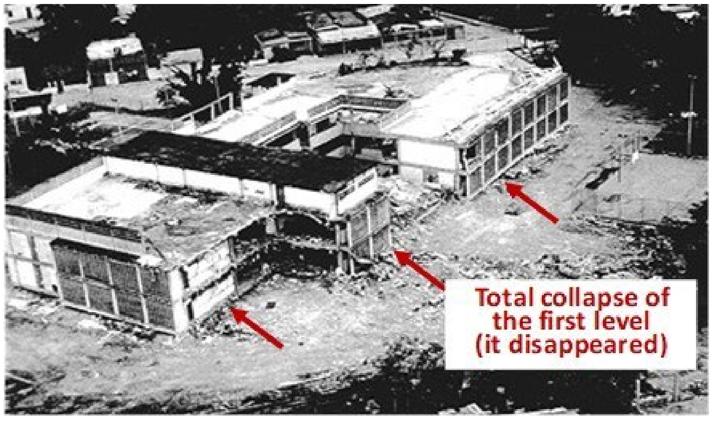

Figure 8. Collapse of the “Liceo Raimundo Martinez Centeno”.

The Portal de Porticos software is based on the theory of concentrated damage developed by Flores López [21]. Figure 9 shows the formation of plastic hinges in the building's transverse frame, and the damage levels (d) are specified, ranging from the minimum value equal to zero to the maximum value equal to unity (de 0 to 1).

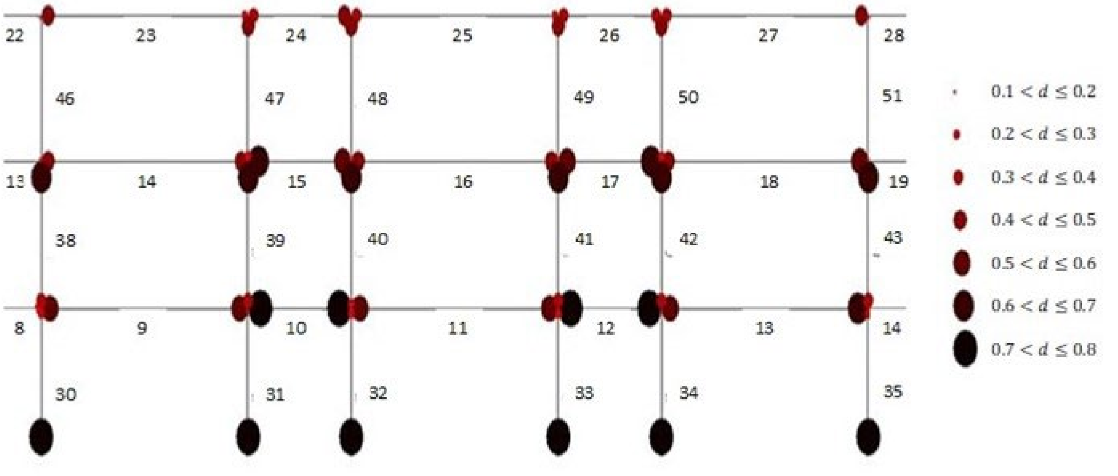

Figure 9. Example of plastic hinges in the LRMC transverse frame.

The similarity of the buildings allows for coherent comparisons to be made between them. In this sense, it is possible to have a prior idea of the vulnerability of the ESDM construction system, and therefore, we can think that the collapse of the building subjected to the maximum probable earthquake is a plausible event. On the one hand, we have the seismic vulnerability of this type of construction, and on the other, we have this building type built repeatedly in areas with high seismicity. Therefore, it is important to study the cases and propose structural retrofitting alternatives to mitigate the likely damages.

2.1.1 Characterization of the building and surroundings

Elementary education works in the building, with an enrollment of 450 students. The first level has the administrative area, library, bathroom, and six classrooms, while the upper level has nine classrooms, the music room, and the bathroom. Doors and windows constantly lock, pipes break, and cracks in the walls are repaired repeatedly. The school grounds do not have an adequate rainwater drainage system.

2.1.2 Characterization of the structure

It was carried out by visual inspection and by reviewing existing information. The building has a typical reinforced concrete frame structure with two levels (first and second floor) in which the frames are differentiated according to the support of the loads; see Figure 10.

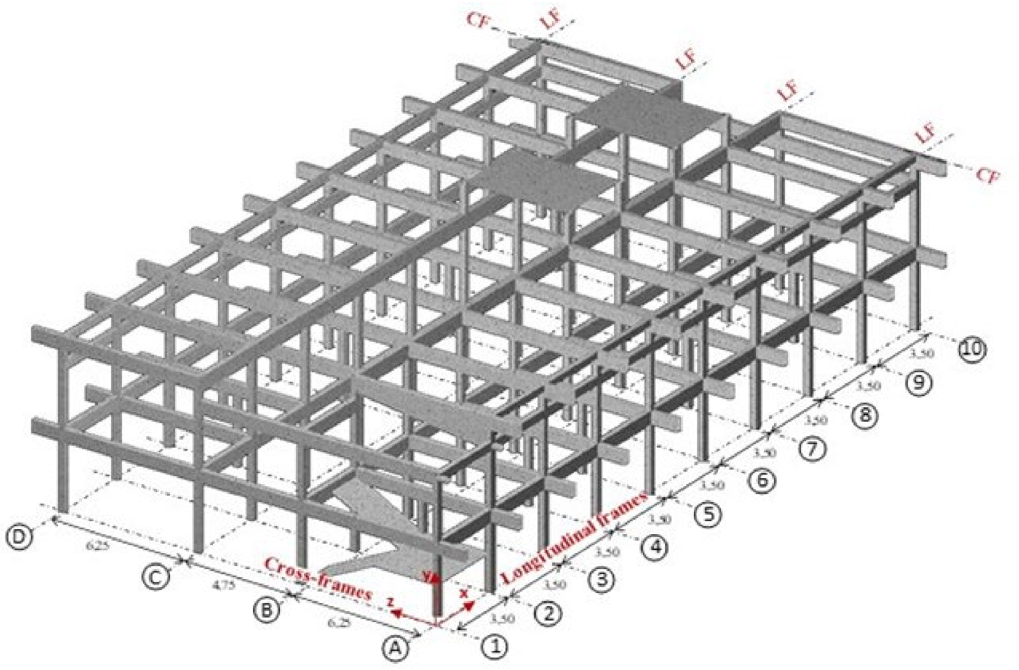

Figure 10. Building Structural System.

"Longitudinal Frames or LF" supports only seismic loads, and "Cross Frames or CF" supports both vertical and seismic loads. Also, the system has a one-way ribbed slab (with beams) system, 20cm thick. The first floor has a height of 3.57m, with beams (25cm x 57cm) in the longitudinal and transverse directions. The second level has a variable height, from 3.88m to 4.55m in the extreme spans. At that level, on axes A and D, the beam section is 25cm x 60cm, and in axes B and C, the section is 25cm x 70cm.

In addition, on the A and D axes, there are auxiliary beams with a cross-section of 25cm x 40cm, located approximately 1.00m below the top beam. Laterally the transversal frames have cantilevered beams, 1.38m long and a cross-section that varies from 25cmx57cm to 25cmx30cm at the outer end. The building has, on the roof, two reservoirs measuring 4.75m x 3.50m x 1m between the axes 6B, 6C, 7B, 7C and 8B, 8C, 9B, 9C. The floor covering is granite with 3cm thick. It also has a 1.5cm coating on slabs and walls. The roof slab is waterproofed, with a layer of concrete on the central part to allow rainwater to flow out.

The review of existing plans and the in-site inspection allowed note several aspects of the architectural design that increasing the seismic vulnerability of the building. The four longitudinal axes (A, B, C, D) and the little columns supporting the reservoirs have a configuration of short columns. The transverse frames have the strong-beam/weak-column configuration: high beams supported by columns with a small cross-section. The reservoirs are part of the structural system, which is not a good aseismic practice, mainly due to the possibility of presenting the inverted pendulum effect in case of a critical seismic event. The stairway is also part of the structure, located in the direct corner of the building between axes C (1-2) and D (1-2). This element displaces the center of rigidity of the structure away from the center of mass and produces more significant torsional effects.

2.2 Step 2: Previous Studies

As it is an ancient building, the project documents were incomplete. Therefore, without the descriptive memorial, with few plans and primarily illegible, a planialtimetric survey of the entire building in its current state was carried out.

2.2.1 Characterization of pathologies

After carrying out the planialtimetric survey and drawing the building plans construction, the digital record of the faults was carried out. These pathological signs consist of cracks in the walls, slabs, beams, and columns, both on the first and second levels. An example of the work carried out is shown in Figure 11.

2.2.2 Characterization of materials

Information on the quality of materials was not found; therefore, destructive tests on the columns were carried out. Twelve specimens using core drills were extracted to determine the concrete's strength and the material's carbonation level. Six different columns were chosen for the ground level and six for the upper one. Before carrying out the tests, electromagnetic detection was used to locate the reinforcement and avoid its cut.

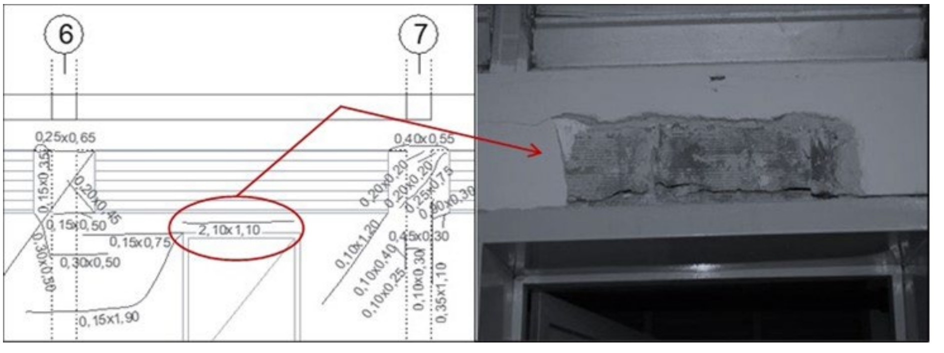

Figure 11. Example of the registration of cracks.

The uniaxial compressive strength tests were performed by the Laboratorio de Suelos y Materiales of the Universidad Centroccidental Lisandro Alvarado (UCLA-Venezuela), summarized in Table 1, using a universal press machine brand RIEHLE, model K-100.

Table 1. Compressive strength (fc) test results on columns.

Identification	d;h	Load (kg)	Stress (kg/cm^2)	CF	fc (kg/cm^2)
Column 8C-CE-FL	d= 7.55 cm; h= 11.93 cm	3540	79	0.9664	76
Column 7B-SF-FL	d= 7.55 cm; h= 11.93 cm	3310	74	0.9920	73
Column 1D-SF-FL	d= 7.55 cm; h= 11.93 cm	2880	64	0.9528	61
Column 5D-NF-SL	d= 7.55 cm; h= 11.93 cm	8980	201	0.9720	195
Column 3B-SF-SL	d= 7.55 cm; h= 11.93 cm	7960	178	0.9712	173
Column 9C-EF-SL	d= 7.55 cm; h= 11.93 cm	8720	195	0.9516	185

Subtitles: NF: north face, SF: south face, EF: east face, FL: first level, SL: second level, d: concrete cylinder diameter, h: cylinder height, A = 44.77 cm^2, Stress=Load/A, CF: Correlation factor, fc=compressive strength.

The carbonation tests were carried out by the Chemistry Laboratory of the Universidad Centroccidental Lisandro Alvarado (UCLA-Venezuela) on six columns. In columns 10A and 3B of the first floor, the depth exceeded the reinforcement coverage. Also, the determination of both the quality and type of the reinforcements was made by exploration. In this sense, scarifications were carried out on columns and beams to verify the diameter and type of steel.

2.3 Step 3: Diagnostic

The modeling of the real configuration of the building in its original state was carried out using the educational version of the finite element software Ram Element System [22]. It was built a digital twin to evaluate the structural behavior on the acting loads and later to obtain both the values of the internal forces in the elements and the verification of the lateral displacements. All the information compiled, and the experimental tests carried out in situ allowed us to define the main parameters to be considered in the numerical simulation.

The compressive strength was defined according to the experimental tests in situ results indicated above in Table 1. For the untested elements, the average strength per level was used. The yield stress was defined according to research by López et al. [23], who recommend 2400 kg/cm^2 for longitudinal reinforcement and 2800 kg/cm^2 for transverse reinforcement. Three-dimensional dynamic analysis with three degrees of freedom per level was performed, considering a rigid diaphragm. According to each load combination, the elements were analyzed for the envelopment of the acting requests, considering vertical and seismic actions, following the normative stipulations COVENIN, 1756-01 [13] and COVENIN 1753-06 [15]. The parameters considered are described below.

(i) For the seismic actions, a design spectrum based on the type S2 soil profile according to the school's location was used.

(ii) As it is a school building, it is classified as Type A with an Importance Factor of 1.3.

(iii) Seismic Zone is 5 with terrain acceleration Ao=0.30.

(iv) By the date of construction of the building, the Level of Detail is ND1; therefore, the Reduction Factor R used was equal to 2.

A one-way ribbed slab (with beams) system was modeled, whose reactions were introduced in the model as loads distributed over the load beams, including the weight of the existing walls, both in CF and LF. The two reservoirs were considered as permanent loads acting on the columns on which they rested and were also considered in the seismic mass. The quantities of longitudinal and transverse reinforcement of the columns were determined for the load combinations' envelope and were later compared with the information specified in the plans, see Table 2.

Table 2: Steel areas (As) on columns.

Column Type	Axes	As (cm^2)		Column Type	Axes	As (cm^2)		Column Type	Axes	As (cm^2)	
		Required	Deficit			Required	Deficit			Required	Deficit
C1	A-1	91.65	86.57	C1	C-4	71.45	66.37	C1	A-8	126.10	114.70
C2	B-1	87.95	80.03		D-4	63.39	58.31		B-8	117.20	105.80
C1	C-1	101.60	96.52	C1	A-5	87.68	82.60		C-8	101.70	90.30
	D-1	134.70	129.62		B-5	74.26	69.18		D-8	97.91	86.51
C1	A-2	96.75	91.67		C-5	74.65	69.57	C3	A-9	135.20	130.12
	B-2	88.32	83.24		D-5	73.40	68.32		B-9	123.90	118.82
	C-2	95.09	90.01	C1	A-6	106.80	95.40		C-9	111.00	105.92
	D-2	118.10	113.02		B-6	96.33	84.93		D-9	105.70	100.62
C6	A-3	116.70	105.30		C-6	81.85	70.45	C3	A-10	135.00	129.92
	B-3	99.84	88.44		D-6	78.84	67.44		B-10	134.60	126.68
	C-3	123.80	112.40	C3	A-7	112.60	107.52		C-10	120.70	112.78
C4	D-3	100.80	84.96		B-7	100.80	95.72		D-10	116.80	111.72
C1	A-4	85.43	80.35		C-7	85.84	80.76				
	B-4	78.23	73.15		D-7	81.24	76.16				

The term deficit used in Table 2 refers to what is needed for the structure to be safe. For example, for first data (column C-1 axis A-1), the required reinforcement of 91.65 cm^2 is presented, and there is a deficit of 86.57 cm^2; that is, the reinforcing bars present in the element is 5.08 cm^2.

Regarding lateral displacements, the standard establishes, in Table 10 of chapter 10, the admissible values for Drift, meaning "Drift" to the quotient between the difference in horizontal displacements between two consecutive levels and the height of the floor between these two levels, COVENIN, 1756-01 [13].

The total displacements (elastic and inelastic) are calculated as the product of the elastic displacements multiplied by 80% of the Response Reduction Factor (R) value. For example, for buildings with ND1, R=2, then displacements should be affected by 1.6. Table 3 indicates these values.

Table 3. Drift unreinforced structure.

Levels	Floor	Displacement (cm)		h (cm)	Drift (/1000)	
		on x	on z		on x	on z
1		7.31	11.36	357	20.48	31.82
	2					
2		12.90	19.22	323	17.31	24.33

2.4 Step 4: Retrofitting

After developing the steps indicated in 2.1 to 2.3 and analyzing the results obtained in these stages, it is decided to provide an alternative to adequate the columns encasing the old member in a stiff jacket, using RC Jacketing. It is a simple method that can be applied to any column cross-section to rehabilitating the structure.

A 3D modeling of the reinforced structure was made using a computer software, Ram Element System [22]. The behavior of the building in ULS was verified, following the recommendations of the norms.

The same structural diagnostic loads were used; only the own weight of the RC Jacketing was added. Same permanent load has lower coefficients in combinations that include the earthquake because, in these cases, it is considered a vertical earthquake.

An earthquake-analysis was used, including a value of 2 for the reduction factor because it was impossible to attend to the required detail level for Project Level II into RC Jacketing. In addition, the maximum value allowed to drift was limited to 7 per thousand to avoid ductility demands on structural elements greater than allowed. For this reason, it was verified that the minimum basal shear obtained in the analysis is superior to or specified in chapter 7 of the standard COVENIN, 1756-01 [13].

The columns were retrofitting with RC Jacketing, and the quantity of the longitudinal reinforcement was increased into a new structural configuration. Finally, the transverse reinforcement was calculated and distributed according to the current regulations; in Figure 12, the cross-section of the reinforced column is observed.

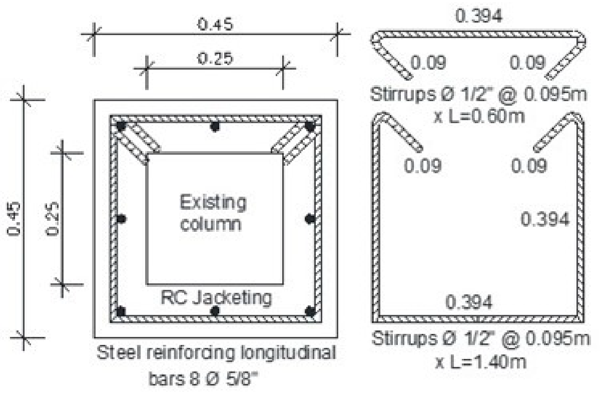

Figure 12. Column with RC Jacketing.

With the reinforcement, the strength of the structural system significantly increased, improving the overall behavior of the building under normal and seismic loads. In addition, the rigidity of the structural system was also strengthened, see Table 4.

Table 4. Drift reinforced structure.

Levels	Floor	Displacement (cm)		h (cm)	Drift (/1000)	
		on x	on z		on x	on z
1		0.80	0.90	357	2.24	2.52
	2					
2		1.34	1.77	323	1.67	2.69

2.5 Step 5: Verification

The dimensioning of the columns was verified, considering the results obtained in the previous steps. From the third step, the column without reinforcement, it is possible to obtain the value of the preload, that is, the value of the axial load to be placed on the S-Model before building the RC Jacketing one. From the fourth step, the column with reinforcement, it is possible to obtain, for each load combination, the values of the moments at the column end (M_1 and M_2) where $|M_2|>M_1$ and the value of the ultimate axial load P_u.

With M$_1$ and M$_2$ and using Equation 8, the factor C_m, is calculated, then the eccentricity e_o is calculated from the relation $e_o = M_2/P_u$. Once the values of both C_m and e_o were known, we used the S-Model software to determine the column strength stresses (M_R and P_R).

The moment amplification factor (δ) is also determined. If δ turns out to be greater than or equal to unity, the values obtained from both M_R and P_R will correspond to the maximum resistant demands for the eccentricity e_o. On the other hand, if δ turns out to be less than unity, there is no moment amplification due to the slenderness of the column, and therefore the values of M_R and P_R are calculated without considering the slenderness. The procedure is iterative; its development is explained below. It starts with arbitration of the neutral line (C_i); with this value, the diagram of unitary deformations of the column cross-section is generated, as indicated in Figure 13.

We observe that the section has four layers, which refer to (i) concrete core from the original section, (ii) cover from the original section, (iii) new concrete from Jacketing, and (iv) cover from Jacketing. In the Figure 13 we have ε_{S1} and ε_{S4}, these parameters represent the unitary strains of the longitudinal reinforcing steel in the upper and lower layers of the RC Jacketing, respectively. Additionally, we have ε_{S2} and ε_{S3}; these parameters represent the unitary strains of the longitudinal reinforcing steel in the upper and lower layers of the original section, respectively. The variables (F$_1$, F$_2$, F$_3$, F$_4$) represent the forces in each layer, and F$_C$ is the compression concrete resultant force. Figure 14 shows the verification algorithm.

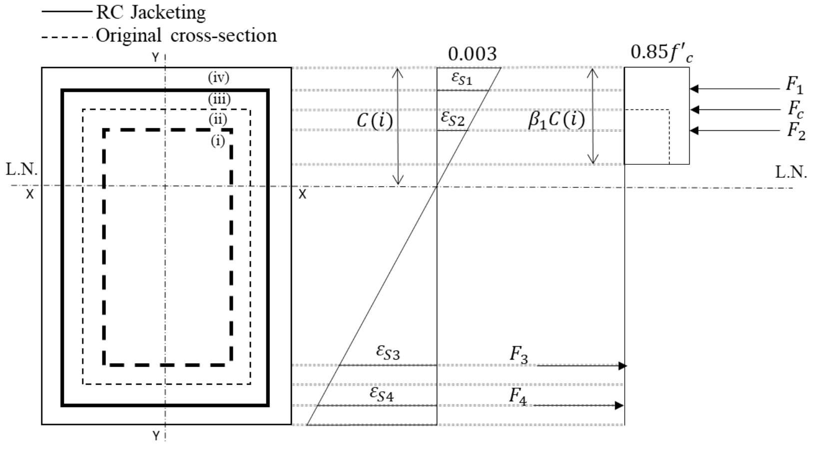

Figure 13. Diagram of deformations and internal forces in the section with RC Jacketing.

The nominal strength moment of section $M_{máxcj}$ is determined by the sum of the moments of each internal force concerning the centroid of the column cross-section. With this, the corresponding eccentricity can be calculated using Equation 16. Of all possible combinations of the pair (P_{cj}, $M_{máxcj}$), the solution is the one those eccentricity of section e_i meets the following equality in Equation 17.

$$e_i = M_{mÁxcj} / P_{cj} \tag{16}$$

$$e_i = e_o + \Delta_{cj} \tag{17}$$

The value of e_o corresponds to the eccentricity with which P_{cj} is applied. When the difference between the values of e_i, calculated according to Equations 17 and 18, is smaller than the arbitrated tolerance, the problem converges, and the strength value of the concrete section is P_{cj}. The resulting moment amplification factor in Equation 19 looks like this:

$$\delta = \frac{C_{m_{cj}}}{\left(1 - \frac{P_{cj}}{P_{Ccj}}\right)} \tag{19}$$

It is important to emphasize that in the case of $C_{m_{cj}} = 1$, the moment amplification factor will always be greater than unity, regardless of the slenderness of the column. Therefore, the value of P_{cj} found according to the described methodology coincides with the axial capacity maximum for the conditions provided. For the cases where $0{,}4 \leq C_{m_{cj}} \leq 1$, the moment amplification factor can be smaller than unity even in slender columns, which means that the largest moment acting in the column coincides with the largest moment M_2 that it acts on one end of the column and therefore the capacity of the column is calculated without taking second-order effects into account.

In the verification, the 2B and 2C columns of the first level were used as an example to show the application of the S-Model software. In the analysis, the seventeen load combinations were used to evaluate the element's behavior and verify its capacity to resist the acting loads. These combinations can be visualized in the results graphs.

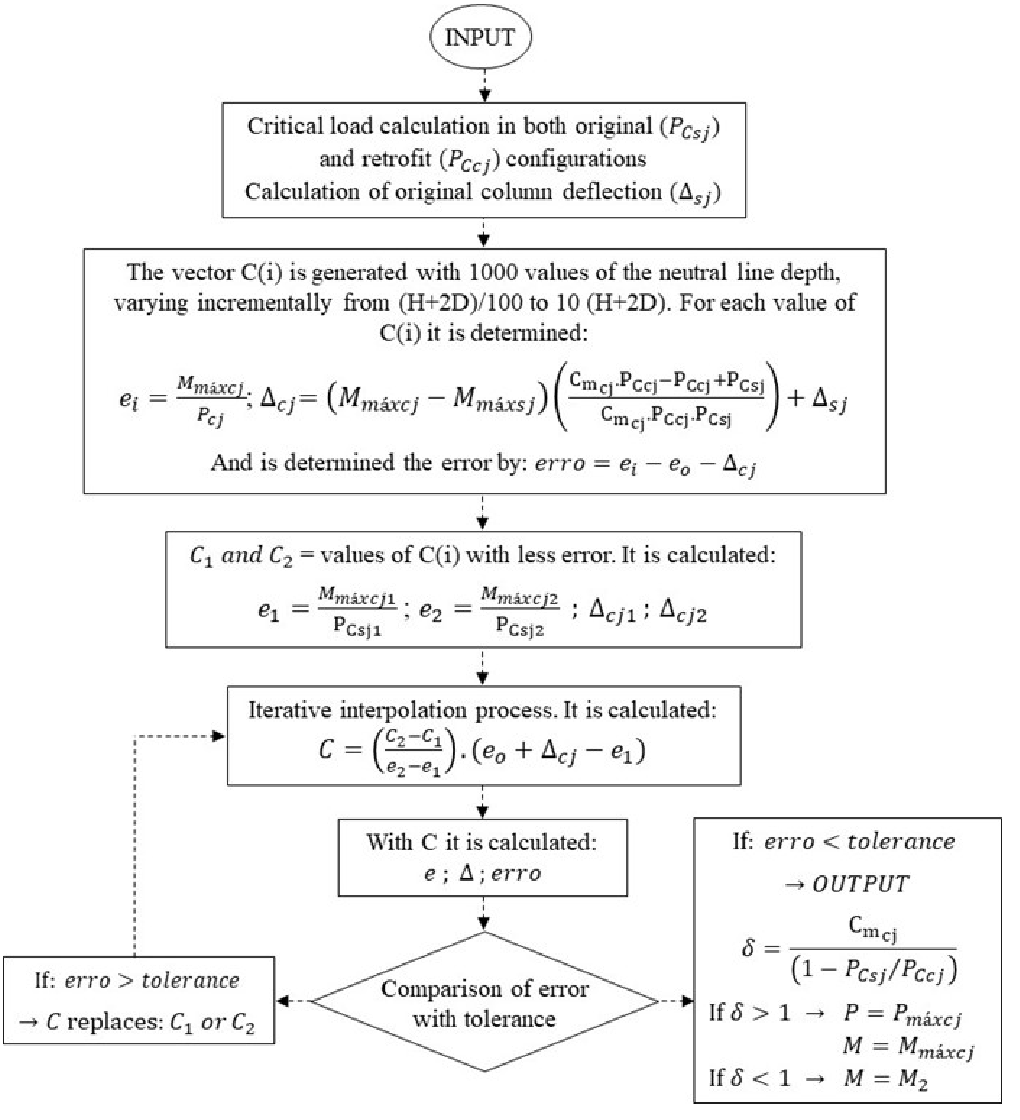

Figure 14. Algorithm for verification.

3 RESULTS AND DISCUSSIONS

A methodology was applied to evaluate RC Jacketing as a reinforcement technique in the columns of a building located in a seismic risk zone. The results are presented only for the columns with moment amplification. The process consisted of applying five consecutive steps: (i) characterization of the case study; (ii) carrying out the previous studies;

(iii) carrying out the structural diagnostic; (iv) carrying out the structural retrofitting; and (v) carrying out the verification.

The results of the first step showed that the ESDM exceeded the useful life for which it was designed. However, this happened because it was built after the last severe earthquake that affected the region, and to date, no significant seismic event has taken place. It is important to highlight that in the western zone where the building is located, return periods have been estimated between 45 and 70 years for earthquakes with a magnitude close to the one that caused the collapse of the LRMC in Cariaco.

The results of the second step allowed us to verify a series of qualitative aspects that certify the vulnerable condition of the ESDM: (i) compressive strength values are lower than indicated in the project; see Table 1; (ii) carbonation levels that exceed the 2.5 cm coverage of the reinforcement, with the possible loss of passivity in the steel; (iii) for current standard, it was observed inadequate distribution of longitudinal and transverse reinforcement all columns; (iv) it was observed pathological manifestations in the building, mainly fissures; and (v) it was observed a seismically deficient architectural archetype, mainly due to the presence of short columns.

The results of the third step allowed us to verify a series of quantitative aspects that certify the vulnerable condition of the ESDM, to current standard: (i) significant deficiencies in reinforced steel quantities, shown in Table 2; (ii) drift values above the limits established, see Table 3; and (iii) deficiencies in the ability of elements to support acting requests.

The results of the fourth step allowed evaluating the structural retrofitting proposal: (i) the drift values were checking according to the standard specifications; see Table 4; (ii) the rigidity of the building was increasing using RC Jacketing on beams and columns; and (iii) in the columns, the cross-section was increased by 55%, generating values of stiffness 12 times greater.

Jacketing layers less than 10cm wide could be more efficient analytically but constructively raises costs. This is because of the amount of longitudinal and transversal reinforcement in the section, interfering with the vibration of the mixture in small spaces and decreasing efficiency. In addition, in these cases, the concrete to be used must be special, with greater fluidity without affecting its strength, which increases costs due to the use of superplasticizers.

The results of the fifth step made it possible to plot the interaction curves of the columns and to compare the demand-capacity relationship. It was used S-Model software to analyze 2B and 2C columns in both directions, which are presented in the following graphs.

Figure 15 shows the results obtained for Column 2B, submitted to flexion compression demands around the x-axis.

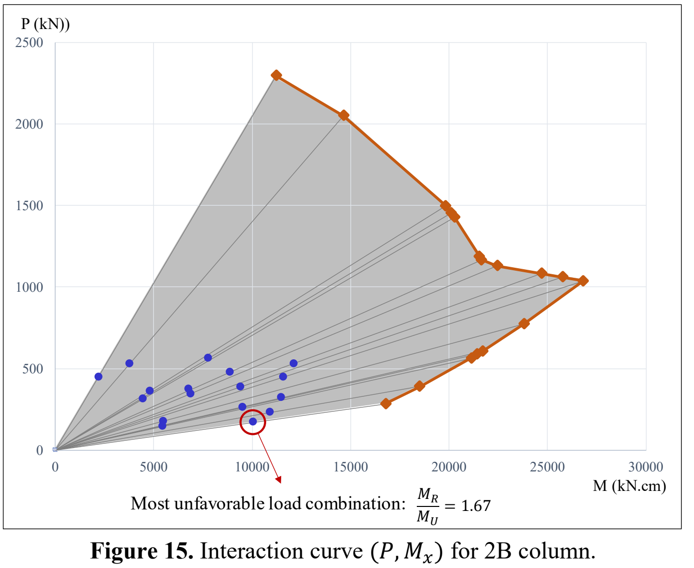

Figure 15. Interaction curve (P, M_x) for 2B column.

We note the ultimate load combinations (P_u, M_{ux}) at the blue color points and the resistant demands for the eccentricity e_o of each combination at the points (P_R, M_{Rx}) under the orange color interaction curve.

For the given combinations, the relationship between the resistant moment M_R and the most unfavorable acting ultimate moment M_U was $M_R/M_U = 1.67$; we can also observe that in none of the combinations the strength was affected by the effects of the slenderness of the column. Figure 16 shows the results in the other direction. We note the ultimate load combinations (P_u, M_{uy}) at the blue color points and the resistant demands for the eccentricity e_o of each combination at the points (P_R, M_{Ry}) under the orange color interaction curve.

The strength was affected by the effects of second-order to one of the combinations. The reduction factor was equal to 1.14 because of the effects of slenderness. However, this was not the governing combination in the design since the relation M_R/M_U was equal to 7 while the ratio M_R/M_U was equal to 1.29 to the worst case.

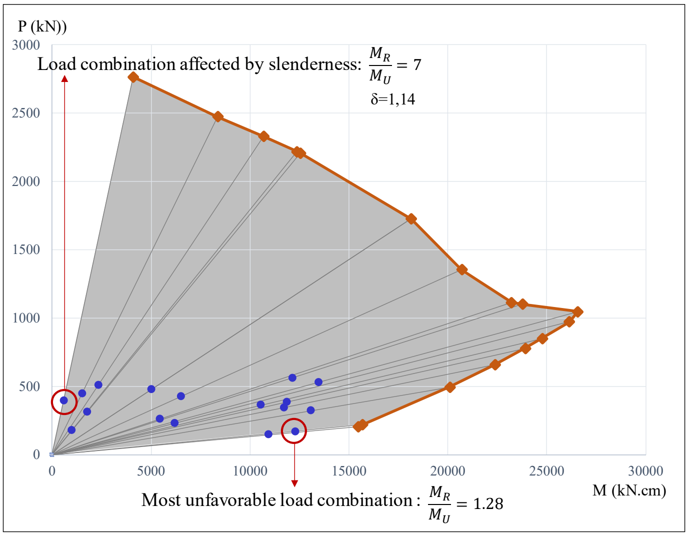

Figure 16. Interaction curve (P, M_y) for 2B column.

Figure 17 shows the results obtained for the 2C column, submitted to flexion compression demands around the x-axis. We note the ultimate load combinations (P_u, M_{ux}) at the blue color points and the resistant demands for the eccentricity e_o of each combination at the points (P_R, M_{Rx}) under the orange color interaction curve.

The strength was affected by the effects of second-order to one of the combinations. The reduction factor was equal to 1.31 because of the effects of slenderness. However, this was not the governing combination in the design since the relation M_R/M_U was equal to 8.37 while the ratio M_R/M_U was equal to 1.13 to the worst case.

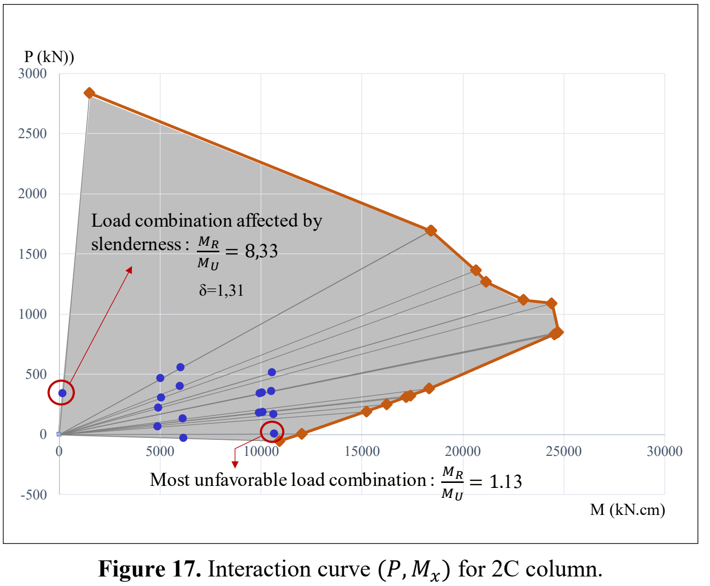

Figure 17. Interaction curve (P, M_x) for 2C column.

Figure 18 shows the results in the other direction. Again, we note the ultimate load combinations (P_u, M_{uy}) at the blue color points and the resistant demands for the eccentricity e_o of each combination at the points (P_R, M_{Ry}) under the orange color interaction curve.

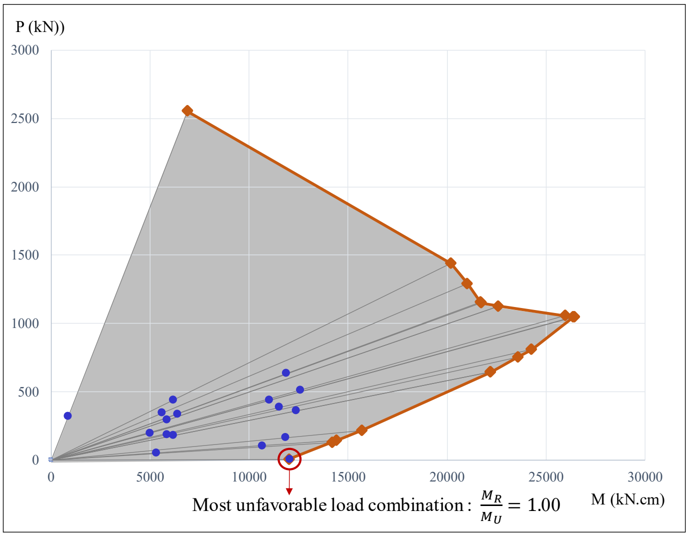

Figure 18. Interaction curve (P, M_y) for 2C column.

The strength was not affected by second-order effects to one of the combinations. The relation M_R/M_U was equal to 1 to the governing combination in the design.

Reviewing the interaction curves indicated above, we noticed that in only two cases, the strength was affected by the slenderness of the column. It is because the dominant load in the design of the columns is the earthquake. The earthquake generates marked bending in double curvature on the columns, verified by observing the dominant relationship M_R/M_U, which corresponds, in all cases, to large eccentricities (higher values of M_U and lower values of P_u).

Additionally, in the two cases where there was the amplification of moments, the eccentricities were more minor (lower value of M_U and higher value of P_u), which did not include the earthquake. We can infer that the effects of column slenderness will be more accentuated and essential in buildings where horizontal loads (earthquake, wind) are not predominant, in other words, where the variable loads are essential, and their location can generate bending in simple curvature of some columns.

The requesting moments of columns 2B and 2C in the y-y and x-x directions, respectively, were increased by slenderness effects. It is evidenced by obtaining amplification factor values greater than unity. However, the combination related to these effects in both cases was not the most critical. Therefore, it is possible to verify that the dimensioning of the columns made with the results obtained using the commercial software was adequate to simulate the case study. In situations where moment amplification occurs in the load combination that governs column behavior, the design takes the risk of being underestimated. In these situations, the quantities of column reinforcement steel calculated with any commercial software will be lower than those calculated with S-Model.

4 CONCLUSIONS

- The applied methodology allowed us to evaluate the structural retrofitting using RC Jacketing in columns.
- The verification using the S-Model software served as a computational tool to determine the admissible strength of columns reinforced with RC Jacketing. Two unusual aspects were considered: (i) the action of the preload on the original column and (ii) the verification of the strength of the section retrofit against all load combinations, including those that may generate moment amplification in the slender columns.
- It was shown that the S-Model software allows verifying the simultaneous occurrence of both moment amplification and critical combination in the dimensioning of columns. Those two aspects will generate higher demands on the quantities of reinforcement. Also, they are not verified in most of the commercial's software used by structural designers.

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REFERENCES

[1] M. Rodriguez, M. Eeri, and R. Park, "Repair and strengthening of Reinforced Concrete Buildings for Seismic Resistance," *Earthq. Spectra*, vol. 7, no. 3, 1991, http://dx.doi.org/10.1193/1.1585636.

[2] J. R. Yepez Aguirre, “Comportamento de pilares de concreto armado reforçados mediante o incremento da sua seção transversal,” Ph.D. thesis, Porto Alegre, Depto. Eng. Civ. UFRGS, POÁ, Brasil. 2019. Accessed: Sep. 26, 2021. [Online]. Available: https://lume.ufrgs.br/handle/10183/204074

[3] U. Ersoy, T. Tankut, and R. Suleiman, "Behavior of jacketed columns," *ACI Struct. J.*, vol. 90, no. 3, pp. 288–293, 1993. Accessed: Sep. 26, 2021. [Online]. Available: http://worldcat.org/oclc/13846957

[4] E. N. B. S. Júlio and F. A. B. Branco, "Reinforced concrete jacketing-interface influence on cyclic loading response," *ACI Struct. J.*, vol. 105, no. 4, 2008. Accessed: Sep. 26, 2021. [Online]. Available: https://www.researchgate.net/publication/268223250

[5] P. Krainskyi, Z. Blikharskiy, and R. Khmil, “Experimental investigation of reinforced concrete columns strengthened by jacketing”, *J. Multidiscip. Eng. Sci. Technol.*, vol. 2, no. 7, 2015. Accessed: Sep. 26, 2021. [Online]. Available:http://www.jmest.org/wp-content/uploads/JMESTN42350952.pdf

[6] E. Golias, A. Zapris, V. Kytinou, G. Kalogeropoulos, C. Chalioris, and C. Karayannis, "Effectiveness of the novel rehabilitation method of seismically damaged RC joints using C-FRP ropes and comparison with widely applied method using C-FRP sheets. Experimental investigation," *Sustainability*, vol. 13, 2021, http://dx.doi.org/10.3390/su13116454.

[7] G. Kalogeropoulos, A. Tsonos, D. Konstandinidis, and S. Tsetines, "Pre-earthquake and post-earthquake retrofitting of poorly detailed exterior RC beam-to-column joints," *Eng. Struct.*, vol. 109, pp. 1–15, 2016, http://dx.doi.org/10.1016/j.engstruct.2015.11.009.

[8] E. Golias et al., "Application of X-shaped CFRP ropes for structural upgrading of reinforced concrete beam-column joints under cyclic loading. Experimental study," *Fibers (Basel)*, vol. 9, no. 42, 2021, http://dx.doi.org/10.3390/fib9070042.

[9] F. E. Audemard and F. A. Audemard, “Structure of the Mérida Andes, Venezuela: relations with the South America-Caribbean geodynamic interaction”. *J. Tectonophysics*, vol. 345, pp. 299-327, 2002, https://doi.org/10.1016/S0040-1951(01)00218-9.

[10] J. J. Hernández and M. Schimitz, “Amenaza sísmica en la región de los andes venezolanos”. *Rev. Fac. Ing. Univ. Cral. Ven. (UCV)*, vol. 34, no. 2, 2019. Accessed: Sep. 26, 2021. [Online]. Available: http://saber.ucv.ve/ojs/index.php/rev_fiucv/article/view/19299

[11] R. Esteves and J. Laffaille, “Sismicidad y prevención sísmica en los andes venezolanos”. Available: http://www.funvisis.gob.ve/old/archivos/www/terremoto/Papers/Doc029/doc029.htm (Accessed: Sep. 26, 2021).

[12] A. Tsonos and G. Kalogeropoulos, "Analytical model for the design of HSFC and UHSFC Jackets with various steel fiber volume fraction ratios for the retrofitting of RC beam-column joints," *Sustainability*, vol. 13, 2021, http://dx.doi.org/10.3390/su132011209.

[13] Comisión Venezolana de Normas Industriales, *Edificaciones Sismorresistentes*, COVENIN 1756:2001, 2001.

[14] Comisión Venezolana de Normas Industriales, *Criterios y Acciones Mínimas Para el Proyecto de Edificaciones*, COVENIN 2002:1988, 1988.

[15] Comisión Venezolana de Normas Industriales, *Proyecto y Construcción de Obras en Concreto Estructural.* COVENIN 1753:2006, 2006.

[16] American Concrete Institute, *Building Code Requirements for Structural Concrete*, ACI 318-14, 2014.

[17] A. Nilson, *Diseño de Estructuras de Concreto*. 12th ed. McGraw Hill, Colombia, 1999.

[18] S. P. Timoshenko and J. M. Gere, *Theory of Elastic Stability.* 3th ed., McGraw Hill, New York, 1969.

[19] B. G. Johnson, *Guide to Stability Design Criteria for Metal Structures*. 3rd ed., John Wiley and Sons, New York, 1976.

[20] R. Rojas Agüero, “Procedimiento para el diseño y validación de la rehabilitación de estructuras vulnerables de concreto armado”, M.S. thesis, Depto. Estruc. ULA, Mérida, Venezuela, 2007. [Online]. Available: http://bdigital.ula.ve/

[21] J. Flores López, “Modelos de daño concentrado para la simulación del colapso de pórticos planos”. *Rev. Intl. Mtd. Num. Cál. Dis. Ing.*, vol. 9, p. 143, 1993. Accessed: Sep. 26, 2021. [Online]. Available: http://hdl.handle.net/2099/8836

[22] Ram Element System. “Software de Análise e projeto 3D estrutural. Versão educativa 16.3.1.8”. Bentley Systems Europe, 2021.

[23] O. A. López, J. J. Hernández, G. del Re, and J. Puig, “Reducción del Riesgo Sísmico en Escuelas de Venezuela”. *IMME*, vol. 42, no. 3, pp. 33-56, 2004.

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