Evaluation of the Influence of the Level of Corrosion of the Reinforcing Steel in the Moment-Curvature Diagrams of Rectangular Concrete Columns

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Abstract — In the present work, it is proposed to include in the theoretical curves of moment-curvature ($m-\phi$) the effect of corrosion by obtaining the decrease in the area of longitudinal reinforcing steel. The corrosion depth will be obtained from the crack width and corrosion length observed in the cover concrete. With the depth of corrosion, the area of steel that is lost will be obtained and this modification will be incorporated into the theoretical procedure to elaborate the $m-\phi$ curves. The forces of the steel will be obtained from an elastoplastic model with curved hardening and the forces of the concrete with a model that considers the effect of confinement.

Index Terms — Moment- Curvature; Corrosion; Crack; Column resistance.

I. INTRODUCTION

The modelling procedures of the plastic hinge of the elements of the reinforced concrete structures consider hypotheses of the behavior of the materials different from the hypotheses used for the design of the element, one of these hypotheses is based on modelling the stress-Strain curves of materials considering the effects of confinement and material deterioration, but current regulations only consider concrete deterioration due to cyclical loads and with these hypotheses, the moment-curvature diagrams that define the plastic hinge are constructed. Although these hypotheses can be used in cases where it is desired to evaluate a concrete structure or perform a non-linear analysis, they do not reflect the effect of the deterioration that the structure element may have due to carbonation of the concrete, due to loss of the section, due to cracking or corrosion of the steel of the reinforcing steel, which is the main cause of the deterioration of reinforced concrete structures, among the main aggressive agents are chloride ions, which according to the literature are considered the most aggressive [1]-[12], the sulfate ions are also considered as aggressive agents, which are present in wastewater, contaminated soils, groundwater, or industrial environments, so their effect on the corrosion of reinforcing steel and object has been of great interest. of diverse

investigations with diverse proposals to mitigate the corrosion by sulphates [13]-[24], whose maintenance due to corrosion problems is billions of dollars in the world [25]-[35], vey particularly in bridges of concrete due to the fact that they are reinforced concrete works of great repair cost and that the vast majority are in aggressive environments [36]-[46]. As mentioned in the previous paragraphs, to mitigate corrosion damage, various research works have been carried out, taking into account the quality of concrete, type of cement, additives, alternative materials to Portland cement, simulating aggressive contact media such as sulfates and chlorides, but most research is focused on mitigating the corrosion of reinforcing steel, and little information is available on the effect of corrosion or the level of corrosion present in important structures such as bridges and its effect on resistance to a seismic event. when they are already damaged by corrosion.

Therefore, the importance of the present work is considered, where the effect that the corrosion of the reinforcing steel has on the stress - strain graph of the concrete confined in rectangular columns was evaluated. The construction of the stress-strain curve of the concrete is done following the proposal of Mander [47], the stress-strain curve of steel with the values proposed by Mendoza [48] and in a first proposal, considering the loss of steel due to corrosion according to the criteria of Castonera [49].

II. MATERIALS AND METHODS

A. Materials

Normally the strengths of the materials, steel, and concrete, which are considered in a new project are conservative, for the modelling of the plastic hinge using the moment – curvature diagrams values that reflect the real resistance of the material must be adopted. For this, in this work it is proposed to consider the forces of steel with an elastoplastic model with curved hardening and for the forces of concrete with a model that considers the effect of confinement.

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1) Strength of steel

The stress-strain curve of steel will be modeled with the parameters proposed by Mendoza [48] and that incorporate the Complementary Technical Standards for Design and Construction of Concrete Structures (NTC, 2017) [50] in Table I.

TABLE I. PARAMETERS TO MODEL THE STRESS-STRAIN GRAPH OF THE

Parameters	REBAR	
	Grado 42	
	MPa	Kg/cm ²
f_y	457	4487
f_{su}	742	7280
f_{sh1}	612	6000
E_s	209218	2052433
ϵ_y	0.0022	
ϵ_{sh}	0.0066	
ϵ_{su}	0.1100	
ϵ_{sh1}	0.0248	
F	4.03

Fig. 1. Stress-Strain model of steel Gr. 42 incorporated in NTC (2017) [50].

2) Strength of concrete

The strength of the concrete considers the effect of the confinement provided by the transverse steel, for which the stress-strain curve is constructed in accordance with Fig. 2 [47], and which is a theoretical model accepted in current standards [50].

Fig. 2. Stress-Strain model of confined and unconfined concrete [45].

In Fig. 2, f'_{co} is the compressive stress of simple concrete, commonly identified as f'_c , ϵ_{co} is the unit strain of

concrete when f'_{co} is reached, which is considered 0.003, f'_{cc} the maximum stress of compression considering the confinement and ϵ_{cc} is the unit strain when it reaches said confinement. For rectangular sections the maximum stress f'_{cc} is obtained from the graph in Figure 3 that involves the confinement relationships f'_{l1}/f'_{co} and f'_{l2}/f'_{co} .

Fig. 3. Confined Strength determination from lateral confining stresses for rectangular sections [47].

f'_{l1} and f'_{l2} are the lowest and highest confining stress respectively in the X or Y direction obtained with (1) that depend on the percentages of longitudinal steel $\rho_{(x \text{ ó } y)}$ and an effective confinement factor k_e .

$$f'_{l(x \text{ ó } y)} = k_e \rho_{(x \text{ ó } y)} f_{yh} \quad (1)$$

k_e in turn, depends on the free spacing between the longitudinal bars w'_i , free spacing between stirrups s' , distance perpendicular to the direction x b_c , distance perpendicular to the direction y d_c and percentage of longitudinal steel ρ_{cc} . See Fig. 4.

Fig. 4. Effectively confined core for rectangular hoop reinforcement [47].

To obtain the confinement effort f'_{cc} , in a less visual way than when using the graph in Fig. 3 and allowing us to use a programmable spreadsheet, programmable equations were obtained which replace Fig. 3. Once the relations f'_{l1}/f'_{co} and f'_{l2}/f'_{co} two of the equations obtained are chosen and with the two results it is linearly interpolated to finally obtain the relation f'_{cc}/f'_{co} .

B. Method

1) Ultimate deformation of concrete

Some authors [51] recommend that the value adopted as the ultimate concrete deformation ε_{cu} be 0.005 for the case of poorly confined concrete. For the case of well-confined concrete, the ultimate deformation can be obtained with (2) [52] derived from Mander's energy balance criterion, it also includes the deformation energy stored in the longitudinal reinforcement.

$$\varepsilon_{cu} = \frac{110\rho_s + 3.4\rho_l + 0.017\gamma\sqrt{f'_c}}{\rho_{cc}(0.94f_y + 302) - \gamma(0.015f'_c - 1.1f'_c - 8)} (f'_c \text{ y } f_y \text{ en MPa}) \quad (2)$$

$$\gamma = 0.45 + 0.5 \sqrt{\frac{P}{f'_c A_g}} \quad (3)$$

In (3) P is the axial force and A_g is the thick section and the expression is only applicable for rectangular sections.

Fig. 5 shows the stress-strain curves for different concrete f'_c following the criteria established in section 2 of this document. The curves are specific for the section of the column of the same figure, which is reinforced with 8 rebars of the number 8, hoop of the number 3 rod with a separation of 10 cm.

Fig. 5. Stress-Strain curves for the left column.

2) Obtaining the moment-curvature diagrams (M-φ)

Using our own code developed in the Visual Basic editor for Excel applications (VBA), the Moment - Curvature curves were calculated following the procedure described by some authors [53]-[54], with the variations that the ultimate deformation of the concrete will not be 0.005 as proposed, if

not obtained with (2). The compressive strength of concrete C_c is calculated with (4), where b is the base of the column section and f'_{cc} is calculated with Mander's criterion.

$$C_c = \alpha f'_{cc} b k d \quad (4)$$

α is a mean stress factor [54] can be calculated with (5), where the numerator represents the area under the concrete stress-strain curve, which in the VBA code is performed by increments of strain and γ , is the centroid factor for any strain ε_{cm} .

$$\alpha = \frac{\int_0^{\varepsilon_{cm}} f_c d\varepsilon_c}{f'_c \varepsilon_{cm}} \quad (5)$$

$$\gamma = 1 - \frac{\int_0^{\varepsilon_{cm}} \varepsilon_c f_c d\varepsilon_c}{\varepsilon_{cm} \int_0^{\varepsilon_{cm}} f_c d\varepsilon_c} \quad (6)$$

Following the procedure described, the diagram of Fig. 7 was obtained, for a value of $f'_c = 250 \text{ kg/cm}^2$, the ultimate deformation ε_{cu} calculated with formula (2) is 0.02145.

Fig. 7. Moment-Curvature curve for the column of Fig. 5.

3) Corrosion damage

The theoretical curves of moment - curvature (M-φ) are carried out under the hypotheses already established above but also under the assumption that the steel of the column is in optimal conditions, where it does not present cracking due to corrosion or decrease in the area of longitudinal steel due to corrosion. When it is desired to evaluate a concrete structure, the conservation conditions of the structure may not be optimal, the specific case of the columns of concrete bridges that are located in coastal cities or road sections near the sea show corrosion damage, due to which when making an evaluation in which it is necessary to have the M-φ diagram, would not be representing the real conditions of the concrete element.

Castorena [49] proposes a model to estimate the depth of corrosion of steel rods from the width of the corrosion crack, which can be measured by field inspection. The model involves only characteristics of the element, that is, the column and the mechanical properties of the concrete.

The model proposes that the corrosion depth X_{corr} is given by the penetration on the steel at the moment of initiating the

cracking of the cover concrete by corrosion X_o plus the penetration in the steel after the first visible corrosion crack X_p has appeared. See Fig. 8.

Fig. 8. Corrosion depth [49].

$$X_{corr} = X_o + X_p \quad (7)$$

4) Penetration of corrosion at the beginning of the corrosion cover concrete cracking X_o

The experimental results [49] show that the width of the first corrosion crack varies between 0.06 and 0.08 millimeters, which is when the initial corrosion penetration X_o occurs and that we can estimate with (8). X_o depends on C which is the value of the free coating that the steel has, d the diameter of the rod and L anodic length, that is, the length of the corroded steel. These variables are shown in Fig. 9.

$$X_o^{0.15} = 0.4553 \left(\frac{C}{d}\right)^{0.04} \delta^{-0.0653} \left(\frac{C}{L}\right)^{0.07} \quad (8)$$

Fig. 9. Variables involved in the penetration of corrosion at the beginning of cracking [49].

It is recommended that the anode length can be fully identified using some appropriate electrochemical technique.

The parameter δ is a function of the radius of the rod r_o , the poisson's moduli of concrete and steel respectively ν_c and ν_a , the modulus of elasticity of steel and concrete E_a and E_c and of R_c which is equal to the sum of the radius of the rod plus the free cover concrete as indicated in Fig. 9 d). δ is obtained with (9) and represents the oxide layer necessary to produce tangential forces equal to the maximum tensile strength of the concrete and initiate cracking at the steel-concrete interface.

$$\delta = 2r_o P_g \left[\frac{1-\nu_a}{E_a} + \frac{(1+\nu_c)R_c^2 + (1-\nu_c)r_o^2}{E_c(R_c^2 - r_o^2)} \right] \quad (9)$$

P_g is the radial pressure at the steel-concrete interface and is obtained with equation (10), where f_t is the concrete tensile stress and for our study it is calculated as established by the NTC (2017) based on the type of concrete being used.

$$P_g = \frac{f_t}{\left[\frac{R_c^2 + r_o^2}{R_c^2 - r_o^2} + \nu_c \right]} \quad (10)$$

If we set the values of the corrosion length to 150 mm and the free cover of 50 mm, we can see in Figure 10 how the diameter of the rod and the concrete resistance influence the initial corrosion penetration X_o . The rods of smaller diameter and the concretes of smaller resistance are those that present higher values of length of initial corrosion and for concretes with value of f_c greater than 290 kg/cm² the stress of the concrete is no longer a variable that affects the value of X_o .

Fig. 10. Corrosion penetration at the beginning of cover concrete cracking for different values of f_c and different diameters of the steel.

5) Corrosion penetration after first visible corrosion crack appeared X_p

After the appearance of the first corrosion crack in the cover concrete, the reduction in radius continues to depend on the crack width w and can be calculated with the formula (11) proposed by Castorena [49]. In figure 11 we can see the variation of X_p for different crack widths.

$$X_p^{0.20} = -1.3565 + 1.8673w^{0.08634} \quad (11)$$

Fig. 11. Corrosion penetration after the first corrosion crack appeared.

III. RESULTS AND DISCUSSION

In research carried out [55] there is a notable difference in the crack width of the different sides of the section (front section and rear section) of specimens exposed to marine environments, this mainly due to the fact that exposure to the aggressive agents is different in the structural elements depending on their orientation, this can be seen in Fig. 12, the concrete elements present greater corrosion damage on one of the faces.

Fig. 12. Column of the Palma Sola bridge located in Coatzacoalcos, Veracruz, México.

Applying the criteria set out above, assuming a corrosion length of 100 mm, an $f_c = 250 \text{ kg/cm}^2$, different crack widths and a cross section of the dimensions of Fig. 5, with rebars of number 8, stirrups of the number 3 at a separation of 10 cm, we obtain Fig. 13.

In Fig. 13, the curve for $w = 0$ corresponds to the section of Fig. 5, considering that the steel has not suffered corrosion damage, the other three curves shown in Fig. 5 are considering that the longitudinal steel in only one of the faces already show corrosion damage and this is estimated according to the width of the crack, in this case, crack widths of 0.06 mm, 1 mm and 1.5 mm were assumed. When the first corrosion crack appears in the cover concrete of an approximate width of 0.06 mm, the depth of the corrosion is a distance of 0.05288 mm and corresponds only to X_o .

Fig. 13. Moment – Curvature curve for the rectangular column damaged by corrosion.

TABLE II: CORROSION DEPTH FOR DIFFERENT CRACK THICKNESSES FROM THE SECTION OF FIG. 5

W (mm)	X_p (mm)	X_o (mm)	X_{cor} (mm)	% Lost steel area
0.06	0.0000	0.0529	0.0529	65.97
1	0.0348	0.0529	0.0877	90.40
1.5	0.0641	0.0529	0.1170	99.38

In Fig. 13 we see how the $M-\phi$ diagram is affected by the corrosion damage that the element may present. The greatest decrease in moment occurs when the first crack appears, reducing the maximum moment by approximately 20%. We can also observe that the ultimate curvature increases as the crack width increases, despite the fact that the percentage of steel loss per affected rod is of the order of 90% (See Table II) for larger crack widths.

IV. CONCLUSIONS

In a process of evaluation of concrete structures through a non-linear analysis where the moment-curvature diagrams are required for the modelling of the plastic hinges, the theoretical approaches that represent the real conditions of the element, in terms of materials and conservation of the structure.

It is feasible to use Castorena's proposal to estimate the reduction of steel radius due to corrosion, since his proposal only requires the observation on site of the crack width, elementary data of the geometry and information and materials of the concrete section.

Including the corrosion damage of the moment-curvature diagrams describes a better behavior of the element, especially for evaluation purposes, since the element can theoretically present a decrease of 20% of the moment when the first crack appears compared to what moment it would reach if the section did not show corrosion.

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