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Durability-Based Optimal Design Methodology for RC Members in Corrosive Environment

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ABSTRACT

As durability-based design of reinforced concrete (RC) structures in corrosive environments is *highly* influenced by chloride-induced corrosion of reinforcing steel bars, this paper first presents brief outline of an experimental investigation recently carried out by the authors on a large number of reinforced concrete test specimens subjected to several *scenarios* of chloride-driven reinforcement corrosion. Concrete specimens were prepared with cementitious material content of 350, 375, and 400 kg/m^3; water–cementitious ratios of 0.4, 0.45, and 0.5; fine to total aggregate ratios of 0.35, 0.4, and 0.45; and cover thickness of 25, 37.5, and 50 mm. The specimens were then exposed to chloride solution of three different concentrations and were tested for determining corrosion-rate using electrochemical and gravimetric-weight loss methods. Numerical analysis of reinforcement-corrosion rates (determined electrochemically and gravimetrically) was first used to determine statistical correlation between corrosion rates obtained by the two methods. Then, the gravimetric reinforcement-corrosion rate results were utilized for developing regression models for reinforcement corrosion rates in terms of concrete quality parameters, concrete cover-thickness, and chloride concentration. The regression models obtained for reinforcement-corrosion-rates were adapted within an *automated* analysis-design-methodology using *Microsoft Excel* solver for durability-based optimal design of RC members subjected to specified chloride exposure corrosive environments. Sample results obtained from the design methodology outlined in this paper are summarized for selected case studies of RC beams and columns.

1. INTRODUCTION

As regards the durability of reinforced concrete structures under known chloride-exposure conditions, chloride ions-driven corrosion of reinforcing steel is known to be a major design problem for RC structures. And while quality of concrete and cover thickness play a major role in initiation and progress of reinforcement corrosion, it is particularly noted that deterioration of RC structures in a corrosive environment (taking coastal region of Saudi Arabia as an example) is mainly attributed to (i) extreme environmental conditions, (ii) substandard quality of construction materials, and/or (iii) inadequate construction practices. Environmental conditions of the area are characterized by wide variations in daily and seasonal temperatures (Saricimen, 1993). Such variations in day to night temperature lead to formation of microcracks in concrete matrix, which accelerates diffusion of aggressive species, such as chlorides, to surface of steel bars. The conditions required for initiating corrosion of reinforcing steel are satisfied, and the resulting corrosion products of confined steel bars would produce expansive forces of magnitude that may exceed by far the tensile strength of concrete matrix (Dyer, 2014). Therefore, reinforced concrete structures in such environments should be designed to satisfy both durability and strength requirement (Alghamdi & Ahmad, 2010; Shameem, Maslehuddin, Saricimen, & Al-Mana, 1995).

For carrying out structural durability-based design of RC members, the following two effects of degradations in concrete and steel should therefore be considered:

(i) loss of concrete cover leading to reduced cross-sectional area of the concrete due to surface deterioration; and

(ii) loss of steel cross-sectional area, steel-to concrete bond, and loss of concrete cover due to expansive forces resulting from reinforcement corrosion products.

For RC structures under aggressive-exposure conditions, excluding frost attack, a model for evaluating the rate of deterioration of surface concrete C_r (mm/year; being the rate of loss of structurally effective concrete), a model was previously given (Pihlajavaara, 1994) as follows in Eq. 1:

$$C_r = \frac{C_{env} C_{cur}}{f_{ck}^{3.3}} \tag{1}$$

where C_{env} = environmental coefficient; C_{cur} = curing coefficient; and f_{ck} = characteristic cubic compressive strength of concrete at 28 days (MPa).

In the Gulf region (within latitude 10°–30°), values of c_{env} were assumed to be within the range of $10–500 \times 10^3$ (Pihlajavaara, 1994), and the curing coefficient, C_{cur}, may be calculated by using the following equation (Vesikari, 1995).

$$C_{cur} = \frac{1}{0.85 + 0.17 log_{10}(d)} \quad (2)$$

where d = the curing time (days).

Loss rate of structurally effective concrete C_r, is calculated by using Eq. 1, and the reduced cross-sectional area of concrete, at any exposure time t, may be calculated by using $C'(t)$ from Eq. 3:

$$C'(t) = C_r(t) \quad (3)$$

The cross-section residual dimensions [namely, width $b'(t)$ and depth $h'(t)$] for a concrete member at any exposure time t, may be calculated using Eqs. 4 and 5:

$$b'(t) = b_o - 2c'(t) \quad (4)$$

$$h'(t) = h_o - 2c'(t) \quad (5)$$

where b_o and h_o are, respectively, initial width and depth of a given RC member.

Value of corrosion penetration rate P_r is determined using Eq. 6,

$$P_r = \frac{W}{F\gamma_{st}} I_{corr} \quad (6)$$

where $\gamma_{st} = 7.85$ g/cm^3; $F = 96500$ A · s; $W = 27.93$ g; The loss of steel-rebar diameter $d'(t)$ leading to a reduced cross-sectional area of steel at any exposure time t is calculated using Eq. 7.

$$d'(t) = P_r(t) \quad (7)$$

Then, a reduced diameter $D'(t)$ of rebar at any exposure time t is given by Eq. 8,

$$D'(t) = D_o - 2d'(t) \quad (8)$$

where D_o is the initial diameter of rebar.

The main objectives of this research work are to

(a) determine reinforcement corrosion rate using electrochemical and gravimetric techniques on a large number of concrete specimens designed with specified combinations of design variables *including* water–cementitious material ratio (w/c), cementitious material content, fine to total aggregate ratio, and cover thickness;

(b) determine correlation between electrochemically and gravimetrically measured reinforcement corrosion rates; and

(c) outline and demonstrate utilization of a methodology for durability-based design of RC members in a specified corrosive environments.

2. EXPERIMENTAL PROGRAM

2.1 Construction materials and properties

This research study is based on type I Normal Portland Cement used with 8% Silica Fume added to all concrete mixtures. All concrete test specimens were prepared with aggregates obtained from two different sources located in the eastern and western regions of the Kingdom (namely, Abu Hadriyah quarries and Taif quarries), referred to hereafter as H-type and T-type aggregates, samples of which are shown in Figure 1. The T-type aggregate is composed mainly of coarse-grained *silicate minerals* (such as quartz and feldspar) known to produce weaker bonds with cement paste than the H-type (mainly *calcareous* rocks, such as limestone and dolomite). And with the established influence of aggregates-type, based on *mineralogical* and *petrographic* characteristics (Bérubé, 2001), on durability of concrete matrix and RC structures in corrosive environments (Saricimen, 1993), the characteristics of the two types of coarse aggregates were previously studied (Alghamdi & Ahmad, 2010). Development of micro-cracking at materials' interface lines and harmful levels of concrete permeability (consequentially undermining structural durability) are *invariably* caused by particular types of coarse aggregates. As such, coarse aggregates' properties were identified as most influential on thermal and stiffness characteristics and incompatibilities between concrete-matrix constituents. And for the purpose of this research work, specific gravity and water absorption of used coarse aggregates were determined, and abrasion test results were obtained (respectively, in accordance with ASTM C128 and ASTM C131), and the results are listed in Table 1. The specific gravity and absorption of fine aggregate were found to be 2.6 and 0.57%, respectively. Super-plasticizer was added during mix design with low water–cement ratio, in order to enhance workability of concrete mixtures.

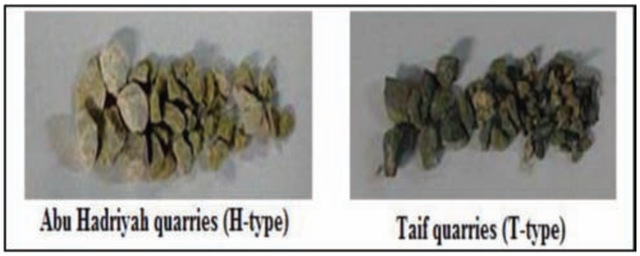

Figure 1. Samples of the two types of coarse aggregates used.

Table 1. Coarse aggregates properties.

Aggregate source	Specific gravity	Water absorption (%)	Abrasion loss (%)
Abu Hadriyah (H)	2.55	1.10	28.86
Taif (T)	2.88	2.01	37.84

2.2 Test specimens and concrete mix design

Steel bars of diameter 16 mm were centrally placed in each one of 486 cylindrical concrete test specimens with height of 150 mm; diameters of 66, 91, and 116 mm; and with three different cover thicknesses of 25, 37.5, and 50 mm. Test specimens were prepared to evaluate corrosion rate and epoxy coating was applied to steel bar at bottom-end and at interface-end extending from within top-end of concrete-matrix upwards as shown in Figure 2.

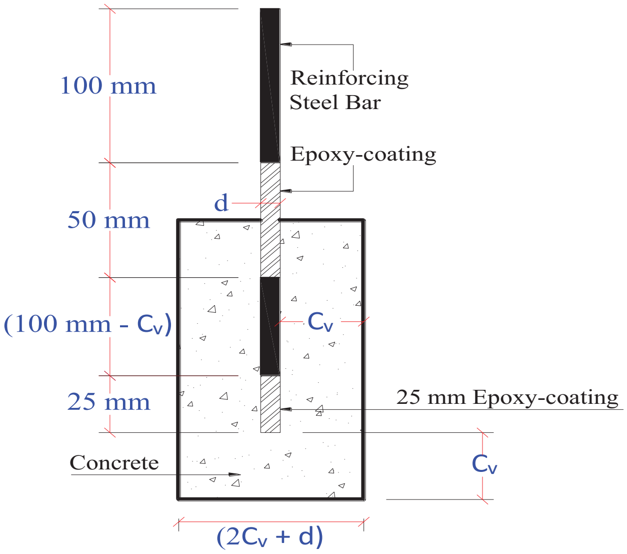

Figure 2. Details of typical test specimen.

Absolute volume method was employed for the concrete mix design. The groups of water–cementitious ratio, cementitious material content, and fine to total aggregate ratio used to prepare the specimens were *respectively* ($R_{w/c}$: 0.4, 0.45, and 0.5), (C_c: 350, 375, and 400 kg/m^3), and ($R_{F/T}$: 0.35, 0.4, and 0.45). Three chloride concentrations of 3, 7, and 12% were used to simulate corrosive conditions. And in order to obtain homogenous concrete matrix having uniform consistency without segregation, all constituents were uniformly mixed together, with the addition of potable water and super-plasticizer, using a revolving drum-type mixer.

Test specimens were de-molded after 24 h of concrete casting, and specimens were then cured for a period of 28 days in water tanks under laboratory conditions and were subsequently partly submerged in chloride solutions to allow corrosion to take place. Samples of test specimens exposed to a chloride solution are shown in Figure 3.

Figure 3. Sample of test-specimens exposed to chloride solution.

2.3 Experimental program

2.3.1 Linear polarization resistance method (LPRM)

The LPRM was employed to measure the corrosion current density (I_{corr}) using PARSTAT 2273 potentiostat equipment (PowerCORR, 2001). As a means of evaluating the instantaneous corrosion rate of reinforcing steel in concrete, the technique is applied to measure corrosion current density using a three-electrode system comprising: (i) reference electrode; (ii) counter electrode (steel plate) which was connected to the respective terminals of the potentiostat; and (iii) steel reinforcement in the concrete specimen (often known as the *working electrode*) polarized to ±20 mV from its equilibrium (*rest*) potential at a perturbation scan rate of 0.166 mV/s.

Steel reinforcement was then polarized after an appropriate *initial delay* of about 1 min, and the linear polarization resistance R_p (kΩ · cm^2) was determined from the slope of the applied potential *versus* measured current plot. Corrosion current density I_{corr} (μA/cm^2) is then calculated using Eq. 9.

$$I_{corr} = \frac{B}{R_p} \qquad (9)$$

where $B = \frac{\beta_a \beta_c}{(\beta_a + \beta_c)}$ (with β_a and β_c being, respectively, anodic and cathodic Tafel constants); R_p = polarization resistance (kΩ · cm^2).

A Tafel plot is normally utilized to find Tafel constants, but when the plot is *not* available, *B*-values of 26 and 52 mV have been recommended (Al-Tayyib & Khan, 1988) for steel bars in *active* and *passive* states, respectively. In this work, a value of 26 mV was used.

2.3.2 Gravimetric weight loss method (GWLM)

Following corrosion rate measurements using LPRM, all concrete samples were then broken for determination of corrosion rate by gravimetric weight loss method. Preparation, cleaning, and estimation of weight loss were done in accordance with ASTM G1-03. The cleaning solution was prepared by adding 20 g of *antimony trioxide* and 50 g of *stannous chloride* in 1000 ml of hydrochloric acid. Samples of corroded steel bars obtained from test specimens before and after cleaning are shown in Figure 4.

Figure 4. Samples of corroded rebars before and after cleaning.

The weight loss ΔW was calculated as

$$\Delta W = W_i - W_f \tag{10}$$

where W_i = initial weight of the bars before corrosion (g) and W_f = weight of the bars after cleaning all rust products (g).

Then the corrosion penetration rate, P_r (μm/year) was determined using Eq. 11:

$$P_r = \frac{1.116 \times 10^7 \times \Delta W}{A \times T} \tag{11}$$

in which A = exposed surface area of rebar (cm^2) and T = exposure time (hours).

Corrosion penetration rate P_r was converted to corrosion current density I_{corr} using a simplified relationship (Alghamdi & Ahmad, 2010) given by Eq. 12:

$$I_{corr} = \frac{P_r}{11.7} \tag{12}$$

3. RESULTS AND DISCUSSION

3.1 Correlation between $I_{corr,g}$ and $I_{corr,e}$

Values of reinforcement corrosion rate measured in terms of corrosion current density using electrochemical and gravimetric methods (designated as $I_{corr,e}$ and $I_{corr,g}$, respectively) obtained for all the test specimens were analyzed with full details presented previously (Alghamdi & Ahmad, 2010). A close inspection of results obtained indicates that effects of aggregate type, cover thickness, and chloride concentration on correlation between the corrosion current density determined by LPRM and GWLM are practically insignificant. Hence, a single plot of corrosion current density values determined by LPRM against corresponding corrosion current density values determined by GWLM for all 486 test specimens is shown in Figure 5 in which good correlation between $I_{corr,g}$ and $I_{corr,e}$ values are clearly reflected by correlation coefficient value nearly equal to 0.96. Correlation analysis also indicates that value of gravimetric corrosion current density is, on average, nearly equal to 86% of values of electrochemical corrosion current density.

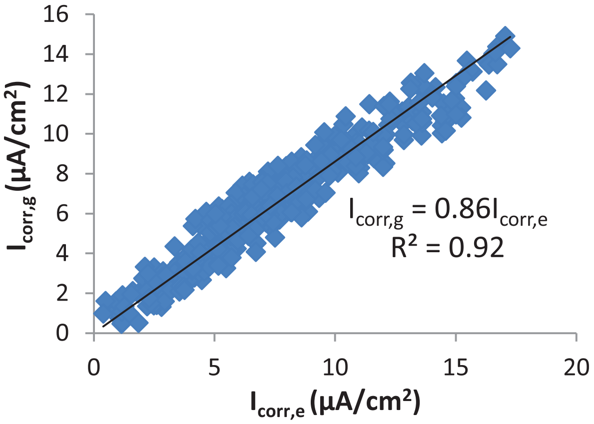

Figure 5. Correlation of $I_{corr,g}$ to $I_{corr,e}$ for all test specimens.

3.2 Regression models for corrosion rate

Since gravimetrically computed reinforcement corrosion rates were found to be more accurate and reliable, the models for predicting corrosion current density in terms of water–cementitious materials ratio (w/cm ratio), fine to total aggregate ratio, cover thickness, cementitious materials content, and chloride concentration were developed using Minitab software (Alghamdi & Ahmad, 2010), separately for the two aggregate types using gravimetric data. A linear-type regression model was selected considering the linear variation of corrosion rate with major design factors (including $R_{w/c}$, C_c, $R_{F/T}$, C_L, and C_v) affecting corrosion rate. Regression models for corrosion current density obtained for both types of aggregates are presented in Table 2.

Table 2. Regression models for corrosion current density.

Aggregate type	Regression models	R²
H	$I_{corr} = -13 + 34.4\ R_{w/c} + 0.024\ C_c + 3.83\ R_{F/T} + 0.097\ C_L - 0.181\ C_v$	0.82
T	$I_{corr} = -18.4 + 43.8\ R_{w/c} + 0.03\ C_c + 4.77\ R_{F/T} + 0.12\ C_L - 0.203\ C_v$	0.94

3.3 Durability-based optimal design methodology

Durability-based design of typical RC members can be performed by utilizing regression models (outlined in the previous section) for prediction of corrosion current density. For a specified design service life (Sarja & Vesikari, 1996), the minimum value of corrosion current density I_{corr} were used to determine loss of steel rebar due to corrosion and was then utilized within a durability-based optimal structural design search methodology.

The following *five-step design procedure* (Alghamdi & Ahmad, 2010) was utilized for durability-based design RC beams and columns:

(i) *Microsoft Excel solver* is used to determine optimum values of water to cementitious material ratio ($R_{W/C}$), cementitious material content (C_C), concrete cover (C_V), and fine to total aggregate ratio ($R_{F/T}$) corresponding to minimum corrosion current density $I_{corr,g}$ for specified chloride concentration using the models developed for $I_{corr,g}$.

(ii) Compressive strength f'_c and elastic modulus E_c of concrete are determined using optimum values of $R_{W/C}$, C_C, and $R_{F/T}$ in the following models (Adamu, 2011).

$$f'_c = -61.24 - 0.056C_c - 19.87\ Exp\left(2.083\ R_{w/c}\right) + 183.45\ R_{F/T}^{0.119} \tag{14}$$

$$E_c = -49.10 - 0.0048C_c - 13.23\ Exp\left(2.083\ R_{w/c}\right) + 145.68\ R_{F/T}^{0.106} \tag{15}$$

(iii) Loss rates of concrete cover and rebar diameter are determined from Eqs. 3 and 7 using the data obtained in steps (i) and (ii) given above. A preliminary section is then specified for a given structural member and the residual dimensions of cross-section and rebar diameter are determined.

(iv) A durability-based structural design of RC concrete members is performed using design information obtained in step (iii) given above.

(v) Adequacy of final cross-section and rebar diameter is checked against prescribed design requirements.

The design methodology for carrying out durability-based optimal structural designs is outlined in the flowcharts shown in Figures 6 and 7 for design of RC beams and columns, respectively. The design procedure was implemented in two Microsoft Excel programs (designated as: RC_B_DDesign and RC_C_DDesign) suitable for automated design of RC beams and columns, respectively, in corrosive environment. Sample values of typical input and output results obtained for durability-based design of typical design case studies for RC beams and columns are summarized in Tables 3 and 4, respectively.

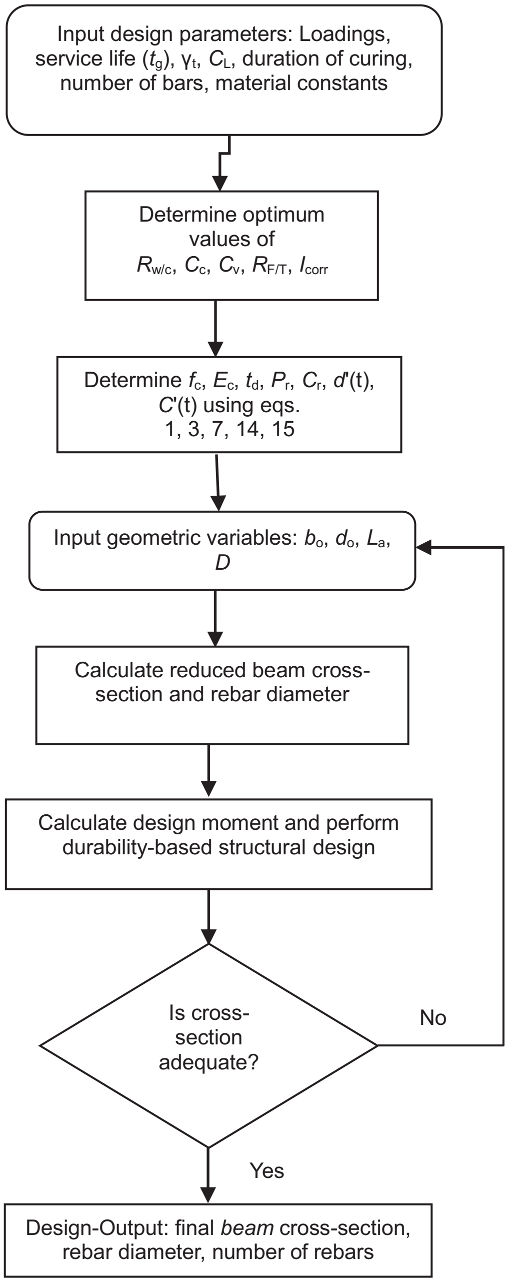

Figure 6. Flowchart for durability-based design of RC beam.

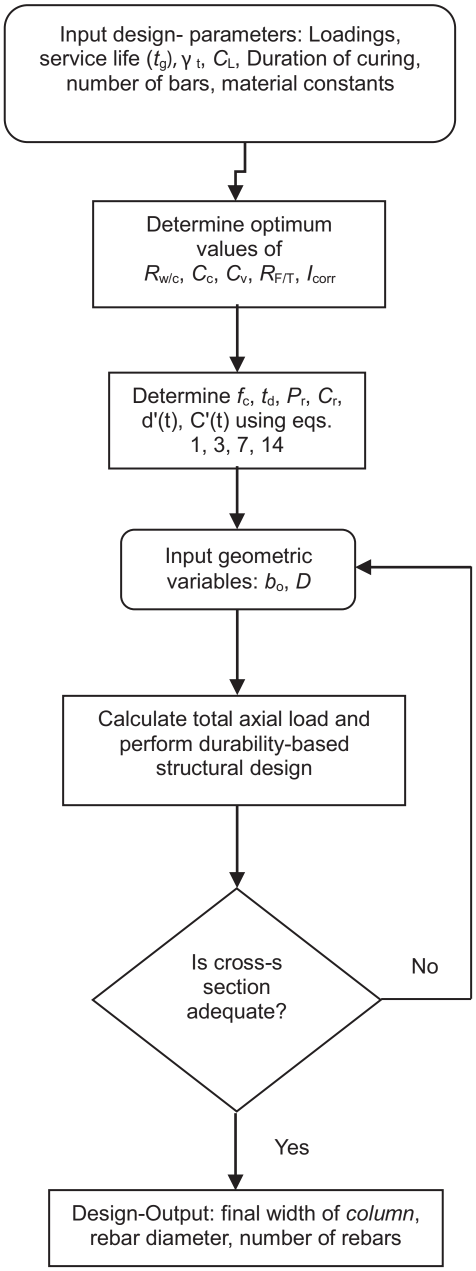

Figure 7. Flowchart for durability-based design of RC column.

Table 3. Sample design results obtained by RC_B_DDesign program for durability-based optimal design of RC beam.*

	Input design values		Optimal design values		
S/N	Target life, t_g (years)	C_L (%)	Beam-width b_{optim} (mm)	Beam-depth, d_{optim} (mm)	Bar diameter $D_{optim} = D + \Delta D$ (mm)
1	40	3	200	528	18.11
2		7	200	403	20.71
3		12	200	310	23.59
4	50	3	225	436	19.60
5		7	225	331	22.49
6		12	225	300	23.63
7	70	3	225	345	22.04
8		7	225	300	23.63
9		12	225	300	23.63

***Note:** Length = 3 m; dead-load = 40 kN/m; live-load 12 kN/m; ΔD = additional increment to steel bar diameter to off-set corrosion effect.

Table 4. Sample design results obtained by RC_C_DDesign for durability-based optimal design of RC column.*

	Input design values		Optimal design values	
S/N	Target service life, t_g (years)	C_L (%)	Column width, b_{optim} (mm)	Bar diameter, $D_{optim} = D + \mathrm{D}$ (mm)
1	40	3	150	15.76
2		7	150	15.76
3		12	150	15.76
4	50	3	150	15.76
5		7	150	15.76
6		12	170	15.76
7	70	3	150	15.76
8		7	268	15.76
9		12	650	15.76

***Note:** Dead-load = 100 kN; live-load = 120 kN.

4. CONCLUSION

In this research study, 486 concrete specimens with centrally placed reinforcing steel bar were subjected to experimental conditions for chloride-induced corrosion. The study was conducted to develop correlation relationship between corrosion current density values determined based on data compiled from linear polarization resistance method (LPRM) and gravimetric weight loss method (GWLM). Reliable models for prediction of reinforcement corrosion rate were also developed and utilized within an automated methodology for durability-based design of RC beams and columns. And to demonstrate utilization of the design methodology, the results obtained from design case studies of sample RC beams and columns are summarized and presented in this paper.

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