Sustainable Design of Reinforced Concrete Structures through CO_2 Emission Optimization

DongHun Yeo, M.ASCE[1]; and Florian A. Potra[2]

Abstract: Efforts are being made to achieve more efficient operation of buildings, with the goal of reducing the construction industry's contribution to energy consumption and greenhouse gas emissions. That contribution also includes the energy embodied in structures; that is, the energy consumed in the processes of extracting, manufacturing, transporting, and installing construction materials (including recycled materials) and elements. In particular, in spite of the use of additives such as fly ash, reinforced concrete (RC) structures, which are large consumers of cement, are responsible for a sizable proportion of worldwide carbon emissions. These emissions can be reduced significantly through the more efficient use of both concrete and steel that can be achieved by optimization. Modern optimization tools are now available that make it possible to perform large volumes of calculations efficiently that are applicable to a wide variety of structural engineering problems. This study presents an optimization approach developed with a view to allowing decision makers to balance sustainability and economic objectives. To illustrate this approach, an RC frame under gravity and lateral loads is considered in this paper. It was found that, depending upon the parameter values used in the calculations, the design optimized with respect to the CO_2 footprint yields a CO_2 footprint that is lower (by 5% to 10%) than the design optimized with respect to cost. The reduction can be smaller for low-rise structures and other structures with predominantly tension-controlled members. However, for structures whose members predominantly experience large compressive forces, such as high-rise buildings, the reduction may be more significant. This also may be true of certain prestressed and poststressed concrete members. Additional research aimed at ascertaining the extent to which this is the case is warranted. **DOI: 10.1061/(ASCE)ST.1943-541X.0000888.** *© 2014 American Society of Civil Engineers.*

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Introduction

Worldwide, buildings are responsible for between 25% and 40% of total energy use (IEA 2005). According to studies carried out by the Organisation for Economic Cooperation and Development (OECD), the residential and commercial building sectors are responsible for approximately 30% of primary energy consumed and of greenhouse gas emissions in OECD countries (OECD 2003).

Most efforts to reduce carbon dioxide (CO_2) emissions during a given building's service life are focused on reducing the energy required to operate and maintain it (i.e., the operating energy). Measures that significantly reduce operating energy have been implemented by design professionals and the building industry (WBCSD 2008). Some of these measures, like solar roofing, are fairly radical. Others, such as reducing the energy consumption of refrigerators, are more incremental in nature. However, operating energy is only one part of the total energy that buildings consume. Indeed, raw material acquisition, transport, processing (manufacturing), distribution, and construction represent embodied energy. Provided that a cradle-to-grave system boundary is employed (Goggins et al. 2010), the calculation of embodied energy also accounts for the energy used for demolition (Yohanis and Norton 2002).

The quantification of the embodied energy and CO_2 footprint for any particular building material is an inexact science and requires a "long view" look at the entire manufacturing and utilization process [using, e.g., life cycle assessment (LCA); see Goggins et al. 2010]. Nevertheless, reasonable estimates of the embodied energy and CO_2 footprint of most common construction materials have been compiled (e.g., Alcorn 2003; Venkatarama Reddy and Jagadish 2003; CTBUH 2009; Hammond and Jones 2008), and will be used in this paper.

The embodied energy of building materials, including concrete, can account for a fairly significant share of the total energy use of a country. Estimates suggest that 10% of the total energy consumption in the United Kingdom and Ireland is embodied in materials (UNDP 2007). Embodied energy's share of total life-cycle energy was estimated to vary from country to country, with estimates ranging as low as 5% and as high as 40% (Sartori and Hestnes 2007). These percentages are likely to increase as the amount of operating energy decreases (Yohanis and Norton 2002). The energy embodied in reinforced concrete (RC) structures contributes a nonnegligible part—as much as 5% to 10%—of that share.

For materials used in typical concrete mixes, the embodied energy and CO_2 footprint values per unit volume are relatively low. However, because concrete is the most widely used material in construction, their total values in RC structures are significant. Also, unlike steel, concrete typically is not recycled for direct reuse in most structures.

For RC structures, embodied energy or CO_2 footprint reduction can be achieved not only by the use of novel building materials, such as low-carbon cements and clinker substitutes

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(Davidovits 1993; Gartner 2004; WBCSD-IEA 2009), and recycling (Thormark 2002), but also by reducing the CO_2 footprint through the optimization of RC structural designs. In current practice, structural designs are typically optimized for total cost or total weight. From the viewpoint of sustainability, however, optimized designs for embodied energy or the CO_2 footprint are desirable as well. The authors emphasize that the CO_2 footprint reduction considered in this paper concerns only the RC structure, and that the CO_2 footprint embodied in the RC used in a building is only a fraction of the overall CO_2 footprint embodied in that building. Nevertheless, the reduction of the footprint embodied in the RC is a useful contribution to the reduction of the overall footprint.

Recent research has served to demonstrate the early interest in considering environmental factors in the optimization of RC structures. Paya-Zaforteza et al. (2009) used an approximate optimization method based on simulated annealing to minimize two objective functions: (1) the total CO_2 emissions embodied in the structure and (2) the total structural cost. The design variables included the type of concrete and steel reinforcement for the columns and beams of each floor, the dimensions of the cross sections of the columns and beams, and the details of the longitudinal and shear reinforcement in the columns and beams. The methodology was applied to six typical building frames, with up to four bays and up to eight floors. The authors considered the objective functions one at a time and found that the optimum structure from the standpoint of minimizing emissions is only marginally (2.8%) more expensive than the optimum structure for minimizing cost.

Villalba et al. (2010) carried out a similar study for cantilever earth-retaining walls with heights from 4 to 6 m and again found that the optimum structure from the standpoint of minimizing embedded CO_2 emissions is only marginally (1.4%) more expensive than the optimum structure for minimizing cost. Interestingly, the authors found that walls optimized for minimum cost require on average approximately 5% more concrete than walls optimized for minimum embedded CO_2 emissions, although the latter require on average approximately 2% more steel. Furthermore, the concrete grade is larger in the case of the emissions-optimized walls.

Yeo and Gabbai (2011) investigated the implications, from the point of view of cost, of optimizing a simple RC structural member (a rectangular beam of fixed moment and shear strengths) such that embodied energy is minimized. The results indicated that optimization of structural member design for minimum embodied energy results in decreases on the order of 10% in embodied energy, at the expense of an increase of about 5% in cost compared to a cost-optimized design. The exact reduction in embodied energy depends significantly on the value of the cost ratio of steel reinforcement to concrete, where that ratio must take into account not only the material costs of the concrete and steel, but also construction costs such as the placement costs of concrete and the installation costs of reinforcement. Also, results show that the minimum-embodied-energy section has a smaller volume of concrete and a larger amount of reinforcement compared to the section designed for minimum cost. These findings confirmed those of Villalba et al. (2010). To ensure that ductility is adequate for design purposes in spite of the increase in the amount of steel, the constraints in the optimization procedure include a constraint with respect to the strain in the reinforcing bars.

The main objective of this paper is to apply an optimization method based on mathematical expressions of constraints and objective functions to a simple case study—a frame structure under gravity and lateral load—to explore the implications, from a cost standpoint, of using the total CO_2 footprint as the objective function to be minimized. The structure considered in the case study is a simplified model that mimics the essential features of an actual frame. For comparison, the implications from the standpoint of the CO_2 footprint are also examined for the case in which the total cost is used as the objective function. In each case, the role of the ratio of the cost of steel to that of concrete on the conclusions is also examined. The research is a first step toward developing more elaborate optimization procedures, based on more than one objective function, to be used as tools for making optimal decisions entailing the societal costs of carbon emissions.

Case Study: Description and Optimization Methodology

Problem Description

The study considers an RC single frame (height $H = 4.7$ m and length $L = 12$ m) consisting of one beam and two columns (Fig. 1). It is assumed that (1) the column has a square section with dimension h_c, (2) the beam has height h_b, and (3) the beam width is $b_b = h_c$. (The latter assumption is adopted for the sake of simplification; in practice, the width of the beam is typically less than the width of the columns to avoid reinforcement interference.) The structure is assumed to be subjected to gravity loading uniformly applied to the beam, and wind-induced lateral loading applied at height H as a concentrated load. Based on the provisions of the ASCE 7-10 Standard (ASCE 2010), and denoting the tributary width of the structure by B, the gravity load consists of dead load and live load, estimated to be $q_D = 2B$ [kN/m^2] and $q_L = 4B$ [kN/m^2], respectively. The wind loads, induced by wind speeds

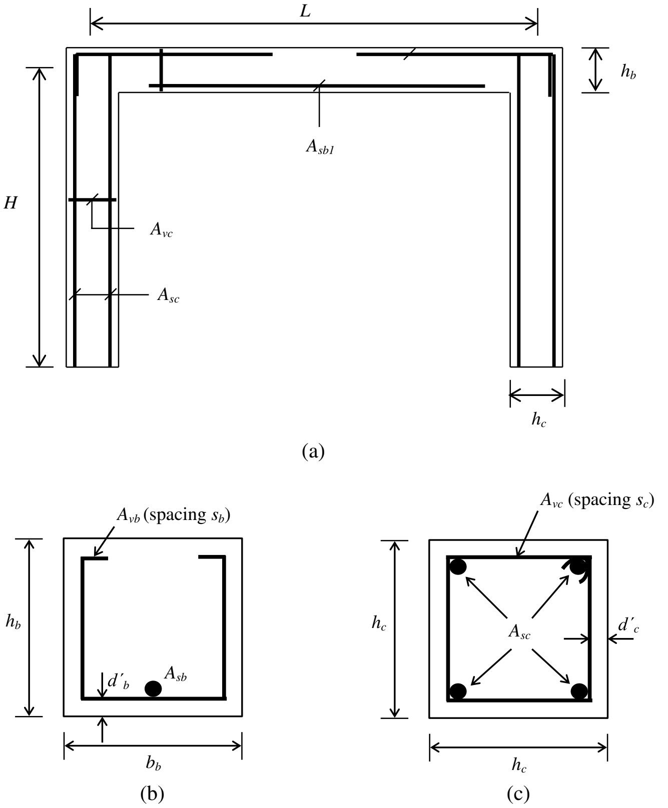

Fig. 1. Schematic of a frame structure and section details: (a) frame structure; (b) beam section; (c) column section

Section	Case	P_u	M_u	V_u
Beam (mid)	1	–	$\frac{q_1L^2(3\beta e+2)}{24(\beta e+2)}$	–
Beam (end)	1	–	$-\frac{q_1L^2}{6(\beta e+2)}$	$-\frac{q_1L}{2}$
	2	–	$-\frac{q_2L^2}{6(\beta e+2)}-\frac{WH}{2}\frac{3e}{\beta+6e}$	$-\frac{q_2L}{2}-\frac{3WH}{L}\frac{e}{\beta+6e}$
Column (end)	1	$\frac{q_3L}{2}-\frac{3WH}{L}\frac{e}{\beta+6e}$	$-\frac{q_3L^2}{6(\beta e+2)}+\frac{WH}{2}\frac{3e}{\beta+6e}$	–
	2	$\frac{q_1L}{2}$	$-\frac{q_1L^2}{6(\beta e+2)}$	–
	3	$\frac{q_2L}{2}+\frac{3WH}{L}\frac{e}{\beta+6e}$	$-\frac{q_2L^2}{6(\beta e+2)}-\frac{WH}{2}\frac{3e}{\beta+6e}$	–
	4	$\frac{q_1L}{2}+1.2\rho_c h_c^2 H$	$-\frac{q_1L^2}{12(\beta e+2)}$	$\frac{q_1L^2}{4H(\beta e+2)}$
	5	$\frac{q_2L}{2}+1.2\rho_c h_c^2 H+\frac{3WH}{L}\frac{e}{\beta+6e}$	$\frac{q_2L^2}{12(\beta e+2)}+\frac{WH}{2}\frac{\beta+3e}{\beta+6e}$	$\frac{q_2L^2}{4H(\beta e+2)}+\frac{W}{2}$

Note: β is defined as the ratio of inertia moment of the column to inertia moment of the beam,

and e is defined as *H/L*.

Fig. 2. Internal forces on critical sections of members

with a 700-year mean recurrence interval (MRI) and a 50-year MRI, were assumed to be $W_{700\text{-}yr} = 1.33BH$ [kN/m^2] and $W_{50\text{-}yr} = 0.814BH$ [kN/m^2], respectively. In addition to those loads, loads due to self-weight of the members were taken into account. Three load combinations were employed:

$$\text{LC1: } 1.2D + 1.6L_l \tag{1a}$$

$$\text{LC2: } 1.2D + L_l + W \tag{1b}$$

$$\text{LC3: } 0.9D + W \tag{1c}$$

where D is the dead load, L_l is the live load, and W is the wind load. The corresponding ultimate design loads [i.e., P_u (axial force), M_u (bending moment), and V_u (shear force)] acting on the critical sections of the beam and columns are summarized in Fig. 2, where q_1, q_2, and q_3 denote uniformly distributed ultimate gravity loads corresponding to the load combinations (LCs) specified in Eqs. (1*a*)–(1*c*), respectively. The design of the frame structure for strength and serviceability is based on the ACI 318-11M Code (ACI 2011). In addition to the loads listed in Fig. 2, two cases were considered where the columns were subjected to additional axial compression forces: (1) $P = 3{,}000$ kN, and (2) $P = 6{,}000$ kN. (These forces are, respectively, about 40% and 80% of the full compression strength of the concrete $f'_c A_g$.) This was done with the goal of assessing the effect of hypothetical gravity loads due to additional floors in multistory buildings.

The objective of this study is to use optimization methods to determine feasible designs that minimize both cost and the CO$_2$ footprint, and to provide an insight into the trade-offs between cost and energy optimization in structural design.

Design Variables and Parameters

The design variables are the beam height h_b; the column height h_c; the total area of the longitudinal reinforcement A_{sb1} for the mid-section and A_{sb2} for the end-section of the beam; the total area of the axial reinforcement A_{sc} of the column; the spacing s_{b1} and s_{b2} of the shear reinforcement for the midspan and end span of the beam; and the spacing s_{c1} and s_{c2} for the midspan and end span of the column, the area of each reinforcement bar provided for shear resistance being $A_{vb} = 201$ mm^2, corresponding to a #5 (U.S.) reinforcing bar. The length of the end span is assumed to be $L_{vb} = L/4$ for each end of the beam and $L_{vc} = H/4$ for the columns, while the length of the midspan is $L/2$ for the beam and $H/2$ for the columns. For numerical convenience, all nine variables are treated as continuous variables. The design parameters, defined as constants during the optimization process, are listed in Table 1, and the authors believe that they represent common values used in RC practice.

Table 1. Design Parameters and Corresponding Values

Parameter	Value
Concrete compressive strength	$f'_c = 40$ MPa (5.8 ksi)
Reinforcement yield strength	$f_y = 420$ MPa (60 ksi)
Modulus of elasticity of steel	$E_c = 2\times10^5$ MPa (29,000 ksi)
Specific mass of concrete	$\rho_c = 2{,}400$ kg/m^3 (150 pcf)
Specific mass of steel	$\rho_s = 7{,}850$ kg/m^3 (490 pcf)
Lightweight concrete factor	$\lambda = 1$ (for normal weight)
Strength reduction factor for shear	$\phi_v = 0.75$
Strength reduction factor for flexure	$0.817 \le \phi_b \le 0.9$
Strength reduction factor for axial force	$0.65 \le \phi_s \le 0.9$
Maximum usable strain at extreme concrete compression fiber	$\varepsilon_{cu} = 0.003$
Tributary width	$B = 7$ m
Concrete cover (includes radius of fictitious bar having area A_s)	$d' = 65$ mm
Area of shear reinforcement [#5 (U.S.)]	$A_{vb} = A_{vc} = 201$ mm^2
Diameter of shear reinforcement [#5 (U.S.)]	$d_{vc} = 15.875$ mm

Objective Functions

The objective functions corresponding to the minimization of cost and the CO_2 footprint are

$$\begin{aligned} f_1 = C^c\Bigg\{ & 2h_c^2 H + h_c h_b L + \left(\rho_s \frac{R_C}{100} - 1\right) \\ & \times \Bigg[0.7LA_{sb1} + LA_{sb2} + \frac{2L_{vb}A_{vb}}{s_b}(2h_b + h_c) \\ & + \frac{(L - 2L_{vb})A_{vb}}{s_{\max,b}}(2h_b + h_c) + (2.2H - h_b)A_{sc} \\ & + \frac{4L_{vc}A_{vc}}{s_c}(4h_c - 4d_b') + \frac{2(H - 2L_{vc})A_{vc}}{s_{\max,c}}(4h_c - 4d_b')\Bigg]\Bigg\} \end{aligned} \tag{2}$$

and

$$\begin{aligned} f_2 = E^c\Bigg\{ & 2h_c^2 H + h_c h_b L + \left(\rho_s \frac{R_{CO_2}}{100} - 1\right) \\ & \times \Bigg[0.7LA_{sb1} + LA_{sb2} + \frac{2L_{vb}A_{vb}}{s_b}(2h_b + h_c) \\ & + \frac{(L - 2L_{vb})A_{vb}}{s_{\max,b}}(2h_b + h_c) + (2.2H - h_b)A_{sc} \\ & + \frac{4L_{vc}A_{vc}}{s_c}(4h_c - 4d_b') + \frac{2(H - 2L_{vc})A_{vc}}{s_{\max,c}}(4h_c - 4d_b')\Bigg]\Bigg\} \end{aligned} \tag{3}$$

where C^c and E^c are the cost and the CO_2 footprint of concrete per cubic meter, respectively; R_C is the ratio of the cost of steel per 100 kg to the cost of concrete per cubic meter; R_{CO_2} is the ratio of the CO_2 footprint of 100 kg of reinforcement steel to the CO_2 footprint of concrete per cubic meter; and ρ_s is the specific mass of steel. The first and second terms in the braces of Eqs. (2) and (3) are the gross volume of the concrete in the columns and the beam. The expressions between the brackets in Eqs. (2) and (3) are the volume V_s of steel in the columns and the beam (see Fig. 1 for details). In Eqs. (2) and (3), the cost of steel in the structure is calculated as a product of the volume of steel V_s and the cost of steel per volume ($C^c \rho_s R_C/100$), while the CO_2 footprint of steel is calculated as a product of V_s and the CO_2 footprint of steel per volume ($E^c \rho_s R_{CO_2}/100$). The product of the term -1 in the expression in parentheses by the expression in the brackets (i.e., the volume of steel V_s) changes the gross volume to the net volume of concrete.

Estimates of the CO_2 footprints and the costs of construction materials can vary with time and location (Alcorn 2003; Guerra et al. 2011; Paya-Zaforteza et al. 2009; Sahab et al. 2005). The values employed in this study are summarized in Table 2.

Table 2. CO_2 Footprint and Cost of Concrete and Reinforcing Steel

Material	CO_2 footprint (Alcorn 2003)	Cost
Concrete ($f_c' = 30$ MPa)	376 (CO_2 kg/m^3)	130 (\$/m^3)
Concrete ($f_c' = 40$ MPa)	452 (CO_2 kg/m^3)	135 (\$/m^3)
Steel, recycled ($f_y = 420$ MPa)	35.2 (CO_2 kg/100 kg)	108 (\$/100 kg)

Note: Costs are given in U.S. dollars.

Formulation of Optimization Problem and Solution Method

Constraints for this optimization problem can be divided into two parts: constraints for serviceability and for strength. The constraints for serviceability are maximum allowable vertical deflection [$L/240$; Eq. (4)] and maximum allowable horizontal deflection [$H/400$; Eq. (5)]:

$$\frac{qL^4}{48E_c I_b}\left(\frac{5}{8} - \frac{1}{\beta e + 2}\right) - \frac{L}{240} \le 0 \tag{4}$$

$$\frac{W_{50\text{-}yr}H^3}{24E_c I_c}\left(\frac{4\beta + 6e}{\beta + 6e}\right) - \frac{H}{400} \le 0 \tag{5}$$

where I_b and I_c are the moments of inertia for the beam and the column, respectively; E_c is the modulus of elasticity of concrete; and $W_{50\text{-}yr}$ is the wind-induced lateral load for MRI = 50 years as previously defined.

The constraints for the strength of the beam include the flexural strengths at midspan and at the ends of the member [Eq. (6)], the minimum and maximum requirements for flexural reinforcement [Eqs. (7) and (8)], the shear strength at the ends [Eq. (9)], and the minimum and maximum requirements for the spacing of shear reinforcement [Eqs. (10)–(12)]:

$$M_u - \phi_b M_n \le 0 \tag{6}$$

$$\max(0.25\sqrt{f_c'}, 1.4)\frac{b_b d_b}{f_y} - A_{sb} \le 0 \tag{7}$$

$$A_{sb} f_y - \frac{3}{7}0.85 f_c' \beta_1 (h_b - d_b') b_b \le 0 \tag{8}$$

$$V_u - \phi_v 0.17\lambda\sqrt{f_c'} b_b (h_b - d_b') - \phi_v \frac{A_{vb} f_y d_b}{s_b} \le 0 \tag{9}$$

$$s_b - \frac{A_{vb} f_y}{\max\left(\frac{\sqrt{f_c'}}{16}, \frac{1}{3}\right) h_b} \le 0 \tag{10}$$

$$s_b \le \begin{cases} \min\left(\frac{d_b}{2}, 600\text{ mm}\right) & \text{for } V_s < 0.33\sqrt{f_c'} b_b d_b \\ \min\left(\frac{d_b}{4}, 300\text{ mm}\right) & \text{for } V_s \ge 0.33\sqrt{f_c'} b_b d_b \end{cases} \tag{11}$$

$$\frac{A_{vb} f_y d_b}{s_b} - 4\lambda\frac{\sqrt{f_c'} b_b d_b}{6} \le 0 \tag{12}$$

where b_b is the beam width; d_b is the distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement of the beam (defined as the difference between the height d_b and the concrete cover d_b'); A_{sb} is the flexural reinforcement; and β_1 is the area of the factor relating the depth of equivalent rectangular compression stress block to the neutral axis depth. Additional variables in these equations have been defined in previous sections of this paper or in Table 1.

The constraints for the strength of the columns are functions of the combined axial forces and moments [Eq. (13)], the minimum and maximum requirements for the area of axial reinforcement [Eqs. (14) and (15)], shear strength [Eq. (16)], and the minimum and maximum requirements for the spacing of ties [Eqs. (17)–(19)]:

$$f(M_u, P_u, \phi_b M_n, \phi_c P_n) \le 0 \tag{13}$$

$$0.01h_c^2 - A_{sc} \le 0 \tag{14}$$

$$A_{sc} - 0.08h_c^2 \le 0 \tag{15}$$

$$V_u - \phi_v 0.17\lambda\sqrt{f'_c}\left(1+\frac{P_u}{14h_c^2}\right)h_c(h_c-d'_c) - \frac{\phi_v A_{vc} f_y (h_c-d'_c)}{s_c} \le 0 \tag{16}$$

$$s_c - \frac{A_{vc} f_y}{\max\left(\frac{\sqrt{f'_c}}{16},\frac{1}{3}\right)h_c} \le 0 \tag{17}$$

$$s_c - \min(h_b, 48d_{vc}) \le 0 \tag{18}$$

$$\frac{A_{vc} f_y d_c}{s_c} - 4\lambda\frac{\sqrt{f'_c}h_c d_c}{6} \le 0 \tag{19}$$

where h_c is the width of the square column, d'_c is the concrete cover, d_c is the distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement of the column, A_{sc} is the area of axial reinforcement of the column, s_c is the spacing of the shear reinforcement with area of A_{vc}, and d_{vc} is the diameter of the shear reinforcement. Eq. (13) represents the analytical expression of the RC eccentricity-dependent axial-force/bending moment interaction equation. All constraints pertaining to RC member design for serviceability and strength [i.e., Eqs. (4)–(19)] are based on the ASCE 7-10 Standard (ASCE 2010) and the ACI 318-11M Code (ACI 2011).

In this study, the constrained nonlinear optimization solver "fmincon" from MATLAB was used. Since this solver is not guaranteed to compute the global optimum of the problem, it is useful to implement the procedure multiple times, selecting for each implementation a random starting point defined by a set of values for the design variables. This avoids obtaining local minima satisfying all the constraints and increases the chances of obtaining the global minimum of the problem.

Results

It is assumed that the cost and CO_2 footprint of concrete are specified as in Table 2. The corresponding values for steel are defined for four values of R_C ($R_C = 0.6$, 0.8, 1.0, and 1.2) and three values of R_{CO_2} ($R_{CO_2} = 0.068$, 0.078, and 0.088). The choice of R_{CO_2} ratios was based on estimates of the CO_2 footprint of recycled steel (approximately 35 kg of CO_2 per 100 kg of steel) and the CO_2 footprint of concrete (approximately 400 kg to 500 kg of CO_2 per cubic meter of concrete; for example, $35/450 = 0.078$) (Alcorn 2003). The ratio between the cost of the cost-optimized frame and the cost of the CO_2-optimized frame is denoted by r_{cost}, and the ratio between the CO_2 footprint of the cost-optimized frame and the CO_2 footprint of the CO_2-optimized frame is denoted by R_{CO_2}.

It was indicated previously that three gravity loadings for the columns were considered: (1) loadings due to the self-weight of the frame, (2) loadings due to case (1) and an additional load $P = 3{,}000$ kN, and (3) loadings due to case (1) and an additional load $P = 6{,}000$ kN. The three cases corresponded to three qualitatively different interaction equation diagrams, corresponding to the case of relatively large, medium, and small eccentricities of the axial force. For case (1), the calculations showed that the difference between the CO_2 footprints inherent in the cost-optimized and CO_2-optimized designs was less than 2%. Therefore, this study focuses on cases (2) and (3).

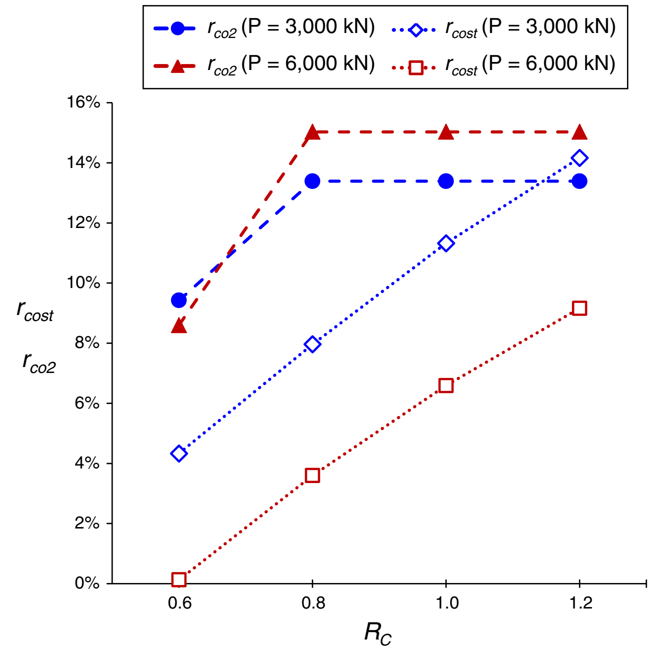

Fig. 3. Dependence upon R_C of the differences in cost and CO_2 footprint (in percentages of totals for the frame) between a cost-optimized frame and a CO_2-optimized frame, for $R_{CO_2} = 0.078$ and $f'_c = 40$ MPa

Dependence upon R_C and R_{CO_2} of Difference in Costs and CO_2 Footprints

First, an investigation was performed that looked into the dependence of design, optimized for cost or CO_2 footprint, on the variation of the relative cost between concrete and steel. Fig. 3 shows, for $P = 3{,}000$ kN and $P = 6{,}000$ kN, the dependence upon R_C of the difference in costs, r_{cost} (in percentages of totals for the frame), between the cost-optimized frame and the CO_2-optimized frame;

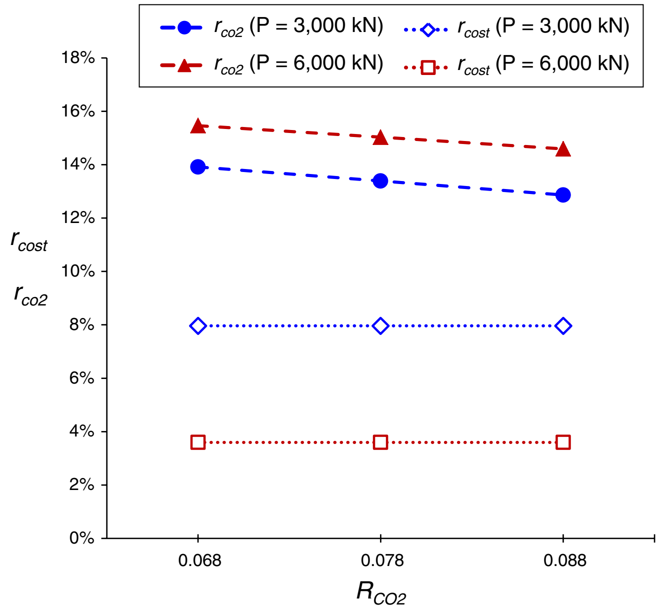

Fig. 4. Dependence upon R_{CO_2} of the differences in cost and CO_2 footprint (in percentages of totals for the frame) between a cost-optimized frame and a CO_2-optimized frame, for $R_C = 0.8$ and $f'_c = 40$ MPa

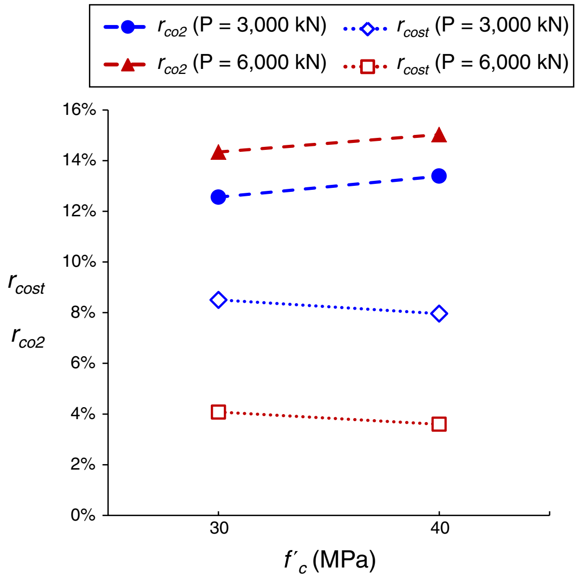

Fig. 5. Dependence upon f'_c of the difference in cost and CO_2 footprint (in percentages of totals for the frame) between a cost-optimized frame and a CO_2-optimized frame, for $R_C = 0.8$ and $R_{CO_2} = 0.078$

and of the difference in CO_2 footprints, r_{co_2}, between the cost-optimized frame and the CO_2-optimized frame. The higher value of R_C corresponds to an increase in the cost of steel and the cost of concrete being fixed. Note that the differences between the CO_2 footprint of the cost-optimized and the CO_2-optimized frame generally increase as P increases. This suggests that the optimization is more effective in reducing the frame's CO_2 footprint if the members are subjected to large compressive forces; in particular, the potential of optimization from this point of view would be stronger for high-rise than for low-rise RC structures.

The dependence of the results upon the assumed values of the concrete and the steel footprint is represented in Fig. 4, which shows that as the ratio R_{CO_2} increases (i.e., as the CO_2 footprint of steel is larger), the advantage of optimizing the CO_2 footprint decreases.

Dependence upon Concrete Compressive Strength of Difference in Costs and CO_2 Footprints

In addition, an investigation was performed that looked into the effects of concrete compression strength on optimization effectiveness (Fig. 5). As the concrete strength increases, the difference r_{cost} between the costs of the cost-optimized and the CO_2-optimized frames decreases. In addition, the difference R_{CO_2} between the CO_2 footprints of the cost-optimized and the CO_2-optimized frames

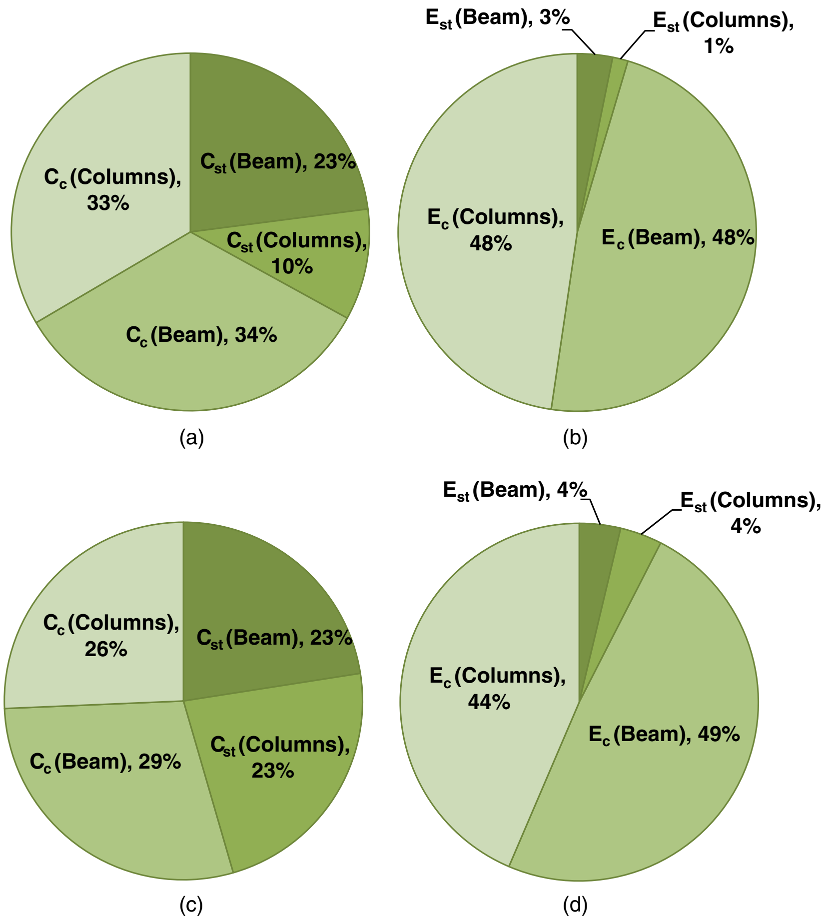

Fig. 6. Contributions of concrete and reinforcement to the total cost and total CO_2 footprint (notations: C_c = cost of concrete; C_{st} = cost of reinforcing steel; $E_c = CO_2$ footprint of concrete; $E_{st} = CO_2$ footprint of reinforcing steel), for $R_C = 0.8$, $R_{CO_2} = 0.078$, $f'_c = 40$ MPa, and $P = 6{,}000$ kN: (a) cost ratio for cost-optimized frame; (b) CO_2 footprint ratio for a cost-optimized frame; (c) cost ratio for a CO_2-optimized frame; (d) CO_2 footprint ratio for a CO_2-optimized frame

slightly increases. Thus, for stronger concrete, the CO_2 optimization is more effective—that is, it results in (1) a smaller increase in cost and (2) a larger reduction in CO_2 footprint with respect to the cost optimization.

Contributions of Concrete and Reinforcement to Costs and CO_2 Footprints

Also considered were the contributions of the concrete and steel to the cost and to the CO_2-footprint, and the question of whether they were different for cost-optimized and CO_2-optimized frames. Fig. 6 represents the contribution of concrete and steel in the columns and beam to the total cost and CO_2 footprint for (1) the cost-optimized and (2) the CO_2-optimized frame, for $R_C = 0.8$, $R_{CO_2} = 0.078$, $f'_c = 40$ MPa, and $P = 6{,}000$ kN. Figs. 6(a and b) show that for the cost-optimized frame, the contribution of concrete to the total cost is greater than for the CO_2-optimized frame, while the opposite is true of steel. Figs. 6(b and d) show that the contribution of steel to the total CO_2 footprint is greater for the CO_2-optimized frame than for the cost-optimized frame, while the opposite is true of concrete. Figs. 6(b and d) also show that most of the contribution to the total CO_2 footprint is due to the concrete, rather than to the steel. Therefore, the statement that concrete has a lower CO_2 footprint than steel, as has been claimed [in Struble and Godfrey (2004) and Ashley and Lemay (2008), among others], is valid only for the footprint of concrete and new steel per unit volume; however, that statement is not applicable to the footprint inherent in the concrete and reinforcing steel used in RC structures.

Conclusions

An exploratory study was presented with the goal of assessing the potential of optimizing RC design for sustainability with respect to CO_2 emissions. The optimization with respect to the CO_2 footprint results in an increase in the relative amount of steel within the members' cross sections; however, the requisite ductility is ensured via constraints specified in the optimization process. The reduction of the CO_2 footprint achieved by optimizing the design to achieve minimum carbon emissions, as opposed to optimizing the design to achieve minimum cost, is of the order of 5% to 15%, depending upon the parameter values being assumed. That reduction can be smaller for low-rise structures and other structures with predominantly tension-controlled members. However, for structures whose members experience predominantly large compressive forces, such as high-rise buildings, the reduction can be significant; this also may be true of certain prestressed or poststressed concrete members. Additional research aimed at ascertaining the extent to which this is the case is warranted.

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