

# Optimization of Reinforced Concrete Structures by Simulated Annealing

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## 1. Introduction

Early attempts of optimised structural designs go back to the 1600s, when Leonardo da Vinci and Galileo conducted tests of models and full-scale structures [1]. A 1994's review of structural optimization can be found in the study by Cohn and Dinovitzer [2], who pointed out that there was a gap between theoretical studies and the practical application in practice. They also noted the short number of studies that concentrated on concrete structures. A review of structural concrete optimization can be found in the 1998's study by Sarma and Adeli [3]. The methods of structural optimization may be classified into two broad groups: exact methods and heuristic methods. The exact methods are the traditional approach. They are based on the calculation of optimal solutions following iterative techniques of linear programming [4,5]. The second main group comprises the heuristic methods, whose recent development is linked to the evolution of artificial intelligence procedures. This group includes a broad number of search algorithms [6-9], such as genetic algorithms, simulated annealing, threshold accepting, tabu search, ant colonies, etc. These methods have been successful in areas different to structural engineering [10]. They consist of simple algorithms, but require a great computational effort, since they include a large number of iterations in which the objective function is evaluated and the structural restrictions are checked.

Among the first studies of heuristic optimization applied to structures, the contributions of Jenkins [11] and of Rajeev and Krishnamoorthy [12] in the early 1990s are to be mentioned. Both authors applied genetic algorithms to the optimization of the weight of steel structures. As regards RC structures, early applications in 1997 include the work of Coello et al [13], who applied genetic algorithms to the economic optimization of RC beams. Recently, there has been a number of RC applications [14-16], which optimize RC beams and building frames by genetic algorithms. Also recently, our research group has applied simulated annealing and threshold acceptance to the optimization of walls, frame bridges and building frames [17-20]. However, despite advances on structural concrete optimization, present design-office practice of concrete structures is much conditioned by the experience of structural engineers. Most procedures are based on the adoption of cross-section dimensions and material grades based on sanctioned common practice. Once the structure is defined, it follows the analysis of stress resultants and the computation of passive and active

Source: Simulated Annealing, Book edited by: Cher Ming Tan, ISBN 978-953-7619-07-7, pp. 420, February 2008, I-Tech Education and Publishing, Vienna, Austria

reinforcement that satisfy the limit states prescribed by concrete codes. Should the dimensions or material grades be insufficient, the structure is redefined on a trial and error basis. Such process leads to safe designs, but the economy of the concrete structures is, therefore, very much linked to the experience of the structural designer.

The structures object of this work are walls, portal frames and box frames which are usually built of RC in road construction and RC frames widely used in building construction. RC earth retaining walls are generally designed with a thickness at the base of 1/10 of the height of the wall and a footing width of 0.50-0.70 the height of the wall. Box and portal frames are used with spans between 3.00 and 20.00 m for solving the intersection of transverse hydraulic or traffic courses with the main upper road. Box frames are preferred when there is a low bearing strength terrain or when there is a risk of scour due to flooding. The depth of the top and bottom slab is typically designed between 1/10 to 1/15 of the horizontal free span; and the depth of the walls is typically designed between 1/12 of the vertical free span and the depth of the slabs. Building frames have typical horizontal beams of 5.00 to 10.00 m of horizontal span that sustain the vertical loads of the floors and transfer them to vertical columns of height between 3.00 to 5.00 m. Moderate horizontal loads are usually included in the design, but high levels of horizontal loading are transferred to adjacent shear walls. The structures here analyzed are calculated to sustain the loads prescribed by the codes and have to satisfy all the limit states required as an RC structure. The method followed in this work has consisted first in the development of evaluation computer modules where dimensions, materials and steel reinforcement have been taken as design variables. These modules compute the cost of a solution and check all the relevant structural limit states. Simulated annealing is then used to search the solution space.

## 2. Simulated annealing optimization procedure

### 2.1 Problem definition

The structural concrete design problem that is put forward in the present study consists of an economic optimization. It deals with the minimization of the objective function  $F$  of expression (1), satisfying also the structural constraints of expressions (2).

$$F(x_1, x_2, \dots, x_n) = \sum_{i=1, r} p_i * m_i(x_1, x_2, \dots, x_n) \quad (1)$$

$$g_j(x_1, x_2, \dots, x_n) \leq 0 \quad (2)$$

Note that the objective function in expression (1) is the sum of unit prices multiplied by the measurements of the construction units (concrete, steel, formwork, etc). And that the constraints in expression (2) are all the service and ultimate limit states that the structure must satisfy. Unit prices considered are given in Table 1 and 2.

### 2.2 Simulated annealing procedure

The search method used in this study is the simulated annealing (SA henceforth), that was originally proposed by Kirkpatrick et al. [21] for the design of electronic circuits. The SA algorithm is based on the analogy of crystal formation from masses melted at high temperature and let cool slowly. At high temperatures, configurations of greater energy

Unit	Cost (€)
kg of steel (B-500S)	0.58
m <sup>2</sup> of lower slab formwork	18.03
m <sup>2</sup> of wall formwork	18.63
m <sup>2</sup> of upper slab formwork	30.65
m <sup>3</sup> of scaffolding	6.01
m <sup>3</sup> of lower slab concrete (labour)	5.41
m <sup>3</sup> of wall concrete (labour)	9.02
m <sup>3</sup> of upper slab concrete (labour)	7.21
m <sup>3</sup> of concrete pump rent	6.01
m <sup>3</sup> of concrete HA-25	48.24
m <sup>3</sup> of concrete HA-30	49.38
m <sup>3</sup> of concrete HA-35	53.90
m <sup>3</sup> of concrete HA-40	59.00
m <sup>3</sup> of concrete HA-45	63.80
m <sup>3</sup> of concrete HA-50	68.61
m <sup>3</sup> of earth removal	3.01
m <sup>3</sup> of earth fill-in	4.81

Table 1. Basic prices of the cost function for the road structures.

Unit	Cost (€)
kg of steel (B-500S)	1.30
m <sup>2</sup> of beams formwork	25.05
m <sup>2</sup> of columns formwork	22.75
m <sup>2</sup> of beams scaffolding	38.89
m <sup>3</sup> of concrete HA-25	78.40
m <sup>3</sup> of concrete HA-30	82.79
m <sup>3</sup> of concrete HA-35	98.47
m <sup>3</sup> of concrete HA-40	105.93
m <sup>3</sup> of concrete HA-45	112.13
m <sup>3</sup> of concrete HA-50	118.60

Table 2. Basic prices of the cost function for the building frames.

than previous ones may randomly form, but, as the mass cools, the probability of higher energy configurations forming decreases. The process is governed by Boltzmann expression  $\exp(-\Delta E/T)$ , where  $\Delta E$  is the increment of energy of the new configuration and  $T$  is the temperature. The present algorithm starts with a feasible solution randomly generated and a high initial temperature. The present SA algorithm then modifies the initial working solution by a small random move of the values of the variables. The new current solution is evaluated in terms of cost. Lower cost solutions are accepted and greater cost solutions are only accepted when a 0 to 1 random number is smaller than the expression  $\exp(-\Delta E/T)$ , where  $\Delta E$  is the cost increment and  $T$  is the current temperature. The current solution is then checked against structural constraints and it is adopted as the new working solution when it is feasible, i.e. when it satisfies the structural constraints. On the other hand, the current solution is discarded when it does not satisfy the structural constraints. The procedure is

repeated many times, which give way to a trajectory of feasible solutions that start in the initial solution and ends up in the converged solution result. The initial temperature is decreased geometrically ( $T=kT$ ) by means of a coefficient of cooling  $k$ . A number of iterations called Markov chains is allowed at each step of temperature. The algorithm stops when the temperature is a small percentage of the initial temperature (typically 1% and 1-2 chains without improvements). The SA method is capable of surpassing local optima at high-medium temperatures and gradually converges as the temperature reduces to zero. The SA method requires calibration of the initial temperature, the length of the Markov chains and the cooling coefficient. Adopted values for the four examples of this study will be given below. The initial temperature was adjusted following the method proposed by Medina [22], which consists in choosing an initial value and checking whether the percentage of acceptances of higher energy solutions is between 10-30 percent. If the percentage is greater than 30%, the initial temperature is halved; and if it is smaller than 10%, the initial temperature is doubled. Computer runs were performed 9 times so as to obtain minimum, mean and standard deviation of the random results. Note that the algorithm is random in the initial solution and in the moves from one solution to the next in the trajectory and, hence, results are basically random. This random nature makes necessary to run several times the algorithm so as to obtain a statistical population of results.

### 3. Application to earth retaining walls

The problem defined in section 2.1 is firstly applied to earth retaining RC cantilever walls used in road construction. This type of structure has already been studied by the authors in Ref. 17, which gives a detailed account of the analysis and optimization of this type of walls, while the present section gives an outline and two additional examples. Fig.1 shows the 22 design variables presently considered for the modelling of the walls. They include 4 geometrical variables (the thickness of the stem and 3 dimensions for the footing), 4 concrete and steel grades (stem and footing) and 14 variables for the definition of steel reinforcement, which includes both areas of reinforcement and bar lengths. Variables are continuous except for material grades which are discrete. The modelling of the reinforcement in concrete structures is very important. It has to be detailed enough to cover the variation of structural stress resultants, but not too complex in order to maintain a certain degree of simplicity and practicability. Note that the present arrangement includes three vertical bars for the main tension reinforcement in the kerb ( $A_1$  to  $A_3$  in Fig. 1), tension top and bottom reinforcement bars in the footing ( $A_9$  and  $A_8$ ) and stirrups in the footing and the bottom part of the kerb ( $A_{11}$  and  $A_7$ ). The remaining bars are basically minimum amounts of reinforcement for shrinkage and thermal effects. Apart from the design variables, a total of 17 parameters are considered for the complete definition of the problem. The most relevant parameters are the total height of the wall  $H$  (stem plus footing, see Fig.1), the top slope of the fill and the acting top uniform distributed load, the internal friction angle of the fill  $\phi$ , the permissible ground stress and the partial coefficients of safety. Structural constraints considered followed a standard analysis by Calavera [23], which includes checks against sliding, overturning, ground stresses and service-ultimate limit states of flexure and shear of different cross-sections of the wall and the footing. No vertical inclination of the earth pressure was considered. Additionally, a constraint of deflection at the top of  $1/150$  of the height of the stem was also considered.

The simulated annealing algorithm was programmed in Visual Basic 6.3 with an Excel input/output interface. Typical runs were 21 minutes in a Pentium IV of 2.41 GHz. The calibration of the SA recommended Markov chains of 1000 iterations and a cooling

coefficient of 0.80. As regards the type of moves, the most efficient move found consisted of random variation of 14 of the 22 design variables. Fig. 2 shows a typical cost evolution by the SA algorithm. Table 3 gives the details of parameters for the two walls analysed of 5.20 and 9.20 m of total height ( $H$  in Fig.1). Table 4 details the design results of the SA analysis for the two walls. The total cost of the walls is 505.06 and 1536.47 euros/m. Results indicate that the inclusion of a limit on deflections of  $1/150$  of the height of the stem is crucial, since otherwise the slenderness of the stem goes up to  $1/24$  and deflections are as high as  $1/40$  of the height of the stem. Should the top deflection be limited to  $1/150$ , the slenderness goes down to  $1/11.4$  and  $1/9.4$ , which is quite similar to the standard  $1/10$  adopted in practice by many practitioners.

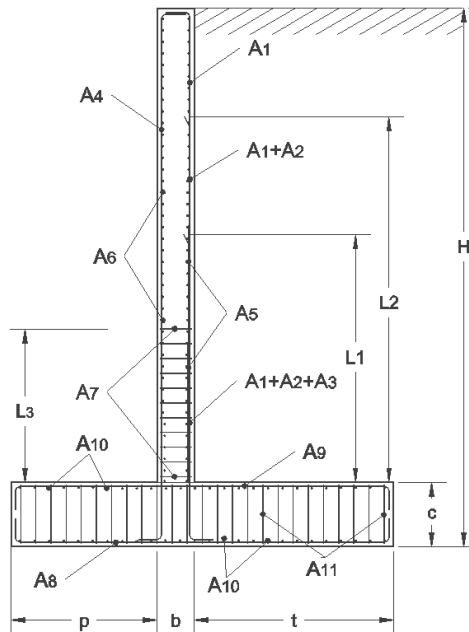


Figure 1. Variables of earth retaining walls for case study 1.

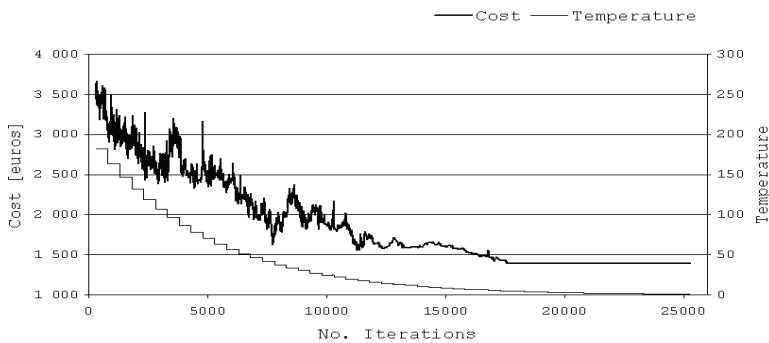


Figure 2. Typical cost evolution of SA algorithm.

Top slope of the fill	0
Uniform distributed load on top surface	10 kN/m <sup>2</sup>
Specific weight of the fill	20 kN/m <sup>3</sup>
Internal friction angle of the fill	30°
Inclination of the earth pressure	0°
Ground friction coefficient	0.577
Permissible ground stress	0.3 MPa
Overturning safety coefficient	1.8
Sliding safety coefficient	1.5
Execution type of control	Normal
ULS safety coefficient of concrete	1.50
ULS safety coefficient of steel	1.15
Max.displacement of the kerb/height of kerb	150
EHE ambient type	Ila

Table 3. Parameters of the reported walls (total height 5.20 and 9.20 m)

Variable	H = 5.2 m	H = 9.2 m
b	0.43 m	0.91 m
p	0.30 m	0.67 m
t	1.34 m	2.10 m
c	0.30 m	1.07 m
fck,ste	35 MPa	40 MPa
fck,foo	25 MPa	25 MPa
fyk,ste	500 MPa	500 MPa
fyk,foo	500 MPa	500 MPa
A1	9.26 cm <sup>2</sup>	22.35 cm <sup>2</sup>
A2	3.04 cm <sup>2</sup>	2.89 cm <sup>2</sup>
A3	4.49 cm <sup>2</sup>	10.97 cm <sup>2</sup>
A4	1.95 cm <sup>2</sup>	2.72 cm <sup>2</sup>
A5	4.65 cm <sup>2</sup>	9.82 cm <sup>2</sup>
A6	9.21 cm <sup>2</sup>	19.42 cm <sup>2</sup>
A7	0.00 cm <sup>2</sup>	0.00 cm <sup>2</sup>
A8	8.52 cm <sup>2</sup>	18.94 cm <sup>2</sup>
A9	12.05 cm <sup>2</sup>	19.44 cm <sup>2</sup>
A10	6.06 cm <sup>2</sup>	12.75 cm <sup>2</sup>
A11	0.00 cm <sup>2</sup>	0.00 cm <sup>2</sup>
L1	0.97 m	2.80 m
L2	0.54 m	0.98 m
L3	0.00 m	0.00 m

Table 4. Summary of best walls (total height 5.20 and 9.20 m)

#### 4. Application to road portal frames

The second example studied relates to portal RC frames used in road construction [24]. Fig. 3 shows the 28 variables considered in this analysis. Variables include 5 geometrical values:

the depth of the walls, the depth of the top slab and the depth of the footing, plus 2 dimensions for the size of the base of the footing; 3 different grades of concrete for the 3 types of elements; 3 types of elements; and 20 types of reinforcement bars following a standard setup. The reinforcement setup includes 2 variables for the basic negative bending moments,  $A_1$  and  $A_8$ , plus a corner additional bar of area  $A_7$ . The positive bending moment in the top slab and the wall is covered by bars  $A_2$  and  $A_9$ . And positive and bending moments in the footing are covered by bars  $A_{15}$  and  $A_{16}$ . Additionally several other bars cover shear in the different parts of the frame. All variables are discrete in this analysis. The total number of parameters is 16, the most important of which are the horizontal free span, the vertical free span, the earth cover, the permissible bearing stress and the partial coefficients of safety. Structural constraints considered followed standard provisions for Spanish design of this type of structure [25,26], that include checks of the service and ultimate limit states of flexure and shear for the stress envelopes due to the traffic loads and the earth fill. Traffic loads considered are a uniform distributed load of  $4 \text{ kN/m}^2$  and a heavy vehicle of  $600 \text{ kN}$ . Stress resultants and reactions were calculated by an external finite element program using a 2-D mesh with 30 bars and 31 sections (out of plane bending moments had to be assumed as a practical one fifth proportion of in plane bending moments). Deflections were limited to  $1/250$  of the free span for the quasi-permanent combination. Fatigue of concrete and steel was not considered since this ultimate limit state is rarely checked in road structures.

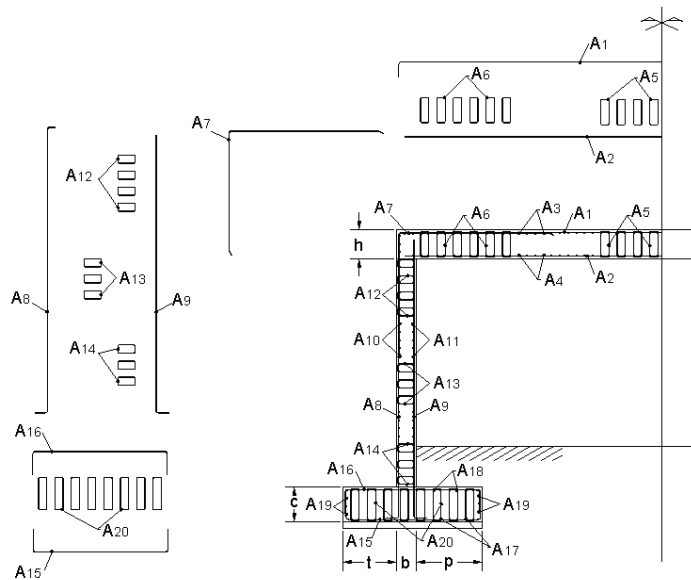


Figure 3. Variables for the RC portal frame.

The SA algorithm was programmed in Visual Basic 6.3. Typical runs were 10.76 hours in an AMD Athlon processor of 1.49 GHz. In this case, the calibration recommended Markov chains of 375 iterations and a cooling coefficient of 0.70, the total amount of iterations being about 7500. The most efficient move found consisted of random variation of 4 of the 28

variables of the problem. Table 5 details the main results of the SA analysis for two portal frames of 10.00 and 15.00 m of horizontal free span, 6.00 m of vertical free span and 0.10 m of asphalt cover (additional parameters are 0.25 MPa permissible bearing stress, specific weight of the fill of 20 kN/m<sup>3</sup>, 30 degrees internal friction angle of the fill and partial safety coefficients of 1.60 for loading and 1.50-1.15 for concrete-steel as materials). The depth of the top slab is only 0.375 m for the 10.00 m span, which means a very slender span/depth ratio of 26.67. The cost of this solution is 2619 euros/m. This best solution was then checked by hand calculations against fatigue of structural concrete. The loading considered was a 468 kN heavy vehicle prescribed for fatigue by the Spanish loading code for bridges [25]. It was found that the solution did not comply with Eurocode 2 limitations for fatigue [27]. Hence, it was concluded that this rarely checked ULS should be included in future works of optimization dealing with road structures.

Variables	L=10.00 m	L=15.00 m
Slab depth	0.375 m	0.450 m
Wall thickness	0.400 m	0.475 m
Footiing depth	0.400 m	0.400 m
Footiing toe	0.950 m	0.650 m
Footiing heel	0.750 m	1.650 m
Footiing conc.	HA-25	HA-30
Wall concrete	HA-25	HA-25
Slab concrete	HA-25	HA-25
A <sub>1</sub>	15ø12/m	15ø12/m
A <sub>2</sub>	10ø20/m	10ø25/m
A <sub>6</sub>	12.06 cm <sup>2</sup> /m	12.56 cm <sup>2</sup> /m
A <sub>7</sub>	15ø12/m	12ø20/m
A <sub>8</sub>	8ø16/m	10ø16/m
A <sub>9</sub>	12ø8/m	6ø16/m
A <sub>15</sub>	10ø16/m	12ø12/m
A <sub>16</sub>	12ø10/m	12ø8/m
A <sub>20</sub>	9.05 cm <sup>2</sup> /m	11.30 cm <sup>2</sup> /m

Table 5. Summary of best portal frames.

## 5. Application to road box RC frames

The third example studied relates to box RC frames used in road construction. A detailed account of the modelling of frames and the SA-TA proposed algorithms can be found in the study by Perea et al [18]. The present section gives an outline of the analysis and optimization procedures and an additional example. Fig. 5 shows the 44 variables considered in this analysis for the modelling of the frames. Variables include 2 geometrical values: the depth of the walls and slabs; 2 different grades of concrete for the 2 types of elements; and 40 types of reinforcement bars and bar lengths following a standard setup.



The reinforcement setup includes 3 variables for the basic negative bending moments,  $A_{14}$ ,  $A_8$  and  $A_{12}$ , plus two corner additional bars of area  $A_6$  and  $A_{12}$ . The positive bending moment in the top slab, the bottom slab and the wall is covered by pairs of bars  $A_2$ - $A_3$ ,  $A_{13}$ - $A_{15}$  and  $A_7$ - $A_9$ . Additionally several other bars cover shear in the different parts of the frame. All variables are again discrete in this analysis. The most important parameters are the horizontal free span, the vertical free span, the earth cover, the ballast coefficient of the bearing and the partial coefficients of safety. Structural restrictions considered followed standard provisions similar to those of portal frames. However, this time the ULS of fatigue was included following the conclusions from the previous section. Stress resultants and reactions were calculated by an external finite element program using a 2-D mesh with 40 bars and 40 sections.

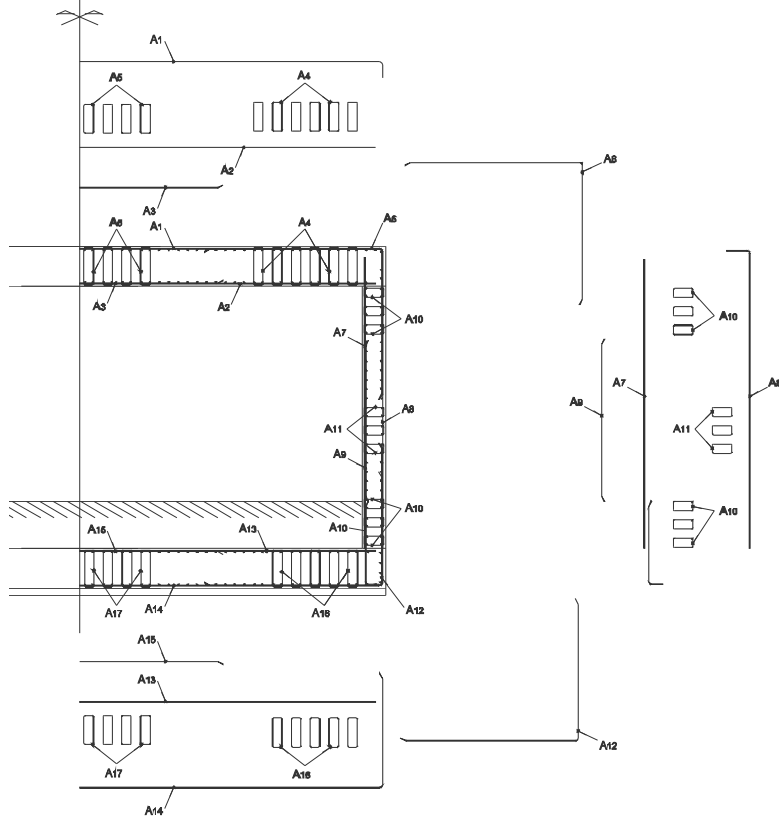


Figure 5. Variables for the RC box frame.

The SA algorithm was programmed this time in Compaq Visual Fortran Professional 6.6.0. Typical runs reduced to 20 seconds in a Pentium IV of 2.4 GHz. In this case, the calibration recommended Markov chains of 500 iterations and a cooling coefficient of 0.90. The most efficient move found was random variation of 9 variables of the 44 of the problem. Fig. 6 details the main results of the SA analysis for a box frame of 13.00 m of horizontal free span, 6.17 m of vertical free span and 1.50 m of earth cover (additional parameters are 10 MN/m<sup>3</sup>

ballast coefficient, specific weight of the fill of  $20 \text{ kN/m}^3$ , 30 degrees internal friction angle of the fill and partial safety coefficients of 1.50 for loading and 1.50-1.55 for concrete-steel as materials). The cost of this solution is 4478 euros/m. The depth of the slabs is 0.65 m of C30 (30 MPa of characteristic strength), which represents a slender span/depth ratio of 20. And the depth of the wall is 0.50 m in C45, which represents a vertical span/depth ratio of 12.34. The overall ratio of reinforcement in the top slab is  $160 \text{ kg/m}^3$ . It may, hence, be concluded that results of the optimization search tend to slender and highly reinforced structural box frames. As regards deflections and fatigue limit states, their inclusion has shown to be crucial. Neglecting both limit states leads to a 7.9% more economical solution, but obviously unsafe. It is important to note that fatigue checks are usually considered in railways designs but, on the other hand, they are commonly neglected in road structures design and, as it has been shown, this may lead to unsafe designs.

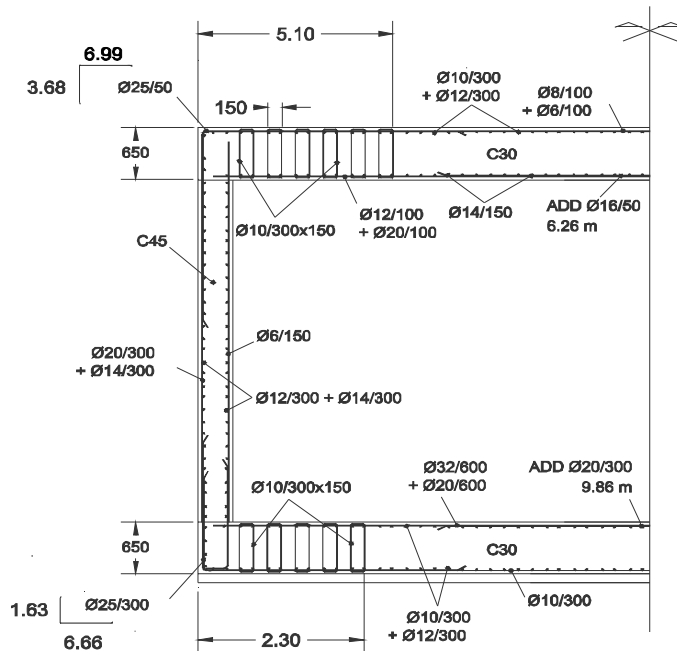


Figure 6. Optimized design of RC box frame.

## 6. Application to RC building frames

The last example studied relates to RC frames commonly used in building construction. A detailed account of the modelling of frames and the SA proposed algorithms is done in the study by Payá et al [19]. The present section gives an outline of the analysis and optimization procedures and an additional example. The RC frame studied here is the symmetrical frame of 2 bays and 5 floors shown in Fig. 7. This example has 95 variables, including 5 material types of concrete, 30 cross-section dimensions and 60 passive reinforcement bars following a standard setup in columns and beams. Fig. 8 shows a typical

longitudinal reinforcement setup of the beams of the structure. It includes a basic top and bottom bars and positive and negative extra reinforcements of a standard length. Variables for beam stirrups include 3 zones of left, central and right positions of transverse reinforcement. Longitudinal reinforcement of columns includes 330 possible values and it varies from a minimum of  $4\phi 12$  to a maximum of  $34\phi 25$  whereas transverse reinforcement of columns includes 21 possible values. The most important parameters are the horizontal spans of the bays, the vertical height of the columns, the vertical and horizontal loads considered and the partial coefficients of safety. Structural restrictions considered followed standard provisions for Spanish design of this type of structure [26,28], that include checks of the service and ultimate limit states of flexure, shear and instability for the stress envelopes due to the vertical loads and the horizontal wind loads. Vertical loads amount to a total uniform distributed load of 35 kN/m (7.00 kN/m<sup>2</sup> of self weight plus life load and 5.00 m of spacing between parallel frames). As regards wind loads, they amount to a total uniform distributed load of 4.5 kN/m. Stress resultants and reactions were calculated by an internal matrix method program using a 2-D mesh. Deflections were limited to 1/250 of the horizontal span for the total load and to 1/400 for the active deflection; which is the part of the deflection measured after construction of the elements that can be damaged due to vertical displacements.

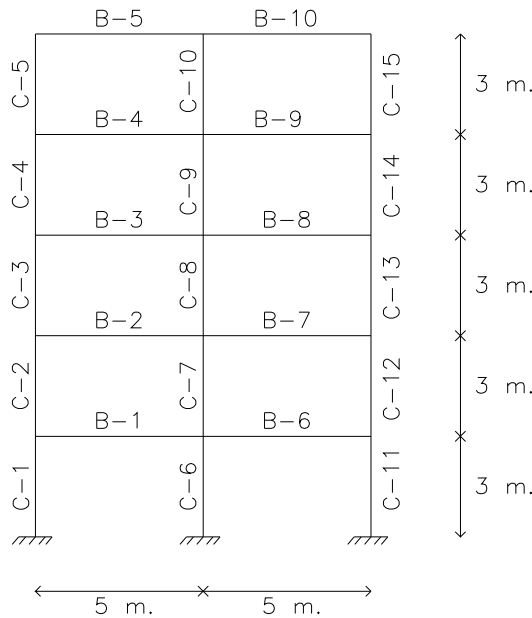


Figure 7. Typical RC building frame of 2 bays and 5 floors.

The SA algorithm was programmed in Compaq Visual Fortran Professional 6.6.0. Typical runs took a time of 97 minutes in a Pentium IV of 3.2 GHz. In this case, the calibration recommended Markov chains of 105000 iterations, a cooling coefficient of 0.80 and two

Markov chains without improvement as stop criterion. The most efficient move found was random variation of 3 or up to 3 variables of the 95 of the problem. Tables 3, 4 and 5 detail the main results of the best SA analysis for the building frame of Fig. 7. The cost of this solution is 4458.08 euros. Concrete is HA-45 in the whole structure. The restrictions that guided the design were the ultimate limit states of flexure and shear in beams and instability in columns, and the service limit state of deflections in beams.

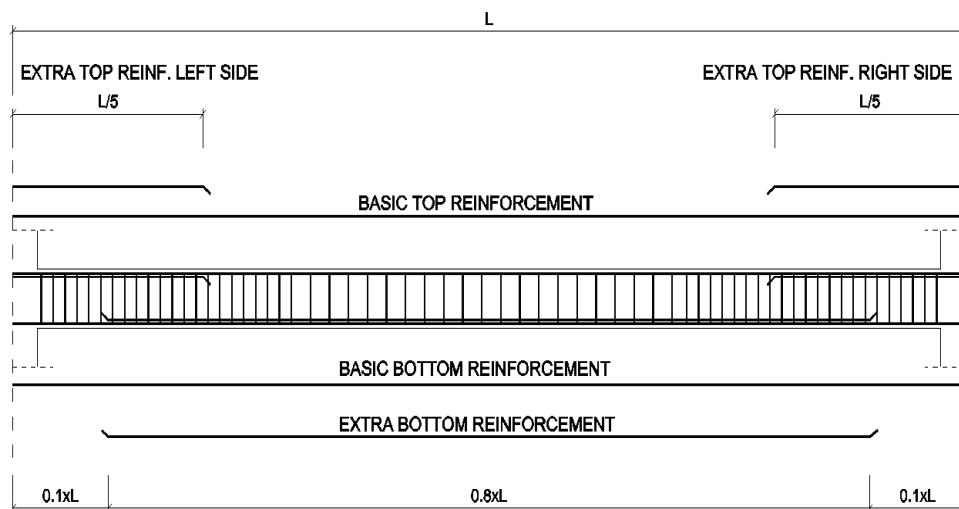


Figure 8. Typical longitudinal reinforcement bars of the beams of RC building frames.

Beam	Dimensions (cm)		Top reinforcement		
	Depth	Width	Base	Extra Left	Extra Right
B-1	0.48	0.20	2 $\phi$ 20	-	1 $\phi$ 25
B-2	0.50	0.20	2 $\phi$ 16	1 $\phi$ 10	2 $\phi$ 20
B-3	0.50	0.20	2 $\phi$ 10	1 $\phi$ 20	2 $\phi$ 25
B-4	0.51	0.21	2 $\phi$ 10	2 $\phi$ 12	3 $\phi$ 16
B-5	0.54	0.22	2 $\phi$ 10	1 $\phi$ 16	2 $\phi$ 25

Table 3. Beam results of the SA: dimensions and top reinforcement

Beam	Bottom reinf.		Shear reinforcement		
	Base	Extra	Left	Span	Right
B-1	3 $\phi$ 12	2 $\phi$ 10	$\phi$ 8/25	$\phi$ 8/30	$\phi$ 6/10
B-2	3 $\phi$ 12	2 $\phi$ 10	$\phi$ 8/25	$\phi$ 8/30	$\phi$ 8/20
B-3	2 $\phi$ 12	1 $\phi$ 20	$\phi$ 6/15	$\phi$ 6/15	$\phi$ 10/30
B-4	4 $\phi$ 10	1 $\phi$ 16	$\phi$ 6/15	$\phi$ 8/30	$\phi$ 8/20
B-5	4 $\phi$ 10	2 $\phi$ 10	$\phi$ 8/30	$\phi$ 8/30	$\phi$ 6/15

Table 4. Beam results of the SA: bottom and shear reinforcement

Column	Dimensions (cm)		Longitudinal Reinforcement			Ties
	a	b	Corners	Side a	Side b	
C-1	0.25	0.25	4 $\phi$ 12	-	-	Ø6/15
C-2	0.25	0.25	4 $\phi$ 16	-	-	Ø6/15
C-3	0.25	0.25	4 $\phi$ 12	2 $\phi$ 12	-	Ø6/15
C-4	0.25	0.25	4 $\phi$ 12	-	2 $\phi$ 12	Ø6/15
C-5	0.25	0.25	4 $\phi$ 16	-	-	Ø6/15
C-6	0.25	0.45	4 $\phi$ 12	-	2 $\phi$ 12	Ø6/15
C-7	0.25	0.40	4 $\phi$ 12	-	-	Ø6/15
C-8	0.25	0.40	4 $\phi$ 12	-	-	Ø6/15
C-9	0.25	0.35	4 $\phi$ 12	-	-	Ø6/15
C-10	0.25	0.30	4 $\phi$ 12	-	-	Ø6/15

Table 5. Column results of the SA for Column results of the SA (columns "b" side is parallel to beams axis).

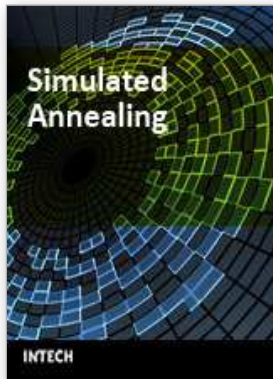
## 6. Conclusions

As regards the SA procedure, it has proved an efficient search algorithm for the 4 case studies of walls, portal and box frames used in road construction and building frames. The study of earth retaining walls optimization shows that the inclusion of a limit of 1/150 on the deflection of the top of the walls is needed. Otherwise, results of the SA optimization are excessively deformable. Results of the optimization of portal road frames indicated the need of including the rarely checked ULS of fatigue in the list of structural restrictions for the optimization of road structures. The study of road box frames shows the importance of the inclusion of the SLS of deflections and the ULS of fatigue. The SA optimization of the 13 m free horizontal span box frame results in a slender and highly reinforced top slab. Results of the optimization of the building frame indicate that instability in columns and flexure, shear and deflections in beams are the main restrictions that condition its design.

## 7. References

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## **Simulated Annealing**

Edited by Cher Ming Tan

ISBN 978-953-7619-07-7

Hard cover, 420 pages

**Publisher** InTech

**Published online** 01, September, 2008

**Published in print edition** September, 2008

This book provides the readers with the knowledge of Simulated Annealing and its vast applications in the various branches of engineering. We encourage readers to explore the application of Simulated Annealing in their work for the task of optimization.

### **How to reference**

In order to correctly reference this scholarly work, feel free to copy and paste the following:

F. González-Vidosa, V. Yepes, J. Alcalá, M. Carrera, C. Perea and I. Payá-Zaforteza (2008). Optimization of Reinforced Concrete Structures by Simulated Annealing, Simulated Annealing, Cher Ming Tan (Ed.), ISBN: 978-953-7619-07-7, InTech, Available from:

[http://www.intechopen.com/books/simulated\\_annealing/optimization\\_of\\_reinforced\\_concrete\\_structures\\_by\\_simulated\\_annealing](http://www.intechopen.com/books/simulated_annealing/optimization_of_reinforced_concrete_structures_by_simulated_annealing)

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