IOP Conference Series: Materials Science and Engineering

PAPER • OPEN ACCESS

Structural reliability of RC elements with electric arc furnace slag as recycled aggregates

To cite this article: M A Zanini *et al* 2019 *IOP Conf. Ser.: Mater. Sci. Eng.* **615** 012098

IOP Conf. Series: Materials Science and Engineering **615** (2019) 012098 doi:10.1088/1757-899X/615/1/012098

Structural reliability of RC elements with electric arc furnace slag as recycled aggregates

M A Zanini[1], E De Stefani[1], F Faleschini[1,2*] and C Pellegrino[1]

[1] Department of Civil, Environmental and Architectural Engineering, University of Padova, Padova, Italy.
[2] Department of Industrial Engineering, University of Padova, Padova, Italy.

[*]Corresponding Author's E-mail: flora.faleschini@unipd.it

Abstract. Sustainable development issues have become very important in the recent years, and the increased sensibility on sustainability of civil engineering structures are inducing to set the attention in recycling operations, also in the context of building materials. Concrete is the most used construction material in the world and its production requires a considerable demand of energy and raw materials, inducing significant gases emissions and huge quantity of natural materials and non-renewable resources exploitation. Electric Arc Furnace Concrete (EAF concrete) is a kind of recycled concrete in which its aggregates are the by-product of steel and iron producing processes, the so called Electric Arc Furnace (EAF) slag: while its environmental positive effect is easily understandable and intuitive, its effective usefulness and reliability in the building field is more difficult to accept. From the literature data, it seems that EAF concrete presents higher values of compressive and tensile strengths and modulus of elasticity than the natural concrete ones, and this might induces one to think that a higher structural reliability could be expected. However, it is worth recalling that this material is characterized by also a higher self-weight, thus, increasing the dead-load values. Hence, in this study, a reliability-based analysis of the structural load-carrying capacity of EAF reinforced concrete (RC) elements is carried out, through a fully probabilistic approach based on Monte Carlo simulations improved by Latin Hypercube method (LHS). The analysed RC elements are designed according to the Italian current construction code, and then, considering all the uncertainties included in the random variables definition, failure probability and relative reliability index are estimated and compared to the reliability of the same RC elements with natural concrete. Results indicate that, for the analysed cases, when EAF slag is used as coarse recycled aggregate, it is possible to guarantee the same reliability level than in RC elements realised with natural concrete.

1. Introduction

Sustainable development issues have become very important in the recent years, and the increased sensibility on sustainability of civil engineering structures are inducing to set the attention in recycling operations, also in the context of building materials.

Electric Arc Furnace Concrete (EAF concrete) is a kind of recycled concrete in which aggregates are the by-product of steel and iron producing processes, so called Electric Arc Furnace (EAF) slag. From literature data, it seems that EAF concrete presents higher values of compressive and tensile strengths and modulus of elasticity than natural concrete ones, and this might induce one to think that also a higher structural reliability could be expected. However, it is characterized by also a higher self-weight, thus, increasing the dead-load values.

The purpose of this research is to give an estimation of the reliability and relative failure probability of reinforced concrete elements with recycled aggregates, and to allow a proper comparison among recycled and conventional concretes.

Analysis has been conducted in simple supported beams, designed considering different section measures, concrete strength classes and different accidental load conditions, following the Italian current construction code.

Matlab software has been used to apply a fully probabilistic approach based on Monte Carlo simulations, and the sampling phase has been improved by LHS method to increase the code efficiency.

2. Designed beams

The analysis is set-up to produce a parametric estimation of Beta index for different RC elements geometries, loads and material properties.

All beams are simply supported, with 5 m span and subjected to uniformly distributed loads. Five different section geometries are evaluated: four of them are 300 mm width, and 300, 400, 500, and 600 mm height, respectively; the last one is 600 mm width and 300 mm height. A concrete cover of 30 mm has been fixed for every section.

Dead loads are the same for all elements, with exception of self-weight that is obviously calculated by considering the geometries of every beam. Four different live load categories are instead considered: according to the Italian Construction Rules (Tab. 1 3.1.II, [1]), accidental load values of 2, 3, 4, 5 and 6 kN/m^2 are evaluated.

Different strength classes of concrete have been chosen: C30/37, C40/50 and C50/60. Every strength class contains the conventional concrete and three EAF concretes with different physical and mechanical properties which are only considered for this reliability analysis.

Longitudinal reinforcement steel area has been designed for the natural concrete, following the semi-probabilistic method of the Italian code [1] for flexure and using B450C steel. Parabola-rectangle and bilinear without hardening stress strain laws for concrete and steel have been used.

3. Recycled concrete

The three classes of EAF concrete with different mixture characteristics are evaluated in the following points:

- Class 1: includes high strength concretes. They are characterized by water/cement ratio lower than 0.5 and aggregates substitution ratio with EAF ones of 50% over the total, that means the EAF slag content is present only as coarse aggregates;
- Class 2: includes medium-low strength concretes. They have a water/cement ratio greater than 0.5 and the same substitution percentage of the previous class;
- Class 3: includes low strengths concretes, with a water/cement ratio greater than 0.5 and a substitution ratio of EAF aggregates greater than 50%, so the EAF slag content is made up of both coarse and fine materials.

Probability distribution function of their cylindric compressive strengths and self-weight are reported in tables 1 and 2. It must be said that EAF concrete of Class 3 for C50/60 has not been considered because it is physically very difficult to produce.

4. Random variables

Reliability of RC elements depends on several random variables and on their probability distribution functions, this parameter is summarized in table 3.

Regarding the variability of the loads, it would include the variability of load values and variability of load presence, but this last one has not been considered, since treating only simple elements static schemes becomes irrelevant.

Table 1. Statistical parameters for concretes strengths [MPa] (c0 indicates natural concrete; c1, c2, c3 indicate class 1, 2 and 3 of EAF concretes).

Concrete	Mean	Std. Dev.	CoV	k = 5%
C30/37 - c0	35.91	3.59	0.10	30.00
C30/37 -c1	48.40	4.52	0.09	40.97
C30/37 -c2	47.90	7.46	0.16	35.62
C30/37 -c3	33.53	3.63	0.11	27.56
C40/50 - c0	47.87	4.79	0.10	40.00
C40/50 -c1	64.52	6.02	0.09	54.62
C40/50 -c2	63.85	9.95	0.16	47.49
C40/50 -c3	44.70	4.84	0.11	36.74
C50/60 - c0	56.51	3.96	0.07	50.00
C50/60 -c1	66.59	4.35	0.065	59.43
C50/60 -c2	67.48	7.36	0.109	55.37

Table 2. Statistical parameters for concretes self-weight.

Concrete	Mean	Std. Dev.	CoV
c0	25.00	0.75	0.030
c1	29.46	1.37	0.047
c2	29.85	3.02	0.101
c3	28.97	1.59	0.055

Table 3. Statistics of random variables.

Random Variable	Mean	St. Dev.	CoV	Probability dist.
Section geometries				
d, b, c	Nom. Value		0.05	Normal
Dead loads (kN/m^2)				
G_2	2.4	0.24	0.10	Normal
Live loads (kN/m^2) [2]				
Cat A,B (q = 2)	1.50	0.27	0.18	Gumbel
Cat C (q = 4)	3.00	0.54	0.18	Gumbel
Cat D (q = 5)	3.75	0.68	0.18	Gumbel
Cat E (q = 6)	4.50	0.81	0.18	Gumbel
Steel properties [3]				
f_y = 450 MPa	493.86	26.60	0.05	LogNormal
Es = 210 GPa	224.789	8.991	0.04	LogNormal
ε_u = 0.07	0.26	0.03	0.13	LogNormal

5. Reliability analysis

After beams design and random variables probability distribution function have been defined, the reliability analysis through Monte Carlo simulations is conducted to give an estimation of Beta index and its failure probability.

Monte Carlo approach belongs to the third level probabilistic methods, and from the practical point of view, consists of a series of experimental trials: for every trial, the involved random variables values are generated in accordance to their probability distribution function and then used to calculate the limit state function: if it is not satisfied, the generated random variables fall in the failure region and the indicator function takes value 1, otherwise it takes 0. After many trials, failure probability can be calculated considering the number of failure and the number of trials that have been done.

In this study, the limit state function is composed by the simple difference between resisting moment and acting moment of the beam section.

All the calculations have been done using Matlab software, and to improve the code efficiency, latin square method has been applied.

Latin square is defined like a square grid of sample positions, in which there must be only one value in every row and in every column. Latin hypercube is the same concept at more dimensions.

The method consists in a near-random sample generation of random variables, whose values go to complete a latin hypercube, that means subdivide the range of possible probability values in n parts, where n is latin hypercube dimension, and extract only one value contained in every part, with respect to the latin hypercube definition. Then, through inverse cumulative function of every random variable, sampled values are derived and they are applied in the simulation phase. In this manner, sampling is more uniform in the whole range of possibility and also allows a reduced computational cost.

Reliability analysis has been conducted following three phases: generation of acting moment probability distribution function, generation of resisting moment probability distribution function, and the failure rate and the relative beta index calculation.

5.1. Acting and resisting moments probability distribution functions generation

Computation of acting moment probability distribution function is done following an iteration procedure:

- for every iteration, LHS method is used, so n = 1000 values for every random variable are generated in according to their probability distribution function: for acting moment, the involved random variables are the section geometries and its dimensions, concrete self-weight and loads; for resisting moment, random variables are section geometries dimensions, and concrete and steel strengths: steel reinforcement area is assumed to be deterministic. Then, every random variable becomes a vector containing n generated values;
- hypothesizing deterministic values of interspace between beams and beam span (I = 4 and L = 5), n values of acting moment are easily calculated;
- n values of resisting moment are calculated with a simple equilibrium of section internal forces, in according to materials stress strain law;
- for every iteration, mean and standard deviation of calculated acting (and resisting) moments are computed, and they are compared to the previous iteration ones: if their differences are smaller than a tolerance value fixed to 0.0001, a good probability distribution is considered achieved and code goes out of the loop;
- with an appropriate Matlab tool, theoretical probability distribution functions to fit better with the decided behaviour of the generated acting and resisting moments probability distribution. Regarding acting one, two different situations are emerged: the best theoretical probability distribution function to fit the acting moment seems to be the normal one in the case with very low accidental load (Cat. A, B), and the Gumbel one in other cases. This is due to the different ratio between permanent and accidental loads values: they follow different probability distribution function, so when their ratio value is high, normal distribution of dead loads prevails on Gumbel distribution of accidental load. Resisting moment is instead clearly well-fitted by a normal distribution function;
- finally, the code computes the resisting moment and gives in output of the parameter of acting and resisting moments probability distribution function.

IOP Conf. Series: Materials Science and Engineering **615** (2019) 012098 doi:10.1088/1757-899X/615/1/012098

5.2. Failure rate and beta index calculation

With the probability distribution functions of acting and resisting moments, β reliability index is calculated as follows.

A deterministic vector M_s of increasing values of acting moment is assumed to discretize the problem. Known the probability distribution function of resisting moment, relative failure probability for each value of M_s is derived: this means, in practice, derive the cumulative distribution function Φ of the resisting moment.

Probability density of each value of M_s is given by acting moment probability distribution function, that can be indicated with ϕ if it is normal, like in the case of low accidental loads.

Failure probability and relative acting moment probability distribution function are represented in figure 1 for the case of category A/B of accidental load, where acting moment is normal, and conventional concrete, evaluated in the section 60 x 30.

Then, multiplied failure and density probability values corresponds each component of M_s, failure rate function λ_f is obtained (2), as showed in figure 2.

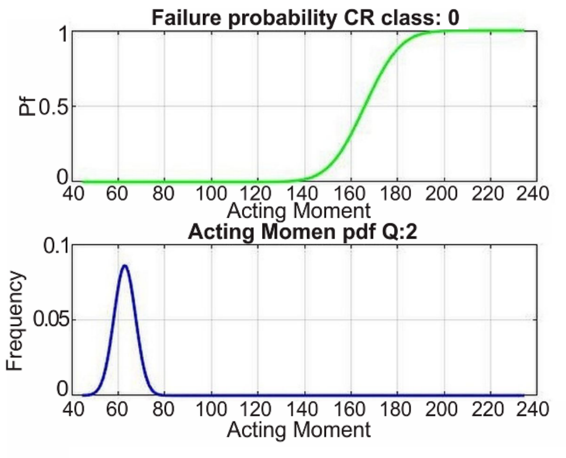

Figure 1. Failure probability and acting moment pdf for section 60 x 30, natural concrete and load category A/B.

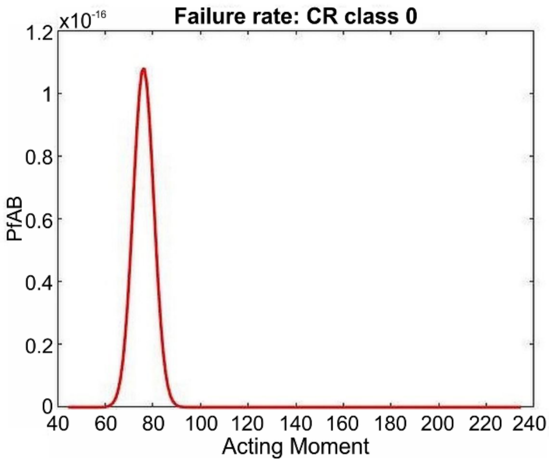

Figure 2. Failure rate function for section 60 x 30, natural concrete and load category A/B.

The area value inside the curve assumes the meaning of failure rate λ_f, and β reliability index, for definition, is computed by equation (3).

$$\lambda_f=\phi(M_s)\cdot\phi(M_s) \tag{2}$$

$$\beta= -\Phi^{-1}(\lambda_f) \tag{3}$$

The meaning of all the procedure can be summarized in the graph in figure 3, in which failure probability can be seen in the intersection area between the probability distribution functions of acting and resisting moments.

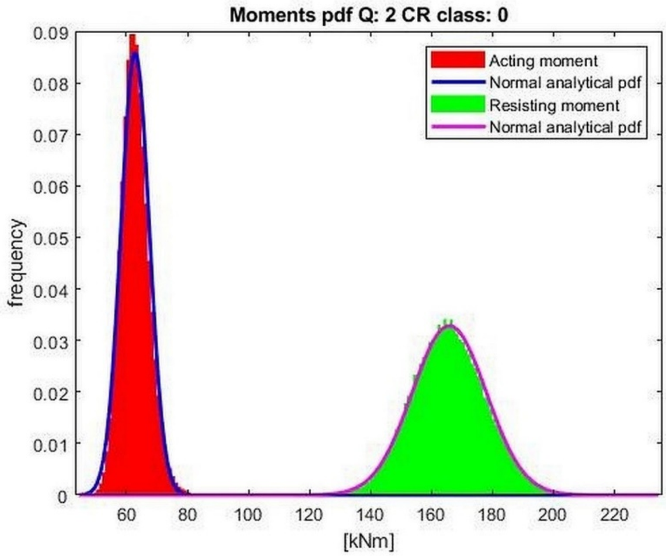

Figure 3. Acting and resisting moment pdf for section 60 x 30, natural concrete and load category A/B.

6. Reliability results

Table 4 displays acting and resisting moments probability distribution functions of section 60 x 30 resulted by the code. Recycled concretes have 15 - 20% higher mean of specific weight than the natural ones, and this shows in the 1.5 - 2% higher acting moment. Also, the standard deviation is a bit higher for recycled concrete, but not as much as the specific weight ones.

Table 4. Statistics of acting moment, M_A, and resisting moment, M_R for beam section 30 x 30 with different concretes and load categories [kNm].

BEAM	C30/37				C40/50				C50/60			
30x30	μ_{MA}	σ_{MA}	μ_{MR}	σ_{MR}	μ_{MA}	σ_{MA}	μ_{MR}	σ_{MR}	μ_{MA}	σ_{MA}	μ_{MR}	σ_{MR}
B_c0	55.88	4.55	137.33	10.21	55.85	4.55	141.04	10.41	55.88	4.56	142.80	10.52
B_c1	57.15	4.57	141.18	10.43	57.20	4.58	143.94	10.62	57.13	4.57	144.23	10.60
B_c2	57.19	4.64	140.87	10.56	57.23	4.64	143.70	10.63	57.21	4.63	144.27	10.64
B_c3	56.96	4.57	136.25	10.19	57.03	4.57	140.22	10.34	-	-	-	-
C_c0	74.65	7.41	204.11	15.65	74.64	7.41	198.43	14.81	74.63	7.41	186.19	13.66
C_c1	75.88	7.41	213.76	16.00	75.88	7.42	204.57	15.15	75.92	7.42	188.73	13.88
C_c2	76.00	7.47	213.00	16.45	76.00	7.47	204.08	15.30	76.04	7.46	188.81	13.97
C_c3	75.72	7.43	201.40	15.69	75.70	7.44	196.71	14.80	-	-	-	-
D_c0	83.99	9.03	222.26	16.97	84.05	9.03	231.19	17.46	84.00	9.03	217.82	16.09
D_c1	85.34	9.05	229.65	17.44	85.29	9.05	239.90	17.78	85.29	9.04	221.40	16.29
D_c2	85.37	9.08	229.00	17.62	85.37	9.08	239.21	18.16	85.36	9.09	221.51	16.42
D_c3	85.15	9.06	220.05	16.94	85.09	9.05	228.74	17.44	-	-	-	-
E_c0	93.36	10.60	249.71	19.34	93.39	10.58	258.04	19.66	93.36	10.58	236.48	17.50
E_c1	94.64	10.59	253.14	20.10	94.58	10.59	269.35	20.11	94.64	10.59	240.79	17.76
E_c2	94.75	10.61	252.70	20.11	94.78	10.62	268.44	20.53	94.77	10.61	240.92	17.92
E_c3	94.52	10.59	248.36	19.23	94.48	10.59	254.86	19.65	-	-	-	-

Resisting moment of EAF concretes is higher than the conventional ones, up to 4% in certain conditions, with their compressive strengths is 30% bigger, with the exception of EAF concrete of class 3. There is no substantial difference in the values of resisting moments.

It must be said that not all the analysed sections resulted in the same behaviour: it is possible to note that by β index values reported in tables 5, 6 and 7, sections 600 and 500 mm height shows the greatest reliability index for conventional concrete.

Table 5. Failure probability and reliability index for C30/37 concrete.

Beams	60x30		50x30		40x30		30x30		30x60	
C30/37	λ_f	β	λ_f	β	λ_f	β	λ_f	β	λ_f	β
B_c0	1.53E-15	7.89	5.46E-13	7.12	1.28E-15	7.91	1.56E-13	7.29	8.30E-13	7.06
B_c1	4.96E-15	7.74	1.55E-12	6.97	1.88E-15	7.86	8.09E-14	7.38	2.32E-12	6.92
B_c2	8.61E-15	7.67	2.11E-12	6.93	3.42E-15	7.79	1.95E-13	7.26	4.41E-12	6.82
B_c3	7.62E-15	7.69	2.16E-12	6.93	5.38E-15	7.73	6.26E-13	7.10	6.16E-12	6.78
C_c0	4.52E-08	5.34	9.73E-09	5.62	2.10E-08	5.48	4.10E-09	5.76	3.94E-08	5.37
C_c1	6.22E-08	5.29	1.04E-08	5.61	1.67E-08	5.52	1.13E-09	5.98	3.77E-08	5.38
C_c2	8.18E-08	5.24	1.39E-08	5.56	2.11E-08	5.48	1.84E-09	5.90	5.60E-08	5.31
C_c3	7.87E-08	5.24	1.60E-08	5.53	3.40E-08	5.40	8.32E-09	5.64	7.53E-08	5.25
D_c0	1.01E-07	5.20	5.17E-08	5.32	2.08E-08	5.48	3.02E-08	5.42	7.19E-08	5.26
D_c1	1.22E-07	5.16	5.16E-08	5.32	1.37E-08	5.56	1.55E-08	5.54	5.76E-08	5.30
D_c2	1.53E-07	5.12	6.39E-08	5.28	1.73E-08	5.52	1.92E-08	5.50	7.70E-08	5.25
D_c3	1.61E-07	5.11	7.90E-08	5.24	3.25E-08	5.40	5.04E-08	5.33	1.25E-07	5.16
E_c0	1.23E-07	5.16	6.96E-08	5.27	4.60E-08	5.34	5.26E-08	5.32	1.18E-07	5.17
E_c1	1.36E-07	5.14	6.17E-08	5.29	2.80E-08	5.43	5.01E-08	5.33	8.87E-08	5.22
E_c2	1.68E-07	5.10	7.43E-08	5.25	3.39E-08	5.40	5.50E-08	5.31	1.14E-07	5.18
E_c3	1.84E-07	5.08	9.98E-08	5.20	7.05E-08	5.26	6.82E-08	5.27	1.99E-07	5.07

Table 6. Failure probability and reliability indices for concrete C40/50.

Beams	60x30		50x30		40x30		30x30		30x60	
C40/50	λ_f	β	λ_f	β	λ_f	β	λ_f	β	λ_f	β
B_c0	8.73E-16	7.96	2.32E-12	6.92	5.46E-16	8.02	3.20E-14	7.50	4.37E-13	7.15
B_c1	4.18E-15	7.76	6.65E-12	6.77	9.04E-16	7.95	3.22E-14	7.50	1.48E-12	6.98
B_c2	7.63E-15	7.69	9.47E-12	6.71	1.53E-15	7.89	4.44E-14	7.46	3.13E-12	6.87
B_c3	4.57E-15	7.75	8.82E-12	6.72	1.50E-15	7.89	9.37E-14	7.36	2.18E-12	6.92
C_c0	3.64E-08	5.38	7.02E-09	5.67	1.23E-08	5.58	7.40E-09	5.66	5.47E-08	5.31
C_c1	5.15E-08	5.32	8.05E-09	5.65	1.13E-08	5.59	3.79E-09	5.78	6.59E-08	5.28
C_c2	6.78E-08	5.27	1.06E-08	5.60	1.33E-08	5.56	4.96E-09	5.73	8.85E-08	5.22
C_c3	6.30E-08	5.28	1.12E-08	5.59	1.83E-08	5.51	1.25E-08	5.57	1.01E-07	5.20
D_c0	8.21E-08	5.24	1.86E-07	5.08	1.11E-08	5.59	1.03E-08	5.61	4.04E-08	5.37
D_c1	1.06E-07	5.19	1.97E-07	5.07	8.58E-09	5.64	4.08E-09	5.77	3.87E-08	5.37
D_c2	1.31E-07	5.15	2.38E-07	5.04	1.06E-08	5.60	5.50E-09	5.71	5.26E-08	5.32
D_c3	1.27E-07	5.15	2.71E-07	5.01	1.64E-08	5.53	1.71E-08	5.52	6.63E-08	5.28
E_c0	9.65E-08	5.21	4.91E-08	5.33	2.33E-08	5.46	2.07E-08	5.48	1.16E-07	5.17
E_c1	1.10E-07	5.18	4.61E-08	5.34	1.66E-08	5.52	7.02E-09	5.67	1.03E-07	5.19
E_c2	1.39E-07	5.14	5.40E-08	5.31	2.02E-08	5.49	9.39E-09	5.62	1.29E-07	5.15
E_c3	1.40E-07	5.14	6.68E-08	5.27	3.27E-08	5.40	3.50E-08	5.39	1.79E-07	5.09

In these sections, in fact, the resisting moment of the EAF concrete is greater than the normal ones about 0.6 - 1%, this is not enough to compensate the increased acting moment and a greater reliability index.

However, RC elements with EAF concretes can guarantee the same reliability level of the relative natural concrete ones.

Table 7. Failure probability and reliability index for concretes strength C50/60.

Beams	60x30		50x30		40x30		30x30		30x60	
C50/60	λ_f	β	λ_f	β	λ_f	β	λ_f	β	λ_f	β
B_c0	9.46E-16	7.95	2.21E-12	6.92	4.01E-16	8.05	1.71E-14	7.58	3.65E-13	7.17
B_c1	3.94E-15	7.77	6.37E-12	6.77	8.35E-16	7.96	2.22E-14	7.55	1.56E-12	6.97
B_c2	6.33E-15	7.71	9.58E-12	6.71	1.31E-15	7.91	3.17E-14	7.50	2.69E-12	6.90
C_c0	3.22E-08	5.41	5.61E-09	5.71	9.82E-09	5.62	3.74E-08	5.38	4.34E-08	5.35
C_c1	5.03E-08	5.33	7.69E-09	5.66	1.05E-08	5.60	3.42E-08	5.40	6.12E-08	5.29
C_c2	6.76E-08	5.27	9.98E-09	5.61	1.24E-08	5.57	3.78E-08	5.38	8.02E-08	5.24
D_c0	7.50E-08	5.25	1.57E-07	5.11	7.98E-09	5.65	4.23E-08	5.36	3.05E-08	5.42
D_c1	1.03E-07	5.19	1.93E-07	5.08	8.23E-09	5.65	3.35E-08	5.40	3.68E-08	5.38
D_c2	1.26E-07	5.16	2.28E-07	5.04	9.39E-09	5.62	3.71E-08	5.38	4.49E-08	5.35
E_c0	8.70E-08	5.23	3.94E-08	5.37	1.64E-08	5.53	1.55E-07	5.12	8.64E-08	5.23
E_c1	1.10E-07	5.18	4.55E-08	5.34	1.59E-08	5.53	1.17E-07	5.17	1.01E-07	5.20
E_c2	1.33E-07	5.15	5.12E-08	5.32	1.71E-08	5.52	1.25E-07	5.16	1.15E-07	5.17

7. Conclusions

In this work, the Monte Carlo simulation has been used to conduct a reliability analysis in the simple supported beam elements with the conventional and recycled concrete: the failure probability and relative reliability index are estimated and compared to the reliability of the same RC elements with natural concrete. Results indicate that, for the analysed cases, when EAF slag is used as a coarse recycled aggregate, it is possible to guarantee the same reliability level than RC elements with natural concrete.

References

[1] D.M. 17/01/2018 Norme tecniche per le costruzioni *Gazzetta Ufficiale della Repubblica Italiana*
[2] Ribeiro S E C and Diniz S M C 2013 Reliability-based design recommendations for FRP-reinforced concrete beams *Engineering Structures* **52** 273 – 83
[3] Cosenza E, Galasso C and Maddaloni G 2014 Uncertainly Analysis of Flexural Overstrength for Capacity Design of RC Beams *J. Of Structural Engineering* **140**(7)