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Differences on specified and actual concrete strength for buildings on seismic zones

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Abstract. The design of reinforced concrete structures strongly depends on the value of the compression concrete strength used for the structural components. Given the uncertainties involved on the materials quality provided by concrete manufacturers, in the construction stage, these components may be either over or under-reinforced respect to the nominal condition. If the structure is under reinforced, and the deficit on safety level is not as large to require the structure demolition, someone should assume the consequences, and pay for the under standard condition by means of a penalty. If the structure is over reinforced, and other failure modes are not induced, the builder may receive a bonus, as a consequence of the higher, although unrequested, building resistance. The change on the building safety level is even more critical when the structure is under a seismic environment. In this research, a reliability-based criteria, including the consideration of expected losses, is proposed for bonification/penalization, when there are moderated differences between the supplied and specified reinforced concrete strength for the buildings. The formulation is applied to two hypothetical, with regular structural type, 3 and 10 levels reinforced concrete buildings, located on the soft soil zone of Mexico City. They were designed under the current Mexican code regulations, and their responses for typical spectral pseudoaccelerations, combined with their respective occurrence probabilities, are used to calculate the building failure probability. The results are aimed at providing objective basis to start a negotiation towards a satisfactory agreement between the involved parts. The main contribution resides on the explicit consideration of potential losses, including the building and contents losses and the business interruption due to the reconstruction period.

Keywords: nominal and actual concrete strength; expected failure costs; R/C buildings; seismic loading; compensation factors

1. Introduction

There are some buildings where, after construction, a discovery is made about the lower than the specified value of the concrete strength at some members. Assume the case of an actual resistance lower than the nominal value. If the difference is not big enough to consider the option of demolition and, if the client and the contractor are willing to reach an agreement, a reliability-based criterion may be used as a rational basis to start a negotiation. Also, when there are zones within the building where the concrete strength is higher than the specified one, the local over strength may contribute to compensate the deficits found elsewhere. This topic was put forward by late Prof. Rosenblueth (Rosenblueth *et al*. 1974, 1986). His treatment included the consideration of intuitive potential losses, representing the earthquake occurrence through a Poisson model by first considering a deterministic market and, later on, a probabilistic one. These pioneering works opened the field to the perspective of generating risk and reliability-based specifications, not just for the construction industry but for any kind of market products in the mid-seventies. More recently, in Arizona, another formulation was proposed to compensate deficits on the concrete strength (Laungrungrong *et al*. 2008), although neither the building failure probability nor the future expected losses were considered.

Damage, redundancy and robustness have also been studied for reinforced concrete structures (Frangopol and Curley 1987, Baker *et al*. 2008, Sykora *et al*. 2011). However, the potential building losses are not calculated in these works.

Unlike the previous research, in this work, the effect that the difference, between the actual and the nominal concrete resistance, may have on both the overall structural safety and the expected losses due to failure consequences, are here explicitly taken into account.

Several existing reliability tools for building structural safety are applied, like in recent works where fragility estimates were estimated for reinforced concrete buildings (Xu and Gardoni 2016, Ramamoorthy *et al*. 2008), and where seismic damages have been predicted for decisión making (Hueste and Gardoni 2009, Ramamoorthy *et al*.

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2006, Williams *et al*. 2009). For example, the life-cycle analysis of structures under seismic loads used by some authors (Gardoni *et al*. 2016, Kumar and Gardoni 2014, Takahashi *et al*. 2004) is followed to consider present and future costs.

In other works, a probabilistic format and expected life-cycle costs were proposed to set the basis for a concrete building code (Rosenblueth and Esteva 1972). Stochastic and renewal theory models have been used to study the system deterioration (Kumar *et al*. 2015, Kumar and Gardoni 2014, Kumar *et al*. 2009) and the time-variation of the failure probability due to degradation has been analyzed for reinforced concrete beams (Teply *et al*. 1999). Recently, the reinforcing effect of new materials, like carbon fiber laminates, was analyzed for reinforced concrete bridge girders (Okeil and Shahawy 2007). The opposite effect, reliability reduction, of adverse conditions on concrete structures has been also studied within a probabilistic framework (Vrouwenvelder 2010). Karimiyan and others (Karimiyan *et al*. 2014) analyzed the progressive collapse of reinforced concrete framed buildings under seismic load for a 6 levels building. They confirmed that the collapse mechanism depends on the whole structural system and that building plan asymmetry may produce that instability and collapse occur earlier than symmetric arrays where there is more opportunity for progressive mechanisms to form. Other studies have developed insight into the damage and loss estimation based on story specific seismic demand models (Bai *et al*. 2014). These works serve as an example of the upgrading/downgrading of the structural safety and may be used for the life-cycle analyses. Uncertainty on limit states parameters has been recently studied for reinforced concrete frames and some uncertainty level of these parameters have been linked to light and severe damage (Yu *et al*. 2016).

Other research works where the importance of taking into account the long-term effects of concrete degradation has been included are the ones by Masi and others (Masi *et al*. 2015) where fragility curves were proposed for reinforced concrete buildings under gravity loads. No seismic condition was included here. In other research study, Mori and Ellingwood (Mori and Ellingwood 1999) assessed the service-life of aging structures under a reliability framework. Other one is the prediction of lifetime performance using simulation procedures (Kong and Frangopol 2005). And the life-cycle cost impact of concrete structures under corrosion (Val 2005). Recently, De Stefano and others (De Stefano *et al*. 2015) found that the seismic performance of reinforced concrete structures may become poor, in terms of unexpected torsional or larger deformations, as a result of the variability on the concrete strength.

There are, also, probabilistic formulations that use data from nondestructive techniques to assess the actual concrete compressive strength (Huang *et al*. 2011).

Codes all over the world try to include these aspects through different design regulations. However, the building design still lacks of systematic procedures to regulate the negotiation of bonus/penalties, including the corresponding changes on failure probability and expected losses, when the provided concrete strength is different as the nominal value.

The numerical applications showed in this paper, follow the design requirements of modern current codes on reinforced concrete structures (GDF 2004a, ACI 2011) and local seismic provisions (GDF 2004b).

In terms of the methodology, spectral shapes and safety factors recommended for the design of small and medium size buildings have been incorporated. Maximum responses, for five prescribed spectral pseudoaccelerations used as plateau spectral ordinates, were obtained for the hypothetical analyzed buildings. Conditional failure probabilities were calculated by using the maximum responses and nominal capacities as mean values and introducing typical dispersions. The total probability theorem (Ang and Tang 2007) is then applied considering occurrence probabilities obtained, for the five prescribed pseudo-accelerations for the soft soil area**,** from a previously reported seismic hazard curve for Mexico City (Ang and De Leon 2005). In this way, the unconditional failure probability, the expected life-cycle costs and the corresponding factors to compensate these costs are calculated. The sensitivity of the compensation factors is explored for different costs of failure consequences of the building.

Again, the purpose is to estimate the compensation amounts that may provide a rational basis to settle the discussion to equilibrate the over/under strength whenever a superavit or deficit occurs on the concrete compressive strength $f'c$ of the structural members. Also, it is important to take into account the level of failure/damage consequences, i.e., the building importance according to the use and expected revenues.

2. Formulation of the proposed procedure

The building failure probability is considered as the failure of the critical frame, and the frame failure probability is defined here as the probability that a load effect, or load combination effect, exceeds the resistance of a number of critical structural members; i.e., that the frame limit state G_f is somehow exceeded (Esteva *et al*. 2002). The load combination includes typical code dead and live loads and the local seismic effects

$$P_f = P(G_f < 0) \tag{1}$$

Where G_f represents the event for which the acting stress exceeds the resistant stresses at a combination of structural members that causes the global frame instability. The frame failure probability is calculated, in a simplified way, as follows:

a) The unconditional (total) failure probability P_{fT} is obtained through the convolution of conditional failure probabilities for given scenario seismic intensities "*a*" and the respective occurrence probabilities of these intensities (Bai *et al*. 2011, Ang and Tang 2007)

$$P_{fT} = \int_a P\langle G_f < 0 \,|\, a \rangle P(a) da \tag{2}$$

b) The global frame failure is characterized by the individual failure of a number *n* of critical members, which are identified through a series of frame response analyses (under the scenario a_i) by sequentially eliminating the most stressed members until a global instability is reached.

c) When the critical members are identified, the frame failure probability may be expressed by the product of the conditional frame failure probability given the failure of these critical members and the failure probability of these critical members, assuming that these failure events are independent

$$P_f(G_{fa_i}) = P\langle G_f < 0 \mid a \rangle = P\langle (G_{s\,a_i} \le 0) \mid F_1 \cap F_2 \cap ...F_n \rangle P(F_1 \cap F_2 \cap ...F_n) \quad (3)$$

Where $F_1 \cap F_2 \cap ...F_n$ constitutes the failure of the set of n members, which involve the global frame failure.

d) In order to identify the group of critical members, it is observed that the n critical members trend to fail in a sequential way (Thoft-Christensen and Murotsu 1986). Therefore, for the loading combination of scenario a_i, the first members to fail are the ones with the largest working ratio, let's say, *a* and *b*. If these members are eliminated, the second group of members close to their failures are the ones with the largest working ratio, from the analysis without members *a* and *b*. This time, let's say that the critical members are *c* and *d*. By repeating the process, the frame eventually will collapse and the set of critical members, for the scenario a_i is the set of all eliminated members.

e) Once the set is complete, the first factor in Eq (3) is 1.0 and the system failure probability is the second factor. This second factor may be expressed, through the sequential above described process, as

$$P(F_1 \cap F_2 \cap ...F_n) = P\langle F_1 \cap F_2 \ldots \mid F_a \cap F_b \rangle P(F_a \cap F_b) \quad (4)$$

$$P\langle F_1 \cap F_2 \ldots \mid F_a \cap F_b \rangle P(F_a \cap F_b) = P\langle F_1 \cap F_2 \ldots \mid F_c \cap F_d \rangle P(F_c \cap F_d) P(F_a \cap F_b) \quad (5)$$

f) For the last unconditional probabilities, it might be assumed that the individual failures of members *a*, *b*, *c* and *d*, are independent. Therefore, the failure probability of this group may be obtained through the product of the individual failure probabilities. The calculation of the failure probability for an eliminated member is explained later, the mean acting force is taken as the maximum stresses at the critical member (with working ratio closest to 1), as resulted from the nonlinear response frame analysis. And the mean resistance is the one calculated for the respective member.

If the critical structural member is a beam, the limit state corresponds to

$$G_i = 1 - M_{act}{}^i / M_r{}^i \quad (6)$$

Where $M_{act}{}^i$ and $M_r{}^i$ are the acting and resisting moment of beam *i*. If the critical structural member is a column, the limit state corresponds to the critical combination of axial load and bending moment simultaneously occurring at a column

$$G_i = 1 - (P_{act}{}^i / P_r{}^i + M_{act}{}^i / M_r{}^i) \quad (7)$$

Where $P_{act}{}^i$ is the acting axial load and $P_r{}^i$ is the resistant axial force, for the most critical combination of axial loads and moments occurring simultaneously at all columns in the building. For the columns, the resistant point ($P_r{}^i$, $M_r{}^i$) is obtained through the interaction diagram (González, 2006): the demand point ($P_{act}{}^i$, $M_{act}{}^i$) is located into the diagram and the line joining the origin to this demand point is extended to intercept the curve of the diagram. The intersection point is the resistant point.

g) For the sake of simplicity, the same uncertainty will be considered only on the acting axial load and bending moment (Bojorquez and Ruiz 2014) and also the same uncertainty will be taken for the resisting moment and resisting axial force. The 4 variables will be considered here as lognormal. (DS410 1998, EC 2000, ISO2394 1998).

h) Commonly, for seismic effects (Rosenblueth 1986), the coefficient of variation (*CV*) of 1/3 has been used for loading effects, the axial load and the bending moment, so they will be assumed here to have a *CV*=0.3. Also, the resisting axial force and bending moment are considered to have a *CV*=0.1. And the expected values of P_{act} and M_{act} are considered as the structural responses corresponding to the spectral pseudo-accelerations scenario, which are taken as mean values of the demand.

i) If the structure is located on a seismic zone and this condition dominates the structural design, then these demands are used to calculate P_f (Eq. (1)). Therefore, the failure probability is assessed for all possible maximum spectral pseudo-accelerations, a, related to the soft soil of Mexico City and, by using the total probability theorem, the unconditional failure probability, P_{fT}, is appraised through the convolution of the conditional failure probabilities and the respective occurrence probabilities of the scenarios *a*. The occurrence probabilities are obtained from the Mexico City seismic hazard curve, as previously developed (De Leon 1997) for the soft soil area. Given that the accelerations in that curve range from 0 to 0.5 g, they are discretized in the 5 values 0.1 g, 0.2 g, 0.3 g, 0.4 g y 0.5 g, and Eq. (2) may be expressed as

$$P_{fT} = \sum_{i=1}^{5} P\langle G_f < 0 \mid a_i \rangle P(a_i) \quad (8)$$

The a_i are the considered values of the spectral ordinates. The conditional member failure probabilities

Table 1 Occurrence probabilities for seismic pseudo-accelerations (soft soil, Mexico City)

a_i	$P(a_i)$
0.1 g	0.865
0.2 g	0.092
0.3 g	0.027
0.4 g	0.011
0.5 g	0.003

$P(G_j<0|a_i)$ are calculated through Monte Carlo Simulation (Ang and Tang 1984).

As above mentioned, the probabilities $P(a_i)$ were previously obtained and are shown in Table 1.

j) Next, the expected life-cycle cost $E(C_t)$ is assessed for a range of five concrete compressive strength values f'_c, from 260 to 340 kg/cm^2 and considering the specified value as 300 kg/cm^2. The expected life-cycle cost is expressed in terms of the initial cost C_i (which is deterministic and obtained from conventional unit cost techniques) and the present value of the expected failure cost $E(C_f)$ (Ellingwood 1997, Takahasi *et al*. 2004)

$$E(C_t) = C_i + E(C_f) \tag{9}$$

The present value of the expected failure cost depends on the present value function, *PVF*, to update the future costs in terms of present value, the failure cost and the failure probability

$$E(C_f) = PVF * C_f * P_f \tag{10}$$

$$PVF = (1 - \exp(-rT) / r \tag{11}$$

where r = net annual interest rate, and T = structure lifetime.

The expected failure cost includes not only the material building loss, but also the failure consequences, as the loss of profit during the building repair/reconstruction, potential fatalities and injuries to occupants.

k) Finally, the compensation factors F are assessed by dividing the expected life-cycle cost corresponding to the provided f'_c by the expected life-cycle cost corresponding to the specified f'_c, assumed to be 300 kg/cm^2, as already mentioned. Also, the influence of the building importance (measured in terms of the expected losses which depend on the number of occupants at risk and the use/profit of the building) on the factor magnitude is explored in terms of the cost ratio: the present value of expected failure cost versus the building initial cost, i.e., $E(C_f)/C_i$. The buildings are designed according to current standards for reinforced concrete buildings (GDF 2004a, b), current practices and current costs in Mexico.

3. Illustration for 3 and 10 levels buildings

3.1 Buildings characteristics

Two reinforced concrete buildings with regular framed structural types, and with 3 and 10 levels, were considered. Fig. 1 shows the plan and elevation views.

The details of cross sections and reinforcement areas of members are shown in Table 2. Stirrups and reinforcement details are assumed to be provided as to exclude shear failure modes and fulfill ductility code requirements (GDF, 2004b).

3.2 Failure probability for 3 levels building

As a first step, the bending and axial resistances (M_r, P_r) interaction diagrams are assessed (ACI 2011, González

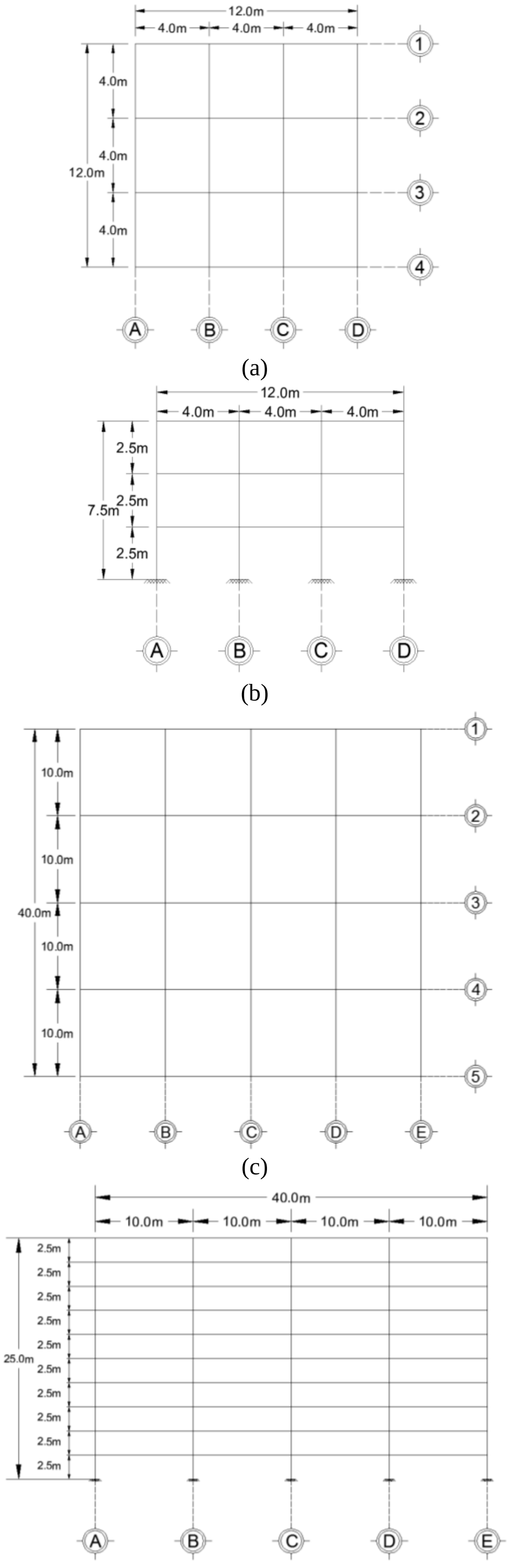

Fig. 1 (a) Plan, 3 levels building (b) Elevation, 3 levels building (c) Plan, 10 levels building (d) Elevation, 10 levels building

Table 2 Cross sections and reinforcement areas for the 2 buildings

Building	Section (cm×cm)	Reinforcement area (cm^2)
3 levels		
Column level 1. Central	30×30	9
Column level 1. Edge	30×30	12
Column level 1. Corner	30×30	16
Columns levels 2 and 3.		
Central	20×20	6.7
Columns levels 2 and 3.	20×20	10.8
Edge	20×20	12
Columns levels 2 and 3.	20×20	12
Corner		
Beams	30×20	2.7
10 levels		
Columns level 1. Central	100×100	250
Columns level 1. Edge	100×100	270
Columns level 1. Corner	100×100	290
Columns level 2. Central	100×100	162
Columns level 2.Edge	100×100	180
Columns level 2.Corner	100×100	200
Columns level 3. Central	100×100	100
Columns level 3. Edge	100×100	120
Columns level 2. Corner	100×100	140
Columns level 4. Central	90×90	90
Columns level 4. Edge	90×90	100
Columns level 4. Corner	90×00	110
Columns level 5. Central	90×90	85
Columns level 5. Edge	90×90	95
Columns level 5. Corner	90×90	105
Columns level 6. Central	90×90	81
Columns level 6. Edge	90×90	90
Columns level 6. Corner	90×90	100
Columns level 7. Central	80×80	70
Columns level 7. Edge	80×80	80
Columns level 7. Corner	80×80	90
Columns level 8. Central	80×80	64
Columns level 8. Edge	80×80	70
Columns level 8. Corner	80×80	75
Columns levels 9 and 10	70×70	50
Beams levels 1, 2	50×70	44
Beams levels 3 to 10	50×60	40

2006) for each a_i and for all values of f'_c. As examples, the interaction diagrams for a corner column of the third level for f'_c=260 kg/cm^2 and 340 kg/cm^2 are shown in Figure 2. Also, the demand point for a_i=0.3 g is plotted with the straight line to identify the resistant point. The determination of the resistant point is graphically shown, as described in the Formulation section. The conditional reliability index and conditional failure probability are estimated for each scenario with Eqs. (4) to (7) and (9). A sample of the results is shown in Table 3 (interior columns) and 4 (beams) of the first level, both for a_i=0.1 g.

Note that, when f'_c varies from 260 to 340 kg/cm^2, the interaction diagram not just translates, but also changes its shape. Therefore, all the diagrams need to be calculated for all f'_c. It can also be observed that, as in the case of interior columns, a small change on f'_c, for example, from 300 to 320 kg/cm^2 or from 300 to 280 kg/cm^2, will lead to a significant change on the failure probability. This is because the change on the interaction diagrams is also significant.

For the case of beams, the differences on f'_c do not generate strong changes on the failure probability. Also it is observed that, in general, the beam failure probabilities are higher than those for the columns. This is in accordance with the design philosophy of "strong column-weak beam". The critical members for the three levels building and, as an example, for a_i=0.1 g and f'_c=280 Kg/cm^2 are listed in Table 3 and shown in Fig. 3.

A sample of the calculation of failure probability, for the 3 levels building, a_i=0.1 g and f'_c=280 Kg/cm^2 is shown, as an example, in Table 4. In this table, URN_1 and URN_2 are uniformly distributed random numbers (one to simulate the

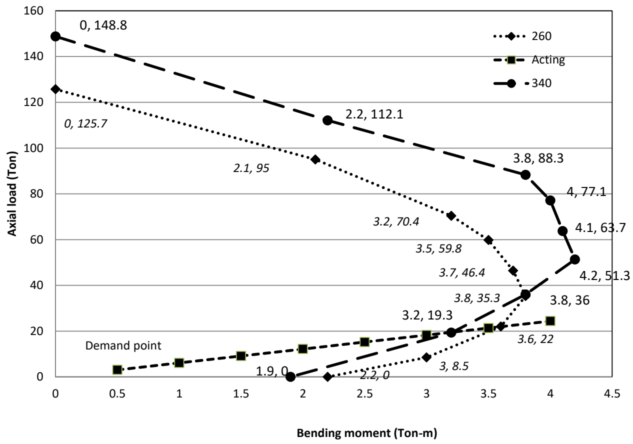

Fig. 2 Examples of columns interaction diagrams and demand point (■) for a_i=0.3 g, f'_c=260 (a) (◆) and 340 Kg/cm^2 (b) (●), and for a corner column, level 3 and three levels building

Table 3 Critical members for three levels building, a_i=0.1 g and f'_c=280 Kg/cm^2

Round of elimination	Elements with maximum working ratio
0	19, 28, 59, 68
1	22, 30
2	25,32
3	62, 65, 70,72, 102, 110
4	17, 24, 35, 39, 57, 64, 75, 79

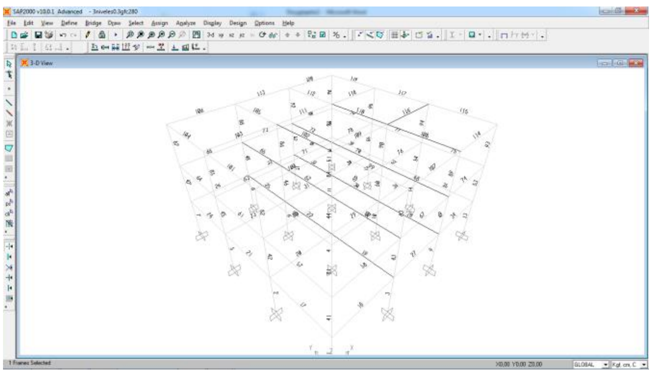

Fig. 3 SAP view of critical members for 3 levels building, a_i=0.1 g and f'_c=280 Kg/cm^2

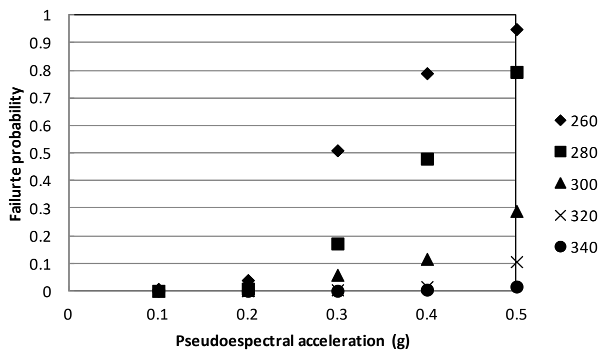

Fig. 4 Conditional failure probabilities for the 3 levels building, several f´c (Kg/cm2) and seismic intensities in terms of "g"

Table 4 Sample of calculation of the failure probability for the 3 levels building, a_i=0.1 g and $f`_c$=280 Kg/cm^2

URN_1	z_1	M_{act}	URN_2	z_2	M_r	G_i	*Ind*
0.016	-2.145	1.523	0.427	-0.184	3.800	-1.496	0
0.841	1.000	3.833	0.912	1.355	4.431	-0.156	0
0.709	0.550	3.359	0.176	-0.929	3.528	-0.051	0
0.781	0.777	3.590	0.588	0.224	3.958	-0.103	0
0.696	0.512	3.321	0.612	0.284	3.982	-0.199	0
0.412	-0.223	2.676	0.542	0.105	3.912	-0.461	0
0.449	-0.129	2.752	0.919	1.396	4.449	-0.617	0
0.237	-0.715	2.316	0.050	-1.646	3.285	-0.418	0
0.831	0.960	3.788	0.726	0.601	4.110	-0.085	0
0.661	0.414	3.227	0.597	0.244	3.966	-0.229	0
0.674	0.451	3.263	0.736	0.632	4.123	-0.264	0
0.879	1.169	4.028	0.641	0.361	4.013	0.004	1
0.784	0.785	3.599	0.314	-0.485	3.688	-0.025	0

Table 5 Conditional failure probabilities for the 3 levels building and for all seismic intensities and strengths $f`_c$

Seismic intensity	$f´_c$=260 Kg/cm^2	$f´_c$=280 Kg/cm^2	$f´_c$=300 Kg/cm^2	$f´_c$=320 Kg/cm^2	$f´_c$=340 Kg/cm^2
0.1 g	0.47	0.1	0.039	4.7×10^{-7}	2.9×10^{-7}
0.2 g	0.789	0. 382	0.27	1.28×10^{-3}	8×10^{-3}
0.3 g	0.91	0.72	0.58	0.36	0.24
0.4 g	0.99	0.98	0.96	0.55	0.44
0.5 g	1	0.995	0.99	0.746	0.56

Table 6 Unconditional failure probabilities for the 3 levels building and for all $f`_c$ (Kg/cm^2)

$f´_c$=260	$f´_c$=280	$f´_c$=300	$f´_c$=320	$f´_c$=340
0.0349	0.0131	0.0039	0.0005	0.0001

acting moment and the other one for the resistant moment), z_1 and z_2 are the corresponding normal transformed numbers, M_{act} and M_r, are the lognormal acting and resistant moments, G_i the limit state for beams (Eq. (6)), and "*Ind*" the failure indicator (0 is survival and 1 is failure). The conditional failure probability is obtained by adding all the outcomes "*Ind*" for all trials and by dividing the total by the number of trials, 10,000 in this case.

The failure probabilities for all a_i and all $f´_c$, are shown in Table 5 and Fig. 4.

The unconditional failure probabilities are calculated by applying the total probability theorem (Eq. 8) and with the occurrence probabilities (Table 1) and the conditional failure probabilities for all $f`_c$ (Table 5). The results are shown in Table 6.

3.3 Failure probability for 10 levels building

The procedure previously illustrated for the 3 levels building is applied to the one with 10 levels. The interaction diagrams for columns and bending resistance for beams are calculated for all members, the conditional reliability indices are estimated, the conditional failure probabilities are assessed and the unconditional failure probabilities are calculated. Moreover, the expected life-cycle costs and the corresponding factor are calculated. Again, all this is carried out for the 4 failure costs. The results are shown in Table 7 and Fig. 5. The initial cost of the building is estimated as 368 million pesos.

Again, as expected, the failure probability in Fig. 5 increases as the pseudoacceleration increases and the $f´_c$ decreases, but it is observed that the inflexion point is 0.4 g.

3.4 Factors calculation for the 3 levels building

With the assessed unconditional failure probabilities, the expected life-cycle cost is appraised by using (10). Previous works (Rosenblueth 1986, De Leon 1997) have shown

Table 7 Conditional failure probabilities for 10 levels building for all seismic intensities and several $f`_c$ (Kg/cm^2) for seismic intensities in terms of "*g*"

Seismic intensity	$f´_c$=260	$f´_c$=280	$f´_c$=300	$f´_c$=320	$f´_c$=340
0.1 g	0.009	0.0007	0.0002	5×10^{-6}	8×10^{-7}
0.2 g	0.016	0.0056	0.0008	9×10^{-5}	4.6×10^{-5}
0.3 g	0.09	0.03	0.006	0.0025	0.00091
0.4 g	0.97	0.74	0.213	0.045	0.0027

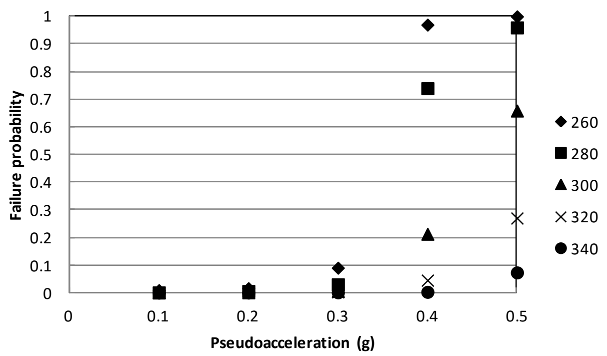

Fig. 5 Conditional failure probabilities for 5 values of $f`_c$ for the 10 levels building

that typical buildings in Mexico have cost ratios $E(C_f)/C_i$ ranging from 20 to 500, according to the use and profit and location of the buildings (apartments, office, etc.). Therefore, the parametric analyses will consider cost ratios of 20, 100, 200 and 500.

The ratios between expected life-cycle costs of the supplied f'_c respect to the one of 300 kg/cm^2, assumed to be the nominal one, are assessed. Table 8 and Fig. 6 show the factors for a building with an initial cost of 10 million pesos. It is observed that, as expected, the factors increase as the f'_c decreases and the values are larger than 1 for f'_c lower than 300 Kg/cm^2, whereas they are smaller than 1 for values larger than 300. Also, the factor increases as the loss ratio due to failure consequences (building importance) increases.

Table 8 Expected life-cycle costs (million pesos) and factors calculation for the 5 values of f'_c

	C_t =	200		1000		2000		5000	
f`c	Pt	E(C_t)	Factor	E(C_t)	Factor	E(C_t)	Factor	E(C_t)	Factor
260	0.0349	95.8	4.8	438.8	7.4	867.6	8.0	2154	8.4
280	0.0131	42.3	2.1	171.4	2.9	332.7	3.1	816.8	3.2
300	0.0039	19.8	1.0	59.0	1.0	108.0	1.0	255.1	1.0
320	0.0005	11.5	0.6	17.3	0.3	24.5	0.2	46.4	0.2
340	0.0001	10.3	0.5	11.3	0.2	12.6	0.1	16.4	0.1

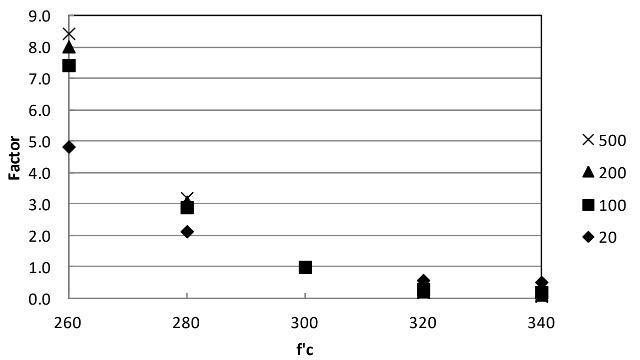

Fig. 6 Conditional failure probabilities for 5 values of f'_c for the 3 levels building

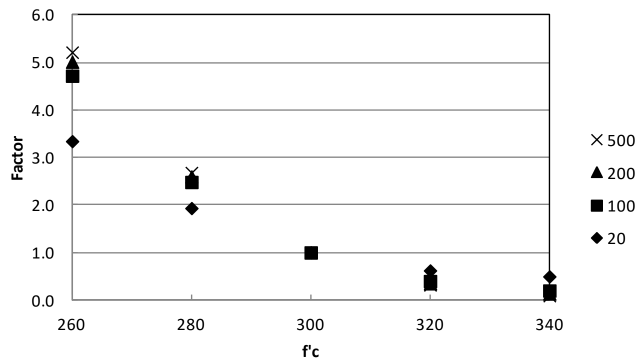

Fig. 7 Factors for 5 values of f'_c and 4 failure costs for the 10 levels building

3.5 Factors calculation for the 10 levels building

Similarly, factor for the 10 levels building are assessed. According to Fig. 7 the factors increase as the f'_c decreases and, of course, as the cost ratio increases.

4. Analysis and Discussion

The compensation factors that were calculated apply for unit volume of concrete (ft^3 or m^3) to the structural members where the over/under strength occurred. Factors are >1 are for the cases of under strength and <1 for the ones of over strength. It may be observed that, as the building importance increases (larger cost ratio), the factors also increase. However, it seems that there is a limit close to the cost ratio of 500 because the points for the cost ratios of 500 and 200 are closer, between them, than the ones from 20 to 100 and those from 100 to 200. That means that, apparently there is a bound for the factors in order to provide the required safety to the building. And, if additional safety is required, is should be provided for other means (i.e., additional reinforcing members).

Although an unrequested over resistance may be considered out of any compensation discussion, it may mitigate the impact of the deficits that might have occurred at other zones of the structure. In any case, the factors introduced here may help as a technical back up to reach an agreement in the negotiation trade-off.

Local site seismicity has an important impact on the building maximum forces and the results may change for sites where the seismic hazard is smaller than the one considered for Mexico City.

The results shown here apply only for the buildings, seismic hazard and costs considered. Other building sizes and types, other seismic exposure and other costs require a specific analysis.

Additional research may focus on the safety redistribution when the differences on f'_c are in some but not all the structural members. In this way, the relative importance of the columns, respect to the beams, might be assessed and the plan and elevation location of the columns may play an important role. Also, the relative importance of the local failure probability of the beams respect to the ones of the columns may be assessed.

Future research may consider combinations of deficit/over strength of concrete that may occur at different structural members, and more detailed analysis should be performed for the specific building characteristics.

Also, it is recommended to study the effect of differences on the f'_c over the safety of important infrastructure facilities, as the failure consequences may be even larger than those for buildings. The effect of aging was not considered and may be a part of the future research.

5. Conclusions

The document presents an initial proposal to systematically and objectively support the discussions about compensations when one or several structural members, within a reinforced concrete building, are found to have an over/under strength on the supplied concrete, respect to the one specified on contracts, when the differences are not too high as to consider the possibility of demolition.

The proposed formulation contributes to promote the weighting of life-cycle consequences beyond the consideration of only initial costs. The compensation factors may contribute to technically support the discussions to balance the expected life-cycle benefits/losses coming from the over/under strength of concrete supplied as compared to the one corresponding to the specified value in contracts. For a range of concrete strength from 250 to 350 Kg/cm^2 the factor varied, for example, from 1 to 5 (for a cost ratio $E(C_f)/C_i$=20) for buildings of 3 levels. However, for the cost ratio of 500, the factor may grow up to 8.5. For the 10 levels building, the factor may go up to 3.3 for a cost ratio of 20 and up to 5.3 for a cost ratio of 500.

The results may serve to update the current codes in Mexico and to improve the concrete acceptance criteria and to enhance some design specifications for buildings construction.

Finally, it is recommended to extend the study to cover other structural types (including walls, foundations, no structural elements, and so on), other geometries (in plan and elevation), the structure age and the deterioration state of not new constructions, among other aspects of the professional practice in Mexico.

Also, reinforced concrete infrastructures, where this kind of deficits on concrete strength may occur, require a specific study to consider failure modes and failure consequences accordingly.

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