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A Practical Methodology for Optimum Seismic Design of RC Frames for Minimum Damage and Life-cycle Cost

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Abstract

The design criteria in current seismic design codes are mainly to control lateral displacements and provide adequate strength to sustain expected design load combinations. However, to achieve the most economic design solutions, the total life-cycle cost (TLCC), which includes both initial structural cost and expected damage cost, should be also considered for the probable earthquakes during the lifetime of the structure. In the present study, the TLCC of the buildings is used as the main objective function for optimum seismic design of reinforced concrete (RC) frames. First, it is demonstrated that the blind increase of the reinforcement ratios does not necessarily reduce the displacement demands and the damage costs. Subsequently, a practical methodology is developed for the optimum seismic design of RC frames based on the concept of uniform damage distribution (UDD). Using an adaptive iterative procedure, the distribution of inter-storey drifts and TLCC of the floors is modified along the height of the structure. To demonstrate the efficiency of the method, 5, 8 and 12 storey RC frames are optimized using the proposed algorithm. The results indicate that, while all predefined performance targets are satisfied, the maximum inter-storey drift ratio and TLCC of the frames are considerably reduced (up to 56% and 45%, respectively) only after a few steps. The proposed method should prove useful for more efficient performance-based design of RC frames in practice.

Keywords: Life-cycle Cost Analysis; Performance-based Design; Uniform Deformation Demands; Seismic Design; RC frames

1. Introduction

Seismic codes use different design parameters to improve the performance of structures to reach an adequate level of safety under earthquake excitations expected to occur during the effective life of the structures. For the RC frames, the dimensions and reinforcement ratios of the elements are primarily designed based on the inter-storey drift limit, and strength criteria, respectively. In recent years, the performance-based design concept is increasingly used to design new structures or retrofit existing substandard systems [1-2]. For the economic assessment of the structures, initial structural costs as well as the expected damage costs are evaluated for the effective lifetime of the structures [3]. The initial structural costs include the structural material, construction quality program, and construction costs, while damage costs mainly include non-structural, economic, human injuries and fatalities, indirect costs, and social probable losses when an earthquake occurs. Since the earthquake is a probability phenomenon, it is necessary to consider the possible effects of the earthquake events in the above assessments [4].

Wen [3] proposed a framework for reliability and performance-based design for natural hazards by considering the structural performance over a lifetime. Design uncertainties were taken into account in terms of hazard demand, structural capacity, nonlinear structural response behaviour, redundancy, the balance of costs and benefits, and target reliability in design for single and multiple hazards. Goulet et al. [5] implemented a performance-based earthquake engineering (PBEE) methodology to predict the collapse safety and economic losses of a four-storey RC moments resisting frame designed according to IBC-2003 used as a benchmark. Performance was quantified in terms of structural and non-structural damage, repair costs, collapse statistics, and fatality losses. It was concluded that the expected annual loss (EAL) estimates are highly sensitive to the manner of estimating the initial stiffness of structural elements.

The damage distribution in the buildings as well as their initial and total life-cycle cost are crucial parameters in the assessment of structures, since reducing damage index at a certain hazard level does not necessarily lead to a reduction in the exceedance probability of damage limits. Sahely et al. [6] developed a framework for the management of buildings and infrastructure and assess the adequacy of alternative investment options. They used life-cycle cost analysis (LCCA) to measure the damage cost due to the future earthquake events during the operational life of structures. Lagaros and Fragiadakis [7] also used LCCA to evaluate the efficiency of ASCE-41, ATC-40 and N2 static pushover methods based on optimally designed buildings. They concluded that the increase in construction costs does not always increase seismic safety. Mitropoulou et al. [8] investigated the effect of the analysis procedure, number of seismic records imposed, adopted performance criterion, structural type (regular or irregular), and the influence of uncertainties on the seismic response and the life-cycle cost analysis of RC structures. Moreover, the LCCA methodology was used as an assessment tool for the structures designed based on a performance-based optimum design methodology. Gencturk and Elnashai [9] provided a problem formulation and a brief review of the existing literature on life-cycle cost (LCC) optimization of structures. Subsequently, a new LCC model was presented to improve some of the shortcomings of existing models. Finally, LCC analysis of an example RC structure was employed to evaluate the efficiency of the proposed methodology.

In Gencturk [10] study, structural optimization was used for LCC assessment of reinforced concrete (RC) and reinforced engineered cementitious composites (ECC), while different response characteristics were used to model the frames. It was shown that both the initial and life-cycle cost of ECC frames are lower due to savings in material and labor cost as well as an improvement in their structural performance. Park et al. [11] used a multi-objective optimization method based on initial retrofit cost and total life-cycle cost (TLCC) to obtain the most suitable retrofit method. It was shown that TLCC is generally governed by the initial retrofit cost, which is relatively high compared to the lifetime seismic damage cost. In another relevant study, Möller et al. [4] presented a general framework for the performance-based design optimization of building structures under seismic demands for the minimum total cost while satisfying reliability levels for the design performance criteria. They divided the total cost into the initial construction cost, the repair costs for the occurred damage, and the associated social costs of economic losses, injuries and fatalities. Gencturk et al. [12] also developed a framework for the comprehensive sustainability assessment of RC structures, which was applied to a five variety of a case study RC frame. The sustainability components were TLCC, downtime, environmental impact, and fatalities.

From studies on optimization and design based on life-cycle cost (LCC), Fragiadakis and Lagaros [13] developed a framework for performance-based optimum seismic design of structures by considering the initial cost or the cost of future earthquake losses during the lifetime of a structural system as objective functions. Similarly, Esteva et al. [14] suggested an alternative approach for life-cycle optimization of structural systems with nonlinear behaviour under severe earthquakes. In their proposed method, the effects of structural damage accumulation under sequences of seismic excitations are taken into account in the assessment of both life-cycle system reliability and structural performance.

Wang et al. [15] proposed a hierarchical life-cycle design (LCD) approach based on the theory of the LCD and LCA, by including the aspects of safety, reliability, durability, economic efficiency, local environmental impacts, social impacts, and global environmental impacts. The results of their study indicate that solutions with strong initial durability design, reasonable future maintenance plan, minimum future maintenance frequency, and high cement replacement rate are more likely to have better comprehensive performance. Shin and Sin [16] focused on the life-cycle cost-based optimal design of yielding metallic dampers. They minimized the expected cost or life-cycle cost by using the Genetic Algorithm (GA) and concluded that although acceleration response-related damage costs were increased, the inter-storey drift related costs were reduced enough to minimize the total lifetime failure cost through the optimum selection of the design parameters and the optimal placement of the devices. Bojórquez et al. [17] proposed reliability-based load factors for the combination of seismic and gravity loads in seismic design of buildings to minimize the expected life-cycle cost of buildings for a specified mean annual failure rate. The optimal load factors were found to be more sensitive to the fundamental period of the structures rather than their effective life. More recently, Nabid et al. [18, 19] proposed a low-cost performance-based optimization, based on uniform deformation theory, for more efficient design of RC frames with friction-based wall dampers. It was shown that optimum designed frames exhibit considerably less maximum inter-storey drift (up to 43%) and global damage index (up to 75%) compared to those designed based on conventional methods.

Above mentioned studies demonstrated that the optimum design solution is generally obtained when a structure satisfies all prescribed performance targets at design time and also provides a balance between the initial structural cost and the expected damage cost over its lifetime. However, due to the nonlinear behaviour of typical building structures under severe earthquakes, most classical optimization methods (such as GA) cannot be practically used for solving this complex optimization problem due to very high computational costs. To address this issue, in the present study, a practical methodology is developed for the seismic optimum design of RC frames for minimum damage and total life-cycle cost by using the concept of uniform damage distribution (UDD). The efficiency of the proposed method in obtaining the optimum design solution with only a few number of analyses is demonstrated through optimum design of 5, 8 and 12 storey RC frames.

2. Life-cycle Cost Analysis (LCCA)

Life-cycle cost is the current value of the total costs that are required to maintain structural conditions during the lifetime of the structure. In general, this cost includes the initial cost of the structure and the cost of damage caused by possible occurrences. The total life-cycle cost (TLCC) can be expressed as a function of time and the design vector as suggested by Wen and Kang [20, 21]:

$$C_{TOT}(t, \boldsymbol{s}) = C_{IN}(s) + C_{LS}(t, \boldsymbol{s}) \tag{1}$$

where, C_{IN} is the initial cost of a new or retrofitted structure; C_{LS} is the present value of the expected damage cost; $\boldsymbol{s}$ is the design vector corresponding to the design loads, resistance, and material properties; and t is the time period. Initial cost refers to all construction costs of a new building such as materials and labours. C_{LS} refers to all values of the expected damage cost of a building after earthquake occurrence such repair cost C_{dam}, loss of contents cost C_{con}, loss of rental cost C_{ren}, income lost cost C_{inc}, cost of injuries C_{inj}, and cost of human fatalities C_{fat}. Therefore, expected damage costs for i^{th} limit-state can be calculated as follows:

$$C_{LS}^{i} = C_{dam}^{i} + C_{con}^{i} + C_{ren}^{i} + C_{inc}^{i} + C_{inj}^{i} + C_{fat}^{i} \tag{2}$$

Considering the Poisson distribution for earthquake events, in the calculation of the damage cost it can be assumed that immediately after an earthquake event, the operation begins to reconstruct and deliver the structure to the initial conditions. Thus, Wen and Kang [20, 21] proposed the following equations for the expected life-cycle cost considering N damage states:

$$C_{LS}(t, s) = (\nu/\lambda)(1 - e^{-\lambda t}) \sum_{i=1}^{N} C_{LS}^{i} P^{i} \tag{3}$$

$$P^{i} = P\left(\theta_{max} > \theta_{max}^{i}\right) - P(\theta_{max} > \theta_{max}^{i+1}) \tag{4}$$

where, P^i is the probability of the i^{th} damage state of a building when an earthquake occurs. θ_{max} is the main characteristic demand parameter, such as maximum inter-storey drift ratio, floor acceleration, and residual drift. Based on Möller et al. [4] study, the total cost is divided into the initial construction cost, $C_0(\boldsymbol{x}_d)$; the cost of repairs for damage produced by earthquakes during the effective life of the structure, $C_d(\boldsymbol{x}_d)$; and the social costs associated with the occurrence of earthquakes $C_s(\boldsymbol{x}_d)$. Therefore, the total life-cycle costs $C(\boldsymbol{x}_d)$ can be estimated by the following equation:

$$C(\boldsymbol{x}_d) = \quad C_0(\boldsymbol{x}_d) + C_d(\boldsymbol{x}_d) + C_s(\boldsymbol{x}_d) \tag{5}$$

In this study, the total life-cycle cost (TLCC) was calculated based on Wen and Kang's relationships [20, 21] and the damage cost defined by Möller et al. [4] using the existing figures in Iran. The repair cost C_d ($\boldsymbol{x}_d$) includes the structural damage costs, non-structural elements damage costs, and contents damage costs all calculated based on the maximum inter-storey drift ratios (i.e. selected damage index) and maximum floor accelerations. The social cost $C_s(\boldsymbol{x}_d)$ consists of re-insertion costs into a normal routine, medical and rehabilitation costs for non-fatal injured victims, costs associated with loss of fatality, and costs associated with loss of business or economic activities.

3. TLCC for a Case Study Example

In this section, the total life-cycle cost (TLCC) for a case study office building in Tehran, Iran, is estimated by calculating the following costs as discussed before:

- The initial cost of a new building (C_{IN}): structural material and construction costs including the costs associate with foundations, columns, beams, and floor slabs erections.
- The present value of the expected damage cost (C_{LS}): damage repair cost of structural elements (C_{dam}), damage repair cost of non-structural sections (C_{nst}), loss of furniture cost (C_{fur}), loss of rental cost (C_{ren}), commercial loss cost (C_{com}), cost of minor injuries (C_{injl}), cost of severe injuries leading to disability (C_{injs}), cost of human fatalities (C_{fat}), and social costs (C_{soc}). It should be noted that the commercial loss depends on companies' downtime working, while the social cost is related to psychological injuries, damage to company brand, and the destructive effects of trauma suffered by survivors.

Typical constructional costs in Tehran, Iran, are used to calculate C_{IN}. To estimate C_{LS}, the maximum inter-storey drift ratios and floor accelerations of the structure under different earthquake intensities are first calculated. The details of the expected damage cost estimations are provided in Table 1.

Table 1. Estimation cost of the losses for the RC frame

NO.	Loss	Related to	Unit	Full loss amount ($)
1	C_{dam}	Damage index	-	Full structural cost
2	C_{nst}	Max floor acc.	Per m^2	92,300
3	C_{fur}	Max floor acc.	Per m^2	74,500
4	C_{ren}	Damage index	Per each company	5,100× Downtime
5	C_{com}	Damage index	Per m^2	17,100× Downtime
6	C_{injl}	Damage index	Per each person	22,900
7	C_{injh}	Damage index	Per each person	2,288,000
8	C_{fat}	Damage index	Per each person	1,000,000
			Per each person	22,900
9	C_{soc}	Damage index	Per each company	85,500
			Per each person	91,000

According to FEMA 227 [22] and ATC-13 [23] recommendations, the criteria for classification of damage costs according to the damage index and acceleration are listed in Table 2.

Table 2. Classification of the levels of damage states [22, 23]

Damage state	Floor acceleration (g)	Inter story drift ratio (%)	FEMA 227 [22]				ATC-13 [23]	
			Mean damage Index of elements (%)	Minor injuries	Major injuries	Fatalities	Downtime (days)	Loss of function (days)
None.	$0.05 < a_{floor}$	$\Delta_{max} < 0.1$	0	0	0	0	0	0
Slight	$0.05 < a_{floor} < 0.10$	$0.1 < \Delta_{max} < 0.2$	0.50	0.00003	0.000004	0.000001	0.9	0.9
Light	$0.10 < a_{floor} < 0.80$	$0.2 < \Delta_{max} < 1.0$	20	0.003	0.0004	0.0001	25	25
Moderate	$0.80 < a_{floor} < 0.98$	$1.0 < \Delta_{max} < 1.8$	45	0.03	0.004	0.001	35	35
Major	$0.98 < a_{floor} < 1.25$	$1.80 < \Delta_{max} < 3.0$	80	0.30	0.04	0.01	65	65
Collapse	$1.25 < a_{floor}$	$3.0 < \Delta_{max}$	100	0.4	0.4	0.2	100	100

In this study, the effective lifetime of the structure and the downtime of full damaged structure are considered to be 50 years and 18 months, respectively. Given that the cost units were extracted from local data in Iran, the annual discount rate λ was assumed to be 15% [24].

All numbers are based on collected data from Iran. The following algorithm is then used to estimate the TLCC of the building:

i) The initial structural cost C_{IN} is calculated for each storey of the buildings separately.

ii) Nonlinear dynamic analyses are conducted on the structure under the accelerograms scaled for different annual probability of occurrence. Table 3 shows the relation between the occurrence probability of the earthquake over a lifetime of 50 years (or the equivalent annual probability of occurrence) with the peak ground acceleration of the design earthquake for the case study example. These data are obtained based on previous seismic hazard assessment studies in Tehran [25].

iii) Maximum inter-storey drift ratios (selected damage index (DI) parameter) and maximum floor accelerations (a_{floor}) of the structure are calculated for each earthquake level. Subsequently, the hazard curve of DI and a_{floor} for each storey is obtained to calculate the occurrence probabilities of each damage state. As shown in Table 3, each PGA level represents an annual exceedance probability for the region of study. It should be noted that based on Gutenberg-Richter recurrence, seismic hazard curve is logarithmic. Therefore, the DI and a_{floor} hazard curves are also presented in logarithmic scale. When the logarithmic exceedance probability values are taken, the linear regression relationship can be used to find the equation of each hazard curve as shown in Eqs. (6) to (8):

Table 3. Relation between annual probabilities of the earthquake events and PGA

Occurrence Probability	Equivalent annual probability of occurrence (%)	Return Time (year)	Peak ground acceleration (g)
50% in 50 years	1.39	72	0.25
10% in 50 years	2.1×10^{-1}	475	0.35
2% in 50 years	4.04×10^{-2}	2475	0.52

$$y = \alpha + \beta x \tag{6}$$

Then:

$$\beta = \frac{cov[\boldsymbol{x}, \boldsymbol{y}]}{var[\boldsymbol{x}]} \quad (7)$$

$$\alpha = \bar{y} - \beta\bar{x} \quad (8)$$

where, x denotes the selected DI (or a_{floor} or residual drift) and y is the natural logarithm of annual exceedance probabilities. $\bar{x}$ is the average of DI (or a_{floor} or residual drift) of outputs from the structural analysis under different earthquake accelerograms and $\bar{y}$ is the natural logarithm of annual exceedance probability for each earthquake intensity. Subsequently, the cumulative distribution function of each damage index is calculated:

$$P(DI \geq DI_i|EQ) = \exp(\alpha) * \exp(\beta * DI_i) \quad (9)$$

$$or$$

$$P\left(a_{floor} \geq a_{floor_i}\middle|EQ\right) = \exp(\alpha) * \exp(\beta * a_{floor_I}) \quad (10)$$

where DI_i and a_{floori} are the maximum inter-storey drift ratio and the maximum floor acceleration of the i^{th} damage state.

iv) Based on the estimated probability functions for DI and a_{floori} corresponding to different damage states, the occurrence probability of each damage state is estimated by using Eq. (4). Subsequently, the different damage costs are estimated by using Eq. (3) and the relationships listed in Table 1. The results are then used to estimate the TLCC by using Eq. (1).

v)

4. Structural Modeling

In this study, three RC frames with 5, 8, and 12 storeys and 3 bays were initially designed in accordance with ASCE07-16 [26] and ACI 318-14 [27]. The site soil profile was assumed to be type C category of ASCE07-16 [26]. The dead and live loads for interior storeys were considered to be 6 and 2 kN/m^2, respectively, while the corresponding loads were reduced to 5 and 1.5 kN/m^2 for the roof level. The short spectral response acceleration (S_s) and the spectral response acceleration (S_1) at 1-s period were assumed 1.3 g and 0.56 g, respectively. Buildings were considered to be ordinary office buildings with medium importance and intermediate ductility. Fig. 1 and Table 4 show the structural details of the initial designed frames used in this study. All of these details can satisfy the capacity demands as well as the provisions of the ACI 318-14 [27] design code.

Figure 1. Geometry details of the 5, 8, and 12-storeys RC frames

Table 4. Reinforcement details of the original RC frames

	Storey	Type of the Element	As (top bars) (mm^2)	As' (bot. bars) (mm^2)	Numbers of Stirrup legs
5 -storey	1&2	Beam	1800	900	2Φ10@100
		Column	1700	1700	3Φ10@150
	3	Beam	1600	800	2Φ10@100
		Column	1250	1250	3Φ10@150
	4&5	Beam	1300	650	2Φ10@100
		Column	1020	1020	2Φ10@150
8-storey	1&2	Beam	2100	1050	2Φ10@100
		Column	2000	2000	4Φ10@150
	3&4	Beam	2000	1000	2Φ10@100
		Column	1500	1500	3Φ10@150
	5&6	Beam	1800	900	2Φ10@100
		Column	1250	1250	3Φ10@150
	7&8	Beam	1500	750	2Φ10@100
		Column	1030	1030	2Φ10@150
12-storey	1,2&3	Beam	1900	950	2Φ10@100
		Column	2000	2000	4Φ10@150
	4,5&6	Beam	1900	950	2Φ10@100
		Column	1500	1500	3Φ10@150
	7,8&9	Beam	1800	900	2Φ10@100
		Column	1100	1100	3Φ10@150
	10,11&12	Beam	1400	700	2Φ10@100
		Column	900	900	2Φ10@150

It should be noted that RC buildings generally consist of 3D frame systems. However, based on the ASCE 41-17 [2] regulations, use of a two-dimensional model is permitted if the building has rigid diaphragms and torsion effects do not exceed the specified limits. In the current study it is assumed that the buildings are regular with rigid diaphragms, and therefore, the frames are independently resist the seismic loads.

Nonlinear dynamic analysis of the structures under earthquake excitations were conducted using the open source software IDARCV7.0 [28]. Beam and column members were cracked modelled using non-linear fibre elements with spread plasticity formulation. Valles et al. [29] demonstrated that the results of the IDARC are validated when the hysteresis curves are accurately modelled. For accurate modelling of the hysteresis curves in this study, the proposed methods in [30] were employed, which have been validated against experimental results. More information on the numerical model calibration of structures can be found in [31, 32]. Using Rayleigh damping model, 0.05 damping ratio was assigned to the first mode and to the mode at which the cumulative mass participation exceeds 95%. P-Delta effects were taken into account in the analyses.

Based on the seismicity of the assumed site, a set of 20 natural accelerograms were selected from the Pacific Earthquake Engineering Research Center (PEER) database [33] as listed in Table 5. All earthquake excitations had high local magnitudes (i.e. Ms>6.0) and were recorded on soil class C of ASCE7-16 [26] at distances ranging from 11 to 118 km. The major components of earthquake records were scaled to the selected design response spectrum using the ASCE7-16 [26] suggested methodology as shown in Fig. 2. The fundamental period of the 5, 8, and 12 storey frames were 1.1, 1.6, and 2.1 s, respectively. It should be mentioned that the same earthquake records have been used in other studies as well (e.g. [34]).

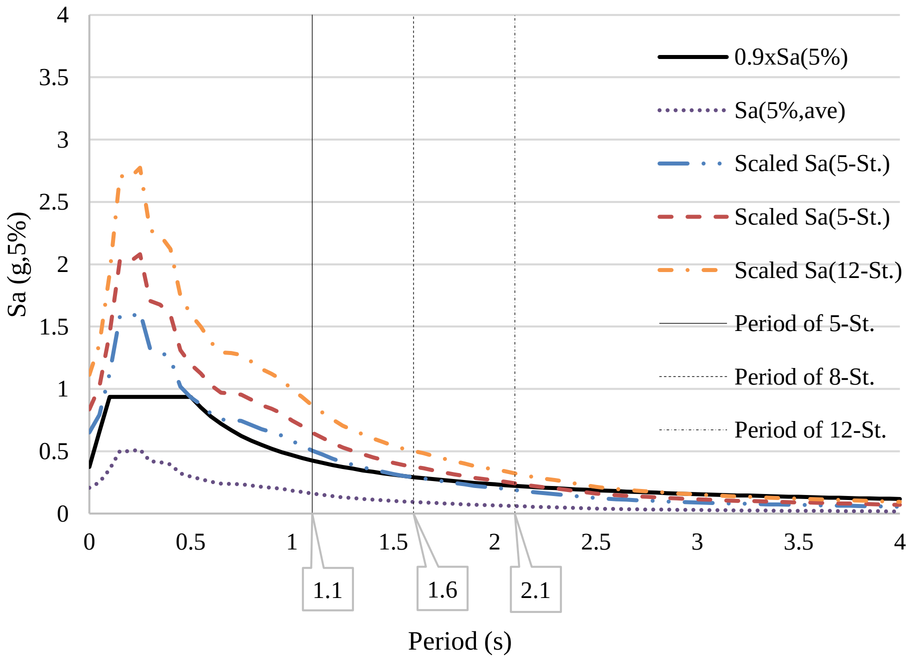


Figure 2. The scaled average spectra for design base earthquake for the 5, 8, and 12-storey RC frames

Table 5. Selected natural accelerograms [34]

Earthquake Name	Location	Year	Magnitude	PGA (g)	Vs(m/s)
Northridge	Littlerock, Brainard Canyon	1994	6.7	0.071	486
Northridge	Castaic Old Ridge Route	1994	6.7	0.56	450
Northridge	Lake Hughes #1	1994	6.7	0.09	425
Northridge	Rancho Paolos Verdes, Hawth	1994	6.7	0.071	580
Imperial Valley	Parachute Test site	1979	6.5	0.2	350
San Fernando	Lake Hughes, #12	1971	6.6	0.35	602
San Fernando	Pasadena, CIT Kresge	1971	6.6	0.1	415
San Fernando	Castaic Old Ridge Route	1971	6.6	0.31	450
Loma Prieta	Gilroy, Gavilon college	1989	6.9	0.35	730
Loma Prieta	Gilroy #6, San Ysidro	1989	6.9	0.167	663
Loma Prieta	Saratoga, Aloha Ave.	1989	6.9	0.50	381
Loma Prieta	Santa Cruz, UCSC	1989	6.9	0.11	713
Loma Prieta	San Francisco, Dimond Heighs	1989	6.9	0.1	583
Morgan Hill	Gilroy#6, San Ysidro	1984	6.2	0.22	663
Morgan Hill	Gilroy, Gavilon College	1984	6.2	0.097	730
Kern County	Santa Barbara, Courthouse	1952	7.4	0.052	515
Kern County	Pasadena, CIT Athenaeum	1952	7.4	0.13	415
N. Palm Springs	Fun Valley	1986	6.0	0.13	389
Whittier Narrows	Cataic, Old Ridge Route	1987	6.0	0.067	450
Whittier Narrows	Riverside. Airport	1987	6.0	0.057	390

5. Seismic Performance Assessment of the RC Frames

In this section, the interactions between the initial structural cost, TLCC and maximum inter-storey drift ratios are investigated. To achieve this, a wide range of RC frames were obtained by changing the reinforcement ratio of the beam and column elements of the initially designed RC frames, while the dimensions of the cross sections were fixed. To provide reasonable design solutions and also restrict the number of the analysis, the reinforcement ratios were discretely changed in three steps by using the minimum, median, and maximum ratios of the limits recommended by ACI 318-14 [27].

For the 5 storey RC frames, the sections and reinforcement ratios of first and second storeys, and fourth and fifth storeys were considered to be similar. This led to 27 (=3^3) types of beams and 27 (=3^3) types of columns in total. For the 8 storey RC frames, the sections and reinforcement ratios of first and second storeys, third and fourth storeys fifth and sixth storeys, and seventh and eighth storeys were assumed to be similar. This resulted in 81 (=3^4) types of beams and 81 (=3^4) types of columns. For the 12 storey RC frames, the sections and reinforcements of the first, second and third storeys, fourth, fifth and sixth storeys, seven, eighth and ninth storeys, and tenth, eleventh and twelfth storeys were similar. This led to 81 (=3^4) types of beams and 81 (=3^4) types of columns. Therefore, for the performance assessments in this section, 729, 6561 and 6561 types of 5, 8 and 12 storey RC frames were considered, respectively (13851 frames in total).

5.1. Effect of initial structural cost on TLCC

All the above mentioned RC frames were analysed under the set of 20 selected accelerograms (see Table 5), and their TLCC was estimated based on the details provided in the previous section. The effect of the initial structural cost on TLCC of the 5, 8, and 12 storey RC frames is depicted in Fig. 3. The results suggest that there is no direct relation between the initial structural cost and TLCC, since the blind increase in the reinforcement ratios of the sections does not guarantee a reduction in the TLCC.

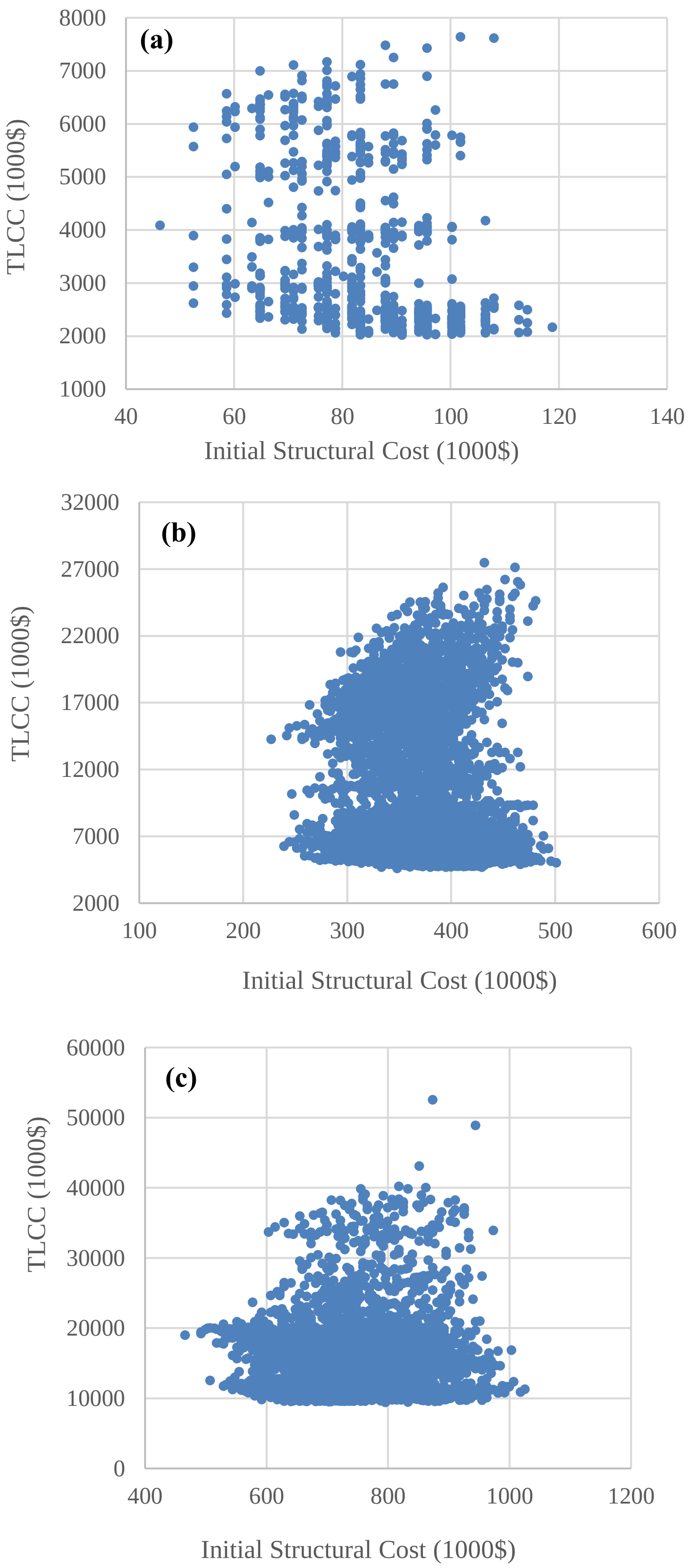
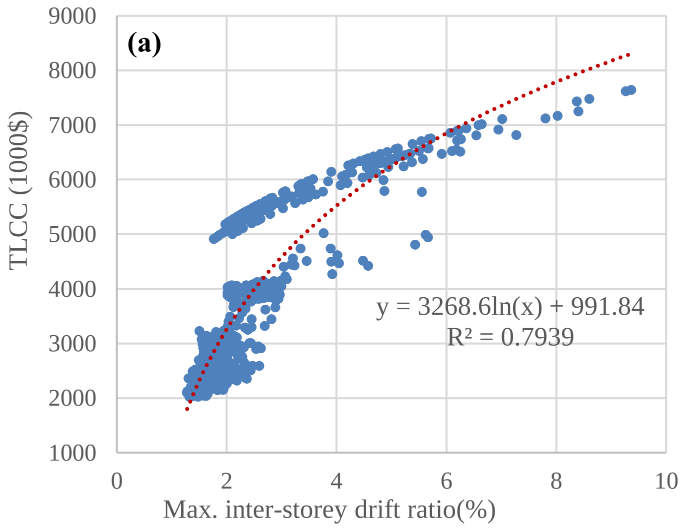
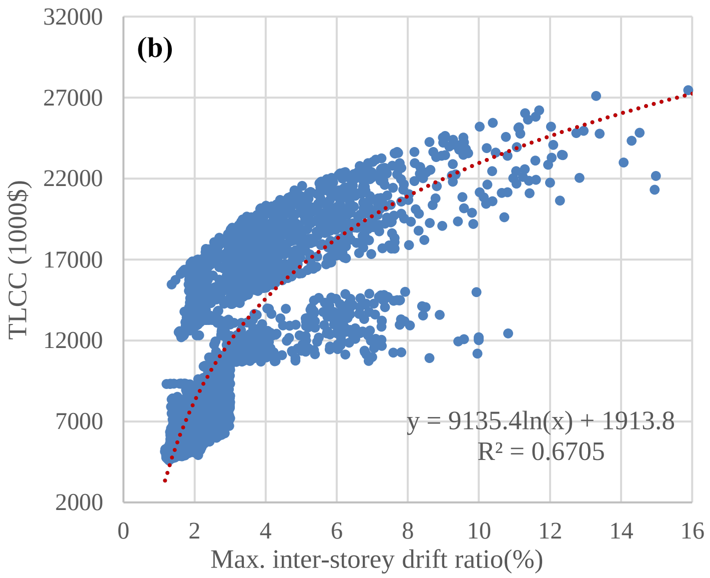


Figure 3. Effect of initial structural cost on TLCC of a)5, b)8, and c)12 storey RC frames, average of 20 selected earthquake

5.2. Effect of maximum inter-storey drift ratio on TLCC

The results of this study indicate that the maximum inter-storey drift ratio (or DI) of the buildings under the design earthquakes can considerably influence in the TLCC. Fig. 4 illustrates the variation in the TLCC of the 5, 8, and 12 storey RC frames as a function of the maximum inter-storey drift ratio. The results show a natural logarithmic trend, which implies that increasing the maximum inter-storey drift is generally accompanied by an increase in the TLCC of the structures.





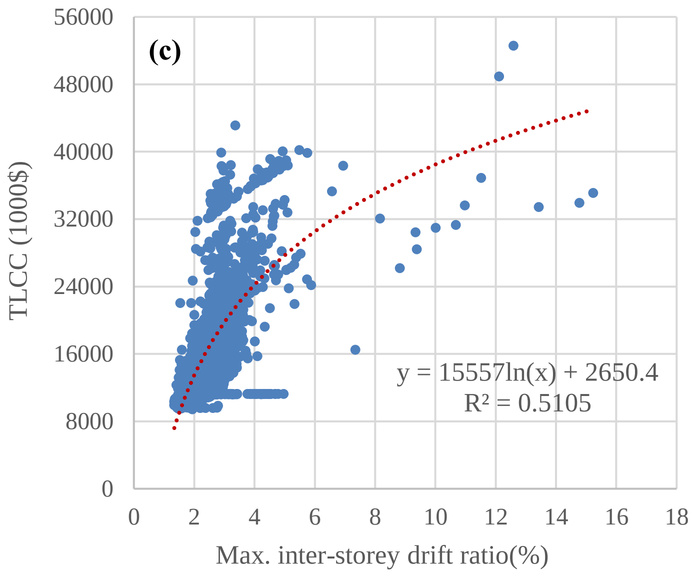
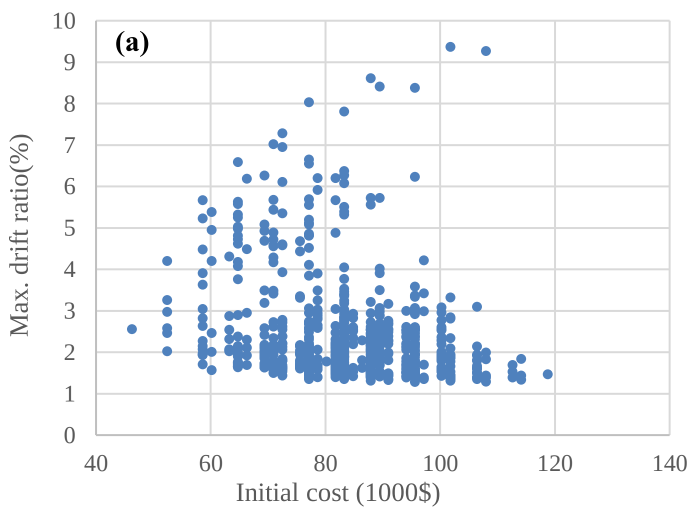
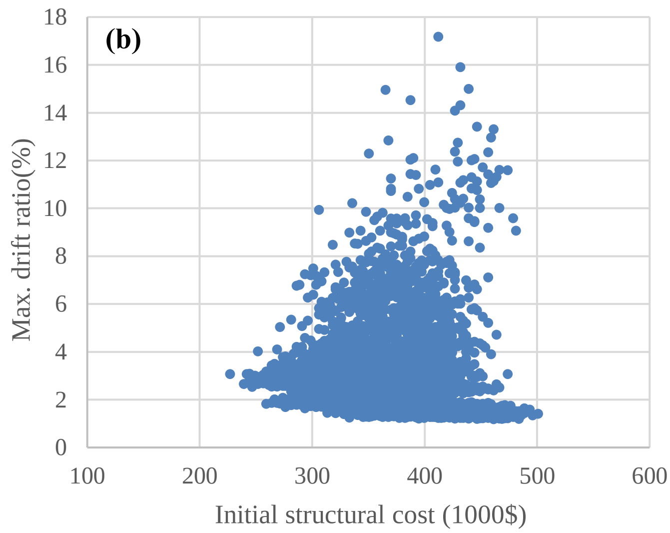


Figure 4. Effect of maximum inter-storey drift ratios on the TLCC of a)5, b)8, and c)12 storey RC frames, average of 20 selected earthquake

5.3. Effect of initial structural cost on maximum inter-storey drift ratio

It is expected that by increasing the initial structural cost of the structure, inter-storey drift ratios are decreased and performance level is enhanced. However, Fig. 5 displays that the blind increasing of the reinforcement ratios will not necessarily decrease the maximum inter-storey drift ratios (and damage) under the design earthquakes. This highlights the importance of developing performance-based design methodologies for RC frames using appropriate design parameters.





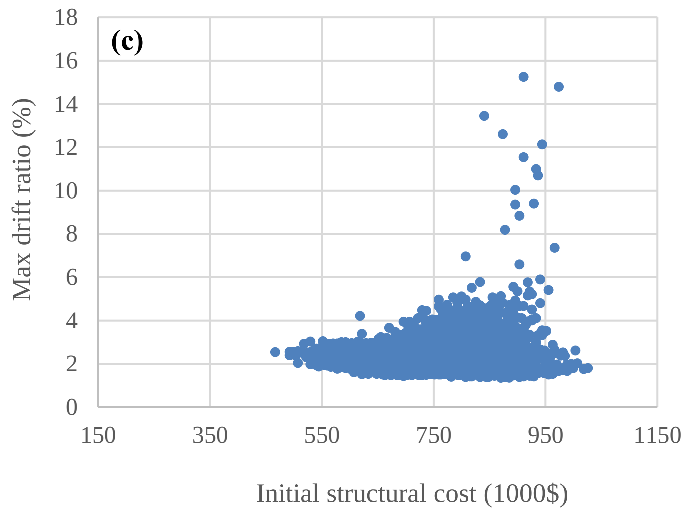
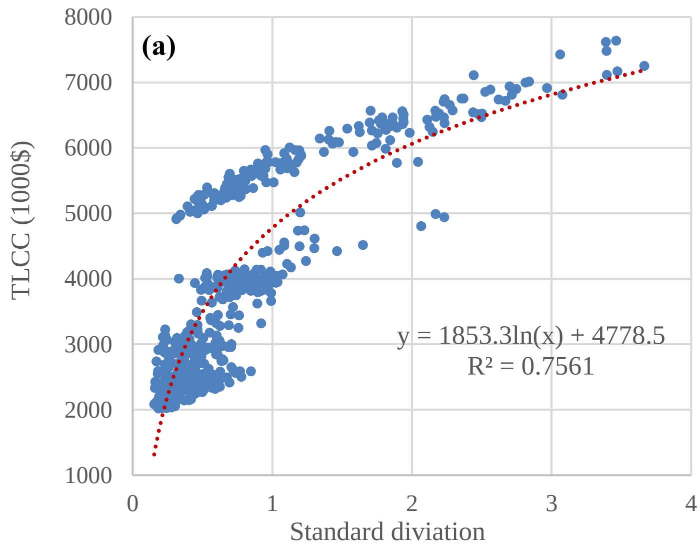
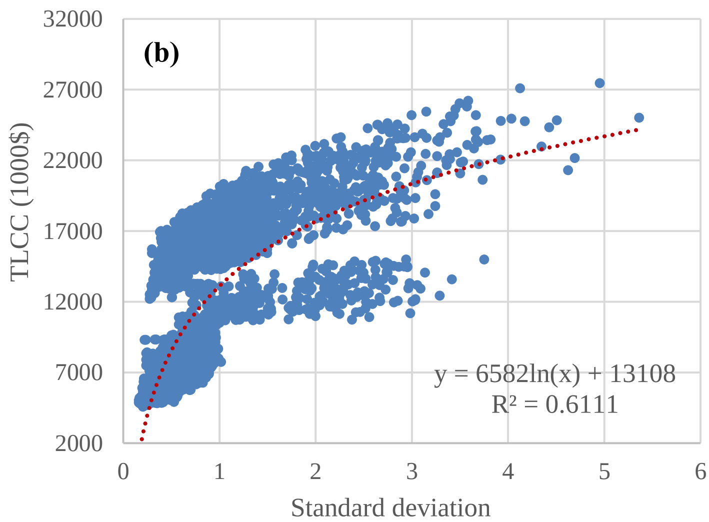


Figure 5. Effect of the initial structural cost on the maximum inter-storey drift ratios of a)5, b)8, and c)12 storey RC frames, average of 20 selected earthquake

5.4. Effect of the standard deviation of maximum inter-storey drift ratios on TLCC

Previous studies on the seismic performance-based optimisation of different structural systems demonstrated that, for the same amount of structural materials, structures with more uniform distribution of deformation demands in general exhibit less damage in comparison with those designed conventionally [e.g. 35-39]. To assess the effect of uniform demand distribution on the life cycle cost, the TLCC of the designed RC frames are compared in Fig. 6 in terms of the standard deviation of maximum inter-story drift ratios of the storeys under the design earthquakes. It is illustrated that the standard deviation of maximum inter-storey drift ratios of storeys has a high correlation with TLCC. It can be seen that by moving towards a more uniform height-wise distribution of maximum inter-storey drift ratios (i.e. lower standard deviations), the TLCC of the frames generally decreases following a logarithmic trend. It means that having storeys weaker or even stronger than the required level can result in an increase in the TLCC. This confirms that the concept of uniform damage distribution (UDD) can be used for the seismic optimum design of RC frames with the TLCC objective function. This will be discussed in the following section in in more details.





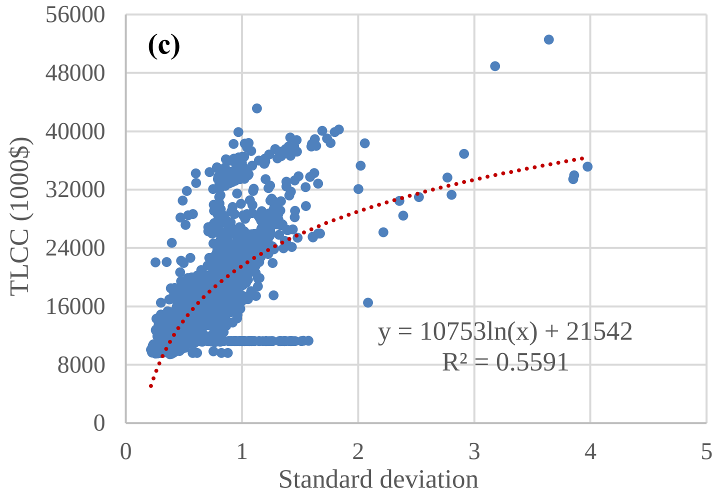


Figure 6. Effect of the initial structural cost on the maximum inter-storey drift ratios of a)5, b)8, and c)12 storey RC frames, average of 20 selected earthquake

6. Optimum Performance-based Seismic Design Method

Hajirasouliha et al. [35] proposed a practical optimisation method for more efficient design of RC frames based on the concept of uniform damage distribution (UDD). In their method, based on the selected performance target, the longitudinal reinforcements of the beam and column elements are gradually redistributed from strong to weak parts of the RC frame using the following equations:

$$[(\rho_{beam})_i]_{n+1} = \left[\frac{(DI_b)_i}{(DI_b)_{ave}}\right]^{\alpha} [(\rho_{beam})_i]_n \tag{11}$$

$$[(\rho_{col})_i]_{n+1} = \left[\frac{(DI_c)_i}{(DI_c)_{ave}}\right]^{\beta} [(\rho_{col})_i]_n \tag{12}$$

where $[(\rho_{beam})_i]_n$ and $[(\rho_{col})_i]_n$ are the longitudinal reinforcement ratio of the ith beam or column at nth iteration; $(DI_b)_i$ and $(DI_c)_i$ represent the selected damage index for the ith beam or column; and $(DI_b)_{ave}$ and $(DI_c)_{ave}$ are the average of damage indices for all beam and column elements, respectively. α and β are convergence parameters (usually ranging from 0 to 1), which are used to improve the convergence of the optimisation problem. It was shown that, for the same amount of material, the optimum design methods experience considerably less total damage compared to their code-base design counterpart [35].

It was shown in the previous section that generally there is a correlation between reducing the standard deviation of inter-storey drifts (i.e. more uniform distribution of the selected DI) and TLCC of the designed structure. Based on this conclusion, for the first time, the concept of UDD is further developed for optimum performance-based design of RC frames based on TLCC using the following algorithm:

i) The initial structure is preliminary designed for gravity and seismic loads. The dimensions of the sections are designed to satisfy the drift limit ratio of the selected seismic code under the design hazard level earthquake, while the longitudinal reinforcement ratios are close to the minimum allowable limit. This means that it is possible to increase the resistance of the elements by increasing the reinforcements.

ii) Non-linear dynamic analyses are conducted on the RC frame under the set of 20 accelerogrames with the PGA levels mentioned in Table 3. Subsequently, the average of maximum inter-storey drift ratios and the average of maximum floor accelerations are calculated and used to estimate TLCC as discussed in previous sections.

iii) Based on the concept of uniform damage distribution (UDD), the distribution of longitudinal reinforcement in beam and column members are modified using the following equations:

$$A_{c,i,new} = A_{c,i,old} \times \left(\frac{DR_i}{DR_{target}}\right)^{\alpha} \times \left(\frac{TLCC_i}{TLCC_{ave}}\right)^{\beta} \tag{13}$$

$$A_{b,i,new} = A_{b,i,old} \times \left(\frac{DR_i}{DR_{target}}\right)^{\alpha} \times \left(\frac{TLCC_i}{TLCC_{ave}}\right)^{\beta} \tag{14}$$

where, $A_{c,i,new}$ and $A_{c,i,old}$ are the area of new and old longitudinal reinforcement of the columns of the i^{th} storey; $A_{b,i,new}$ and $A_{b,i,old}$ are the area of new and old reinforcement of the beams of the i^{th} storey; DR_i is the average of the maximum inter-storey drift ratios of the i^{th} storey under the set of the design earthquakes; DR_{target} is the target inter-storey drift ratio for the selected earthquake intensity level; $TLCC_i$ and $TLCC_{ave}$ are the TLCC of the i^{th} storey and the average of TLCC in all storey levels; and α and β are the parameters which control the convergence of the optimisation problem [36, 37]. At each step, the reinforcement ratios are also calculated and checked to be within the minimum and maximum limits suggested by the selected design code. In this study, it is assumed that adequate shear confinement reinforcement is provided for each member. The appropriate values for α and β parameters in the equations (13) and (14) should be identified to ensure the convergence of the optimisation problem as will be discussed in the next section.

iv) Using the calculated reinforcement values in the previous step, a new RC frame model is developed and the optimization design is repeated from the step (ii). The optimisation process stops when the changes in the area of the reinforcement is negligible for all beam and column members. At this stage, it is assumed that the converge is achieved.

The objective of the seismic design for the office buildings used in this study is considered to be Life Safety (LS) under design earthquakes with the occurrence probability of 10% chance of in 50 years (see Table 3). The target inter-storey drift ratio was taken as 2% in accordance with seismic design guidelines such as FEMA356 [1]. However, the proposed design methodology is general and other performance targets can be adopted.

7. Numerical Results

The optimum design procedure introduced in the previous section was applied for seismic design of the 5, 8, and 12-storey RC frames using different α and β are the convergence parameters (α and β). While using very small convergence parameters can significantly increase the number of analyses required to achieve the optimum solution, large convergence parameters may result in divergence of the optimisation problem. Figs. 7 to 10 illustrate the variation of TLCC, structural cost and maximum inter-storey drift ratio during the optimisation of the selected RC frames using different convergence factors (α=0.15, β= 0.08; α=0.6, β= 0.5; α=1.0, β= 0.9; α=1.6, β= 1.5). The results in Figs. 6 and 7 indicate that for the 5 and 8-storey frames (low to mid-rise buildings) the proposed optimisation method did not converge when large convergence factors (α=1.6, β=1.5) were used. On the other hand, the convergence speed was very slow by using very small convergence factors (α=0.15, β=0.08). For these structures, a good convergence was observed for α=0.6, β=0.5 and α=1.0, β=0.9. It is shown in Fig. 8 that 12-storey frame (high–rise building) was more sensitive to the selected convergence factors and convergence was not achieved by using α=1.0, β=0.9 and α=1.6, β=1.5. The reason can be attributed to the effects of higher modes and more number of design parameters in tall buildings. Similar to the previous case, the convergence rate was very slow when very small convergence factors (α=0.15, β=0.08) were used. Based on the results, α=0.6, β=0.5 could provide the best convergence rate with no significant fluctuations.

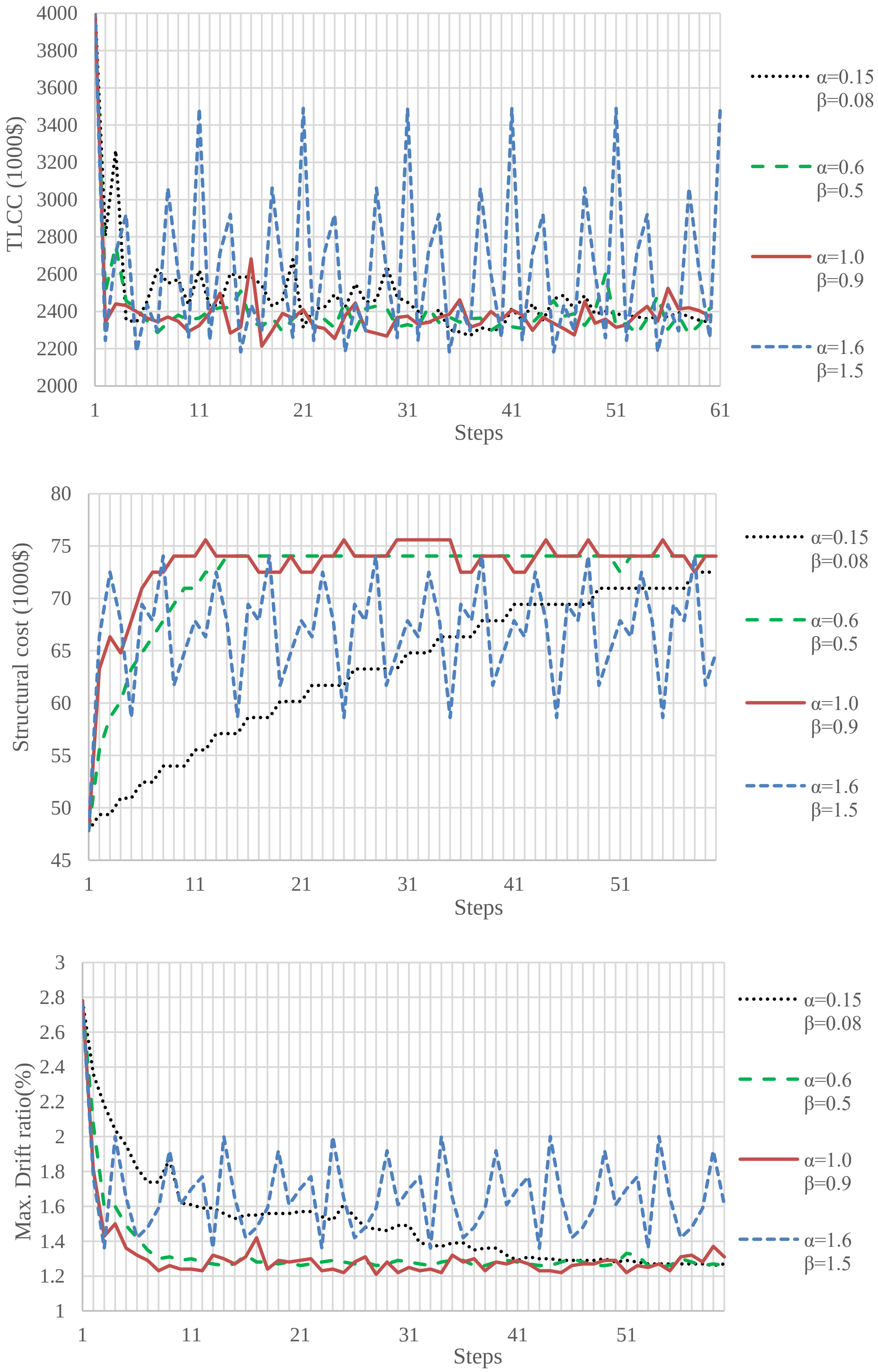


Figure 7. Variation of TLCC, structural cost and maximum inter-storey drift ratio during the optimisation of 5 storey RC frame using different convergence factors

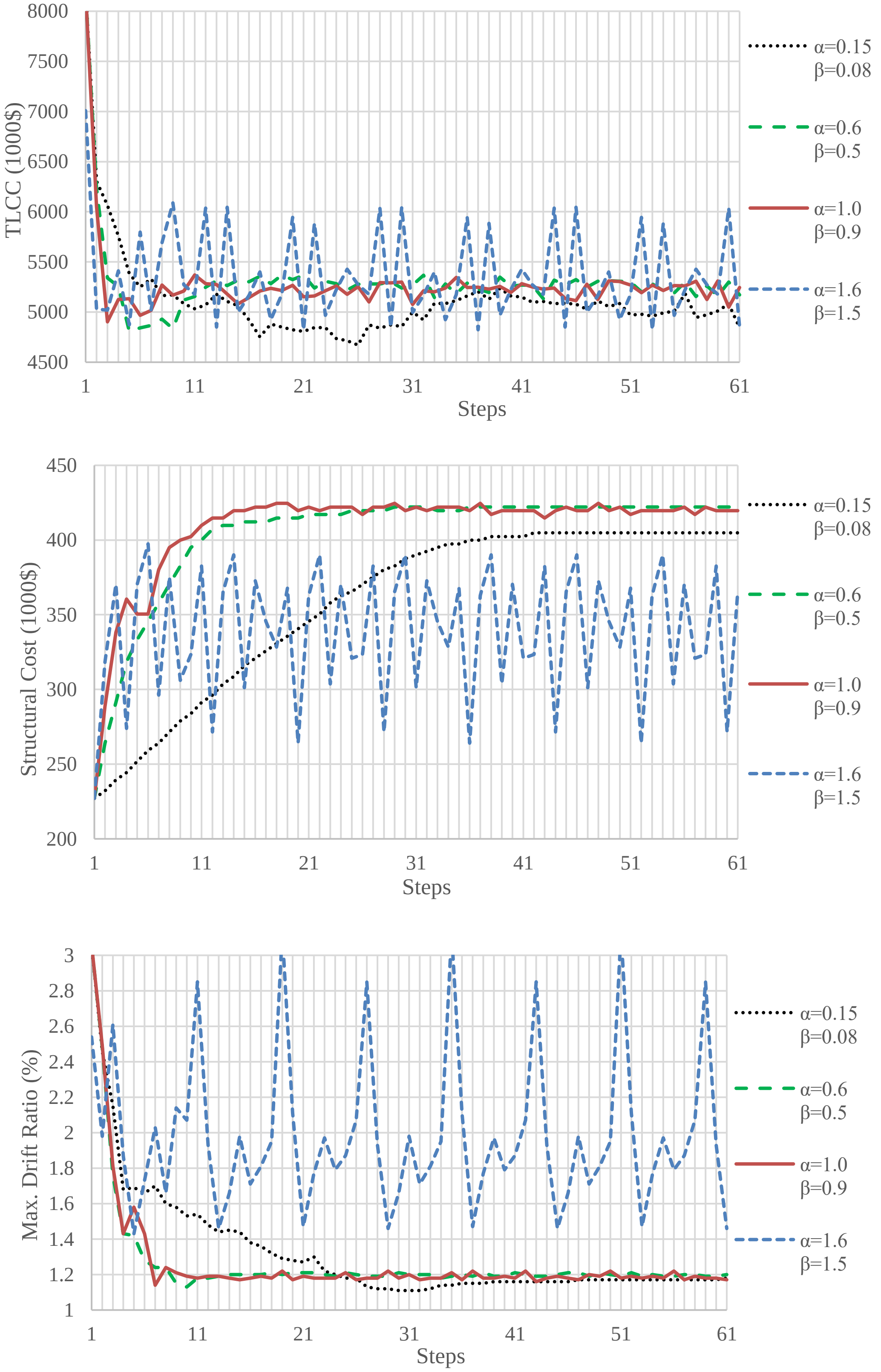


Figure 8. Variation of TLCC, structural cost and maximum inter-storey drift ratio during the optimisation of 8 storey RC frame using different convergence factors

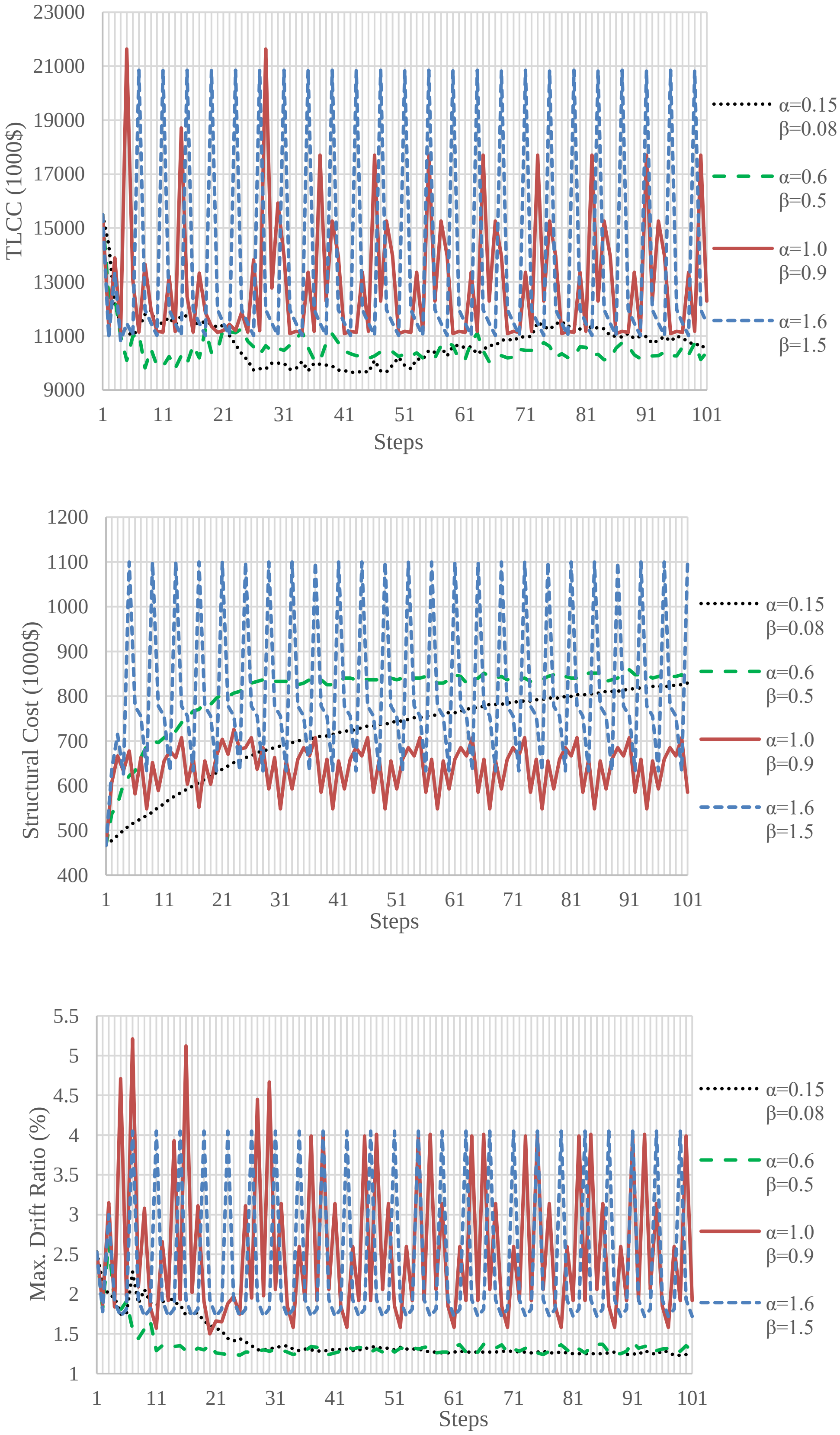


Figure 9. Variation of TLCC, structural cost and maximum inter-storey drift ratio during the optimisation of 12 storey RC frame using different convergence factors

For better comparison, Tables 6 to 8 compare the TLCC, maximum inter-storey drift ratio and structural cost of the preliminary code-based designed frames with those optimised using different convergence factors.

Table 6. Result of the optimum seismic design of the 5st. RC Frame

	Initial structure	Optimum seismic designed			
		α=1.60 β=1.50	α=1.00 β=0.90	α=0.60 β=0.50	α=0.15 β=0.08
Number of steps to converge	-	Not Converged	10	15	60
TLCC (1000$)	4018	-	2215	2290	2352
Inter-storey drift ratio (%)	2.76	-	1.26	1.26	1.26
Structural cost (1000$)	47.8	-	72.5	74.1	72.5

Table 7. Result of the optimum seismic design of the 8st. RC Frame

	Initial structure	Optimum seismic designed			
		α=1.60 β=1.50	α=1.00 β=0.90	α=0.60 β=0.50	α=0.15 β=0.08
Number of steps to converge	-	Not Converged	15	20	30
TLCC (1000$)	7008	-	5082	4835	4840
Inter-storey drift ratio (%)	2.54	-	1.20	1.15	1.13
Structural cost (1000$)	227	-	425	382.6	380.2

Table 8. Result of the optimum seismic design of the 12st. RC Frame

	Initial structure	Optimum seismic designed			
		α=1.60 β=1.50	α=1.00 β=0.90	α=0.60 β=0.50	α=0.15 β=0.08
Number of steps to converge	-	Not Converged	Not Converged	20	105
TLCC (1000$)	15500	-	-	10071	9660
Inter-storey drift ratio (%)	2.53	-	-	1.24	1.30
Structural cost (1000$)	466.6	-	-	833	722

In Fig. 3 the lowest TLCC for the 5 storey building was obtained from 729 analysis of different structures, wherein the optimal design of RC frames, a RC frame was obtained from only 16 analysis. Although the TLCC of the optimal structure is approximately 10% more than the minimum TLCC of Fig. 2, but the optimal structure has the TLCC about 50% lower than the original structure and, obtained only from only a few numbers of analysis.

It can be seen from Figs. 6 to 9 that the proposed optimisation method could considerably reduce both the TLCC and maximum inter-storey drift ratio of the frames in only a few steps (less than 20 non-linear dynamic analyses). This highlights the low computational cost of the method compared to the conventional optimisation techniques such as Genetic Algorithm (GA), which generally require over 1000 non-linear dynamic analyses to converge to the optimum solution [37, 40]. The results indicate that the TLCC of the optimum design 4, 8 and 12-storey frames is up to 45%, 31% and 38% less than their code-based design counterparts, respectively. The maximum inter-storey drift ratios (selected DI) are also reduced by around 50% in the optimum design frames to satisfy the predefined performance target. This implies that the optimised structures not only require less TLCC compared to the initial design solutions, but also they suffer considerably less damage under the design earthquakes. It can be also noted that the initial structural costs generally increased by using the proposed optimisation method, while the TLCC was always decreased. This highlights the fact that optimisation based on the initial structural costs may not necessarily lead to the optimum design solution over the effective life of the structure.

The results of this study, in general demonstrate the reliability of the proposed performance-based optimisation method to minimise both damage and life-cycle costs of RC structures in seismic regions.

8. Summary and Conclusions

A practical methodology was developed for optimum performance-based design of RC structures with minimum structural and non-structural damage and total life-cycle cost (TLCC). The proposed method is based on the concept of uniform damage distribution (UDD), in which the structural materials are gradually redistributed using an adaptive iterative procedure to exploit the full capacity of all members. In this study, the life-cycle damage costs included the expected loss of the structural, non-structural, furniture, rental, commercial, minor injuries, major injuries leading to disability, human fatalities, and social costs. First the seismic response of a wide range of 5, 8 and 12 storey RC frames (13851 structures in total) was investigated through incremental dynamic analyses under a set of 20 earthquake excitations. It was shown that the blind increase of the reinforcement ratios (i.e. increasing the initial structural cost) does not guarantee a reduction in the maximum inter-storey draft ratios (selected DI) and TLCC of the structures. However, the results suggested that increasing the maximum inter-storey drift is generally accompanied by an increase in the TLCC. Subsequently, the efficiency of the proposed optimisation method was demonstrated by optimising 5, 8 and 12 storey RC frames under the selected earthquake records representing the design spectrum. It was shown that by using appropriate convergence factors (α and β), the optimum solution is generally obtained in only a few steps that demonstrates the low computational cost of the method compared to conventional optimisation techniques such as Genetic Algorithm (GA). The results indicated that the TLCC of the optimum design 4, 8 and 12-storey frames was up to 45%, 31% and 38% less than their code-based design counterparts, respectively, while the maximum inter-storey drift ratios were also reduced by around 50%. Although TLCC was considerably reduced for the optimum design frames, the initial structural cost was generally higher than the conventional designs. This highlights the fact that optimisation based on the initial structural costs does not necessarily lead to the best design solutions when the whole life-cycle cost is considered.

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