ADAPTIVE MULTI-LEVEL PERFORMANCE-BASED SEISMIC OPTIMISATION OF RC FRAMES

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Abstract

Current seismic design guidelines are based on equivalent "force-based" design principles, which primarily aim to satisfy life safety requirements under a specific seismic hazard level. As a result, structural damage is not controlled under different earthquake intensity levels, and this can lead to unacceptable structural performance and high economic loss during earthquakes. Moreover, optimum seismic design of non-linear structures under seismic excitations is challenging and computationally expensive. This study presents an adaptive method for multi-level performance-based optimisation of RC frames to minimise both structural damage and material usage by considering predefined seismic performance and practical design constraints. The proposed method is based on the concept of Uniform Damage Distribution (UDD) that exploits the full capacity of structural elements by gradually shifting material from strong to weak parts. Section sizes and reinforcement ratios are simultaneously optimised to satisfy Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) under multiple seismic hazard levels, using ASCE-41 target limits for inter-storey drifts and plastic rotations. The proposed method is demonstrated by optimising the design of 5- and 10-storey RC frames under a set of spectrum compatible design earthquake records. In both cases, the optimum design solution was obtained in only a few steps. Compared to the initial designs by current codes, the optimum solutions required up to 20% and 43% less concrete volume and steel reinforcement weight, respectively. They also exhibited a considerably lower global damage index (up to 88%), less concentrated maximum inter-storey drift (up to 58%) and less maximum plastic rotations (up to 78%). The proposed method provides a low-computational cost solution for more efficient and practical seismic design of RC frames.

Keywords Structural optimisation; Reinforced concrete frames; Multi-level performance based design

1. Introduction

The new generation of seismic design codes produced since the 1980s (e.g. Eurocode 8, IBC 2021, Chinese code GB 50011) utilise equivalent "force-based" design to determine the additional actions from the design earthquakes. Hence, they mainly aim to achieve a "life safety" design goal under a specific seismic hazard level (i.e. 10% probability of exceedance in 50 years), which cannot guarantee structural safety in future earthquakes with higher intensity levels. Furthermore, this approach cannot directly control structural seismic performance (i.e. member deformations and storey lateral drifts) and in turn efficiently manage structural and non-structural damage in earthquake events. Another important factor in seismic design is high economic loss due to structural and non-structural damage, even when the "life safety" objective is achieved. For example, during the Christchurch 2011 earthquake, it was found that through structures successfully protected occupants' lives, the economic costs on regional recovery and reconstruction were extremely large and approximately equivalent to 19% of New Zealand's GDP (Stevenson et al., 2014).

Performance-based design (PBD) is considered the future direction in seismic resistant design, and its approach has been introduced in several recent design guidelines (i.e. ATC-40, ASCE 41, FEMA 356). This design approach is able to address some of the limitations in conventional "force-based" seismic designs, in which design criteria are expressed in terms of a set of performance objectives that directly relate to specific building behaviour requirements (i.e. immediate occupancy, life safety, collapse prevention) under different seismic hazard levels. In performance-based design methodology, the pre-defined performance objectives are satisfied by directly checking structural seismic performances (i.e.

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inter-storey drifts, element plastic rotations or loads) against target limits. Hence, it provides more direct control of structural and non-structural damage. Although the PBD guidelines provide better control on structural damage under various multiple seismic hazard levels, current design generally adopts a “trial-verification-modification” process. This is time-consuming and may also lead to inefficient design solutions.

Reinforced concrete (RC) frames are the most common structural systems used worldwide. Interest relating to the optimisation design of RC frames has increased in the last 20 years. Compared to traditional seismic design, optimum designs use computational techniques to achieve a cost-effective and reliable seismic resistance in structures. For example, Genetic Algorithms (GA) that are inspired by a biological process are widely used in structural optimisation methodologies. In such design optimisation frameworks, multiple discrete design variables including section dimensions, diameter and number of longitudinal and transverse reinforcement bars are considered. Optimum design solutions are decided by searching from pre-determined databases of design variables and checking against a set of design constraints (Mergos, 2018, 2017). To minimise initial construction costs of RC frame-shear-wall structures, Li et al., (2010) divided the entire design optimisation process into strength optimisation and stiffness optimisation. Genetic Algorithm (GA) and Optimality Criteria (OC) were applied to optimise section sizes and rebars details, respectively. Other optimisation methodologies such as Evolution Strategies (ES), were also applied to optimise design variables in beams and columns, respectively, for RC frames under seismic loads (Fragiadakis and Papadrakakis, 2008).

Zou et al., (2007) minimised total material cost for RC frames by utilizing an Optimality Criteria (OC) performance-based methodology. In the optimisation process, objective functions subjected to design constraints were first converted into an unconstrainted Lagrangian function. The stationary conditions of the function were then solved to remodify specific design variables at each iteration. (Papazafeiropoulos et al., 2017) employed a gradient-based first-order optimisation methodology to optimise structural stiffness in each storey and achieve uniform distribution of dissipated energy for RC frames.

In general, search-based optimisation methodologies such as GA and ES are computationally expensive as thousands of analysis iterations are required, while the efficiency and accuracy of design optimisations are also significantly affected by the pre-defined search space. Other optimisation methodologies previously utilised in this field (e.g. OC, first-order algorithm) are normally categorised as gradient-based optimisation. Such optimisation techniques require gradient calculations on both objective function and specific design constraints at each iterative step. This may lead to high computational demand especially in the case of a nonlinear structural system as a complex mathematical model is required. High computational efforts required in all aforementioned optimisation techniques may prevent engineers from utilizing optimisation methodologies in practical seismic design.

It also should be noted that the optimum design of RC frames can be very challenging since complex non-linear structural behaviours (i.e. cracking of concrete, yielding of reinforcement) need to be considered. Some previous studies optimised steel reinforcement as the main and only design variable for RC frames, while its cross sectional sizes were initially designed based on current design codes and kept unchanged in the optimisation process (Bai et al., 2016; Hajirasouliha et al., 2012). However, section size and reinforcement weight cannot work as independent design variables, as both have significant impact on structural ductility under seismic loads.

The objective of this study is to develop a low computational cost multi-level performance-based seismic design optimisation of multi-storey RC frames based on the concept of Uniform Damage Distribution (UDD). According to the proposed design method, pre-selected design variables (i.e. section size, reinforcement ratios) are updated iteratively in each storey until a more uniform height wise distribution of specific performance is achieved. In this manner, material capacities in most of the storeys are fully exploited, which in turn minimises the total material usage while ensuring that structural damage is minimised by satisfying performance and design constraints under multiple seismic hazard levels ranging from elastic to inelastic states. Both inter-storey drift and plastic rotations in elements are considered as performance parameters to control structural damage at global and local levels, respectively. The efficiency of the proposed method is demonstrated by optimising the design of 5- and 10-storey RC frames under a set of spectrum compatible design earthquake records.

2. Performance-based design optimisation methodology

Design variables

This study considers both longitudinal reinforcement in beam ($\rho_{\mathrm{b,top}}$ and $\rho_{\mathrm{b,bottom}}$) and column (ρ_c) elements and cross-sectional dimensions of each structural member as design variables in the optimisation process. In more detail, it assumes that each column element has a square cross section and its dimension (D) is one of the design variables. For the beam section, its depth (H) and width (B) are also considered as design variables. Moreover, the study assumes that each RC member has an adequate amount of transverse reinforcement at each iteration, which is approximately proportional to flexural reinforcement quantity. When the topology of the structure is pre-decided and kept unchanged in the design optimisation, the total material usage of the RC frame is minimised as:

$$\text{Minimise:}\ \ \mathrm{V_c}, \mathrm{W_s} \tag{1}$$

Where: V_c is the total concrete volume in the frame (in m^3), W_s is the total longitudinal reinforcement weights (in kg). The stated design objective is achieved while satisfying a set of performance constraints to control structural damage under multiple seismic hazard levels. To have an optimum design solution for practical application, the above-mentioned design variables should also satisfy design constraints required in Eurocode 8 for medium ductility class buildings (DCM) and in Eurocode 2. Therefore, the minimum dimension of beam and column elements is set as 250mm, the minimum and maximum longitudinal reinforcement ratios in columns are limited to 1% and 4%, respectively; the minimum tension reinforcement ratio in beams is limited to 0.3% to avoid brittle failure of rebar before concrete cracks. "Strong column-weak beam" design principle is satisfied by checking flexural stiffness of elements at each beam-column joint.

Performance parameters and corresponding target limits

The proposed design optimisation considers plastic hinge rotations in beams (θ_b) and column (θ_c) as one of the performance parameters to measure structural response at the element level under major and severe earthquakes. The inter-storey drift ratios ($\Delta_{max,i}$) are simultaneously employed as an effective parameter to control more global structural damage. To provide more accurate responses results, non-linear time history analysis (NTHA) is applied to predict specific seismic performance.

In this multi-level performance-based design optimisation, following ASCE 41-13 recommendations on deciding performance levels and their corresponding earthquake excitations, three performance objectives are expected to be satisfied: (i) Immediate Occupancy (IO) under minor earthquakes (50% probability of exceedance in 50 years), (ii) Life Safety (LS) under Design Basis earthquakes (DBE) (10% probability of exceedance in 50 years), (iii) Collapse Prevention (CP) under Maximum Considered earthquakes (MCE) (2% probability of exceedance in 50 years). According to the design philosophy in PBD, the "capacity" of a structure should be checked against "demand" of an earthquake at specific performance level. In this study, once the target performance level is decided (e.g. IO, LS, CP), the performance parameters are evaluated though NTHA, using the following formula:

$$\Delta_{max,i} = \frac{\delta_i - \delta_{i-1}}{h_i} \leq \Delta_{target} \tag{2}$$

$$\begin{bmatrix} \theta_I \\ \theta_J \end{bmatrix} = \begin{bmatrix} -k^p|_{x=0} l_{pI} \\ k^p|_{x=L} l_{pJ} \end{bmatrix} \tag{3}$$

Where: δ_i and δ_{i-1} is the relative maximum lateral displacement of two adjacent i and i-1 floor levels, respectively; and h_i is storey height at i^{th} floor. θ_I and θ_J are plastic rotations at ends I and J of a beam or column element, respectively; $x = 0$ and $x = L$ describe the locations of both ends of an element; l_{pI} and l_{pJ} are physical lengths of plastic hinges near ends I and J, respectively; $k^p|_{x=0}$ and $k^p|_{x=L}$ are plastic curvatures at both ends of the element. Plastic hinge rotation of an element (θ_b or θ_c) is then selected as the larger value between θ_I and θ_J.

The limiting values of plastic hinge rotation corresponding to specific performance objectives are determined by incorporating formulations of plastic rotation capacity of RC elements recommended in ASCE 41-13. In this study, the plastic rotation capacity for beam ($\theta_{target,b}$) and column ($\theta_{target,c}$) elements is mainly controlled by structural flexure. It increases with a reduction in axial load or an increasement on transverse reinforcement ratio. Details on the rotation capacities of RC beams and columns are given in Table 1 and Table 2, respectively, where P is the column axial load, A_g is the column cross section area, f_c' is concrete compressive strength, V is design shear force in columns, b_w is section width, d is distance between compression rebar to centroid of tension reinforcement, A_v is shear reinforcement area, s is spacing of shear reinforcement, ρ is tension reinforcement ratio, ρ' is compression reinforcement ratio, ρ_{bal} is reinforcement ratio producing balanced strain conditions, and V_b is beam shear force.

It should be noted that ASCE 41-13 only measures structural seismic performance by using plastic hinge rotations. The target limits of drift ratios (Δ_{target}) are decided according to design guidelines in ASCE 41-06 as 1%, 2% and 4% at IO, LS and CP performance levels, respectively.

$\frac{P}{A_g f_c'}$	$\frac{V}{b_w d\sqrt{f_c'}}$	$\rho = \frac{A_v}{b_w s}$	Performance Level	
			LS	CP
≤ 0.1	≤ 3	≥ 0.006	0.045	0.060
≥ 0.6	≤ 3	≥ 0.006	0.009	0.010
≤ 0.1	≤ 3	≤ 0.0005	0.010	0.012
≥ 0.6	≤ 3	≤ 0.0005	0.003	0.004

Table 1: Column plastic rotation capacity ($\theta_{target,c}$) (unit: rad) (ASCE, 2013)

$\frac{\rho - \rho'}{\rho_{bal}}$	$\frac{V_b}{b_w d\sqrt{f_c'}}$	Performance Level	
		LS	CP
≤ 0.0	≤ 3	0.025	0.050
≥ 0.5	≤ 3	0.020	0.030
≤ 0.0	≥ 6	0.020	0.040
≥ 0.5	≥ 6	0.015	0.020

Table 2: Beam plastic rotation capacity ($\theta_{target,b}$) (unit: rad) (ASCE, 2013)

Proposed methodology

During the seismic design process of RC frames, this study assumes that structures behave nearly elastically under minor earthquakes and experience inelastic response under major to severe earthquakes. Moreover, since the design variable “section size” plays a more dominant role in providing structural lateral stiffness and controlling elastic drift performance for RC frames; the quantity of steel reinforcement is a more effective variable for providing structural ductility to structures beyond the occurrence of first yielding. The entire design optimisation process can be divided into: (i) element size optimisation at the elastic phase, (ii) reinforcement ratios optimisation at the inelastic phase. The performance objective IO is satisfied by optimising element sizes. After that, performance objectives LS and CP are simultaneously satisfied by primarily modifying the reinforcement ratio in each element. The overall design optimisation procedure can be summarised as:

1. Establish an initial design under gravity and seismic loads that satisfies all required design constraints based on conventional seismic design codes, such as Eurocode 8.
2. Determine the magnitude of peak ground acceleration (PGA) corresponding to different performance objectives and seismic hazard levels. These values of PGA will be used to scale a set of spectrum-compatible earthquake records in nonlinear time history analysis.
3. Obtain the maximum seismic response (e.g. inter-storey drift, plastic hinge rotation) as an average of the maximum responses relevant to each selected earthquake record in order to capture record-to-record variability.
4. Apply an optimisation methodology based on the concept of Uniform Damage Distribution (UDD). Hence, the specific design variables (e.g. element size, reinforcement ratio) increase in storeys where the seismic responses exceed the limiting values at target performance levels; while they decrease in those storeys undergoing less response compared to target limits. As suggested in previous studies, in a nonlinear system, such remodification should be made gradually in an iterative process to avoid fluctuation issues (De Domenico and Hajirasouliha, 2021; Nabid et al., 2017). It should be noted that the design solution at each iteration should also check against all specific design constraints to achieve practical applicability.
5. Calculate the coefficient of variation (COV) of specific performance parameters among all storey levels in each iterative step. Repeat steps 2 to 4 until the difference of COVs is minimised and remains stable in subsequent iterations. As a result, a more uniform distribution of deformation is

achieved at least under one seismic hazard level. The final design is also checked to sustain gravity loads.

3. Modelling and assumptions

To investigate the efficiency of the proposed optimisation method, two 3-bay RC frames with 5 and 10 storeys were modelled with a uniform height of 3m, and details of geometry information are illustrated in Figure 1. To represent substandard buildings in high-seismic regions, the frames were designed with important class I and medium ductility class (DCM), subjected to seismic loads which were calculated by using Eurocode 8 design response spectrum for a medium seismic activity region (peak ground acceleration (PGA) equal to 0.4g). The frames were assumed to be located on soil type C, and to account for structural nonlinearly a behaviour factor q=3.9 was considered. The dead and live loads for intermediate storeys were taken to be 4.6 kN/m^2 and 2 kN/m^2, while for the roof the dead and live loads were reduced to 4 kN/m^2 and 0.7 kN/m^2, respectively. The nominal compressive strength of concrete and yielding strength of steel reinforcement were 30MPa and 500MPa, respectively. The initial frames satisfied safety, serviceability and durability design criteria of Eurocode 2 and 8.

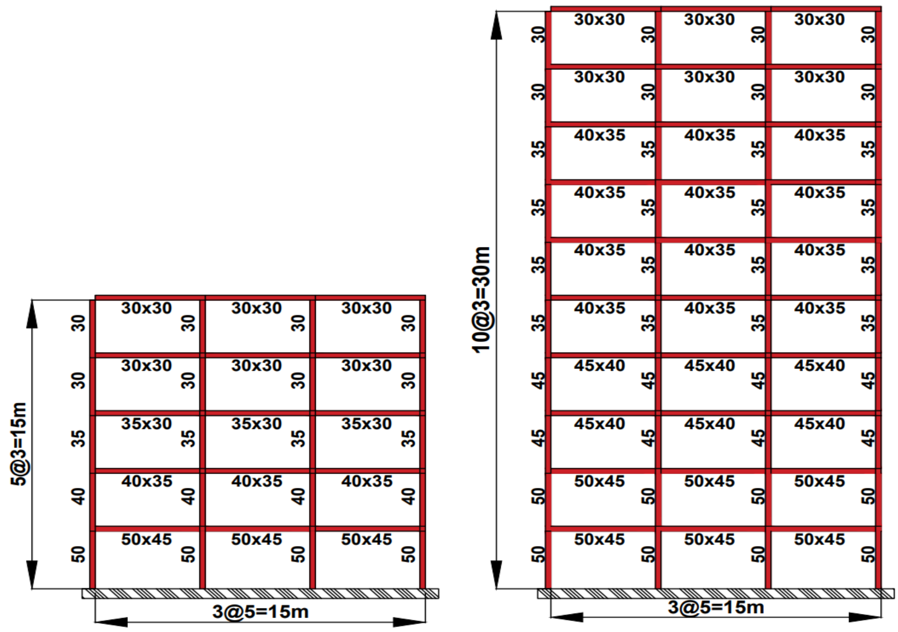

Figure 1: Geometry and dimensions of beam and column members of 5- and 10-storey RC frames (unit: cm)

The "Concrete02" and "Steel02" models, as proposed by (Filippou et al., 1983) and (Mohd Yassin, 1994) respectively, were used to simulate material properties of concrete and reinforcement steel in the finite element software OpenSees (McKenna et al., 2006), for modelling and analysis of the RC frames. Beam and column elements were modelled using distributed-plasticity force-based nonlinear finite element ("forceBeamColumn"). This element model utilises six "Modified Gauss-Radau" integration points distributed along the length of the element, allowing for the occurrence of nonlinearity at any location within specific plastic hinge region (Neuenhofer and Filippou, 1997). The physical length of the plastic hinge region is calculated in each iterative step following the equation provided in Eurocode 8, part 3:

$$l_{pl} = \frac{L_v}{30} + 0.17h + 0.24\frac{d_{bL}f_y\ (MPa)}{\sqrt{f_c}\ (MPa)} \tag{4}$$

Where: d_{bL}, f_y and f_c are mean diameter of tension reinforcement, concrete compressive strength and steel yield strength, respectively. L_v is the shear span at member ends, and h is the depth of the cross-section.

Rayleigh damping with a constant ratio of 5% was modelled in frames and was assigned to the first model and any model whose cumulative mass participation exceeds 95%. P-Delta effects were considered in the analysis. The effects of concrete cracking on flexural and shear stiffness of all elements were considered following recommendations from Eurocode 8.

4. Code-based design spectrum and selected earthquake records

In the proposed seismic design optimisation framework, the seismic hazard levels in a specific seismic region are represented by Eurocode 8-based response spectrum. To assess the seismic performance of the selected frames during the optimisation process, six seismic ground motion records that are fully compatible with the specific code-based response spectrum were generated using target acceleration spectra compatible time histories (TARSCTHS) (Papageorgiou et al., 2002). In addition, to demonstrate the efficiency of the proposed optimisation methodology, fifteen natural earthquake records were selected from the Pacific Earthquake Engineering Research Center (PEER) database (2020) and from the SIMBAD database (Smerzini et al., 2014). The details of these selected natural earthquake records were presented in Table 3, which includes information such as earthquake source, magnitude, location and other relevant parameters.

Figure 2 presents the elastic response spectrum for artificial and natural earthquake records, as well as their mean response spectrums. It is observed that the mean response spectra of both artificial and natural earthquake records compare well with the Eurocode 8 design spectrum, with difference within a ±10% tolerance, across a wide range of periods that cover the fundamental periods of the two selected RC frames. It is worthing nothing that the mean response spectra of artificial earthquakes provide a closer approximation to target design spectrum compared to natural earthquake, as natural ground motions generally exhibit more random characteristics. Overall, both generated artificial earthquakes and selected natural earthquakes can be considered as good representatives of the Eurocode 8-based design response spectrum for the specific seismic hazard level considered in this study, which corresponds to the DBE level with PGA of 0.4g. It should be noticed that, the records can be simply scaled to different seismic hazard levels by adjusting the PGAs to target levels, allowing for a realistic and comprehensive assessment of structural seismic performances.

No.	Earthquake	Mw	Station ID/component	PGA(g)	PGV (cm/s)	PGD (cm)
1	1992 Cape Mendocino	6.9	CAPEMEND/PET000	0.590	48.4	21.74
2	1999 Duzce	7.2	DUZCE/DZC270	0.535	83.5	51.59
3	1979 Imperial Valley	6.5	IMPVALL/HE04140	0.485	37.4	20.23
4	1989 Loma Prieta	6.9	LOMAP/G03000	0.555	35.7	8.21
5	1994 Nothridge	6.7	NORTHR/NWH360	0.590	97.2	38.05
6	1987 Supersition Hills	6.7	SUPERST/BICC000	0.358	46.4	17.50
7	1990 Manji Abbar	7.4	MANJIL/ABBAR-T	0.496	52.1	20.77
8	1999 Kocaeli	7.5	KOCAELI/DZC270	0.356	46.3	17.66
9	2000 Tottori Prefecture	6.6	TTR009/y	0.611	36.3	13.00
10	1995 Kebe Hyogo	6.9	JMA/y	0.832	91.1	20.36
11	2005 NW Off Kyushu	6.6	FKO006/y	0.279	57.7	16.75
12	1989 Loma Prieta	6.9	LGPC/x	0.531	51.5	55.21
13	2007 Niigata prefecture	6.6	NIG018/x	0.506	83.8	34.26
14	1994 Northridge	6.7	ST_24279/x	0.583	74.9	17.70
15	2000 South Iceland	6.4	ST_109/y	0.706	105.1	26.36

Table 3: Selected natural ground motion records

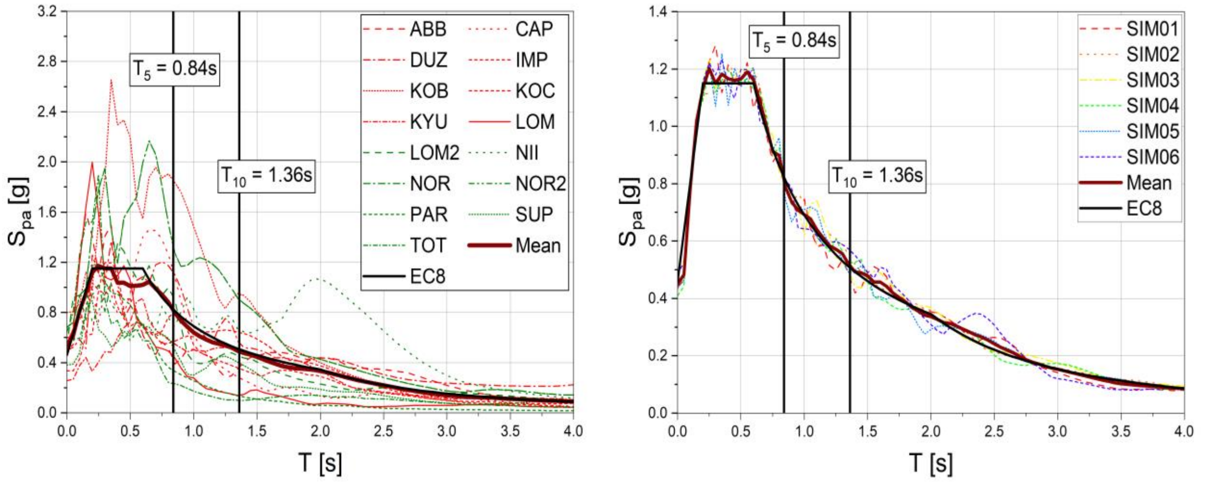

Figure 2: Eurocode 8 design response spectrum and acceleration spectra of natural records (left) and artificial records (right)

5. Optimum design solutions

Performance assessment

In this study, 5- and 10-storey RC frames are optimised using the proposed optimisation methodology for six spectrum-compatible artificial earthquake records. The average seismic responses (i.e. inter-storey drift and plastic rotation ratios) relevant to these six records are calculated and compared between the optimum design solution (named as "optimum design") and the initial design solution codified by Eurocode (named as "initial design"). Figure 3 presents a comparison of the height-wise distribution of maximum inter-storey drift ($\Delta_{\max}$) of 5- and 10-storey RC frames at IO, LS and CP performance levels.

To further investigate the efficiency of the proposed optimisation framework, the seismic performance of the same optimum design solution is also assessed under fifteen independent natural earthquakes that have a mean response spectrum close to the target code-based design spectrum. Figure 4 shows the average inter-storey drift ($\Delta_{\max}$) under a set of fifteen spectrum-compatible natural earthquakes, for 5- and 10-storey optimum and initial designs, at multiple performance levels.

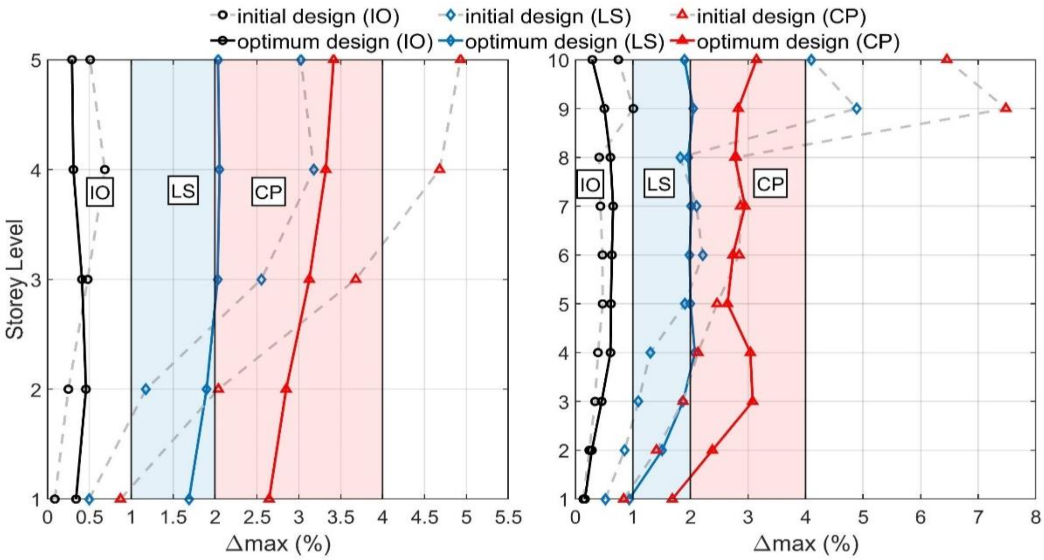

Figure 3: Height-wise distribution of Δ_{max} for optimum and initial design solutions for 5- and 10-storey frames, average results under six artificial records at IO, LS, CP performance levels

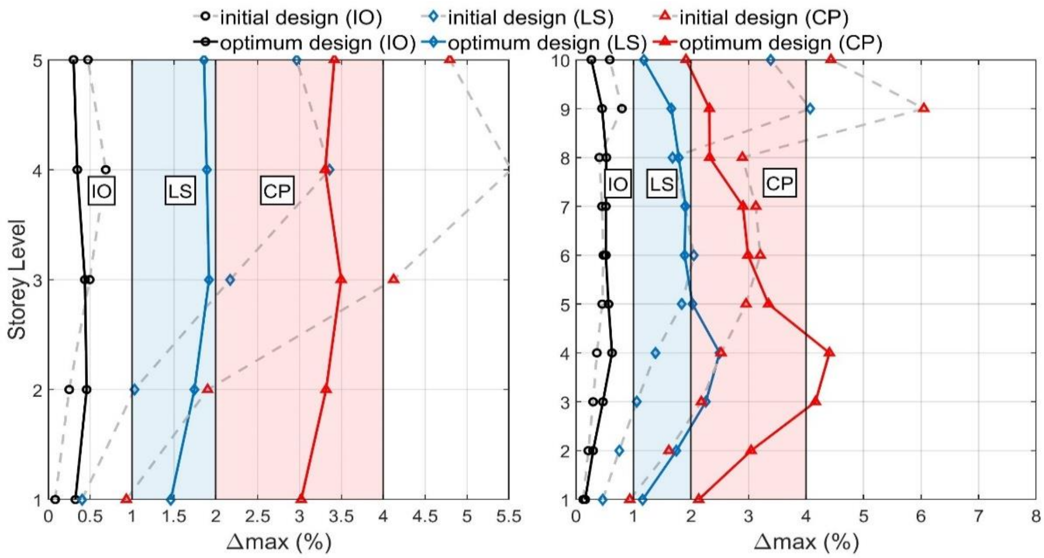

Figure 4: Height-wise distribution of Δ_{max} for optimum and initial design solutions for 5- and 10-storey frames, average results under fifteen natural records at IO, LS, CP performance levels

The results in Figure 3Figure 4 provide insights into the comparative performances of optimum and initial design solutions, under different earthquake intensity levels, using both artificial and natural earthquake records for performance assessment. It is observed that, compared to the initial designs, the frames designed optimally using the UDD optimisation methodology exhibit a more uniform height-wise distribution of inter-storey drift and less concentrated maximum drift. This can effectively prevent soft storey failure in a certain storey. It should be noted that optimum designs show similar trends of drifts under both artificial and natural spectrum-compatible records and they satisfy pre-determined performance targets within 10% tolerance under multiple seismic hazard levels. Nevertheless, the seismic responses more closely approach the target limits under the artificial earthquakes, as natural earthquakes have more random acceleration vibration characteristics. Overall, the UDD optimisation method utilised in this study helps to reduce maximum inter-storey drift ratios by up to 43% and 58% for 5- and 10-storey frames, respectively, when subjected to a set of spectrum-compatible earthquakes.

Figure 5 shows the maximum plastic rotation ratios (the ratio of demand to capacity) in columns for the same optimum solution subjected to the selected artificial and natural earthquake excitations, as well as the rotation ratios of the initial frames under the artificial records, at LS and CP performance levels. The results are obtained by calculating the average value of plastic rotations under a group of earthquakes, and they are the maximum plastic rotation ratio among all columns in each of the referenced frames. The error bars in the histogram represent the corresponding standard deviation of responses under either the selected six artificial earthquakes or the fifteen natural records.

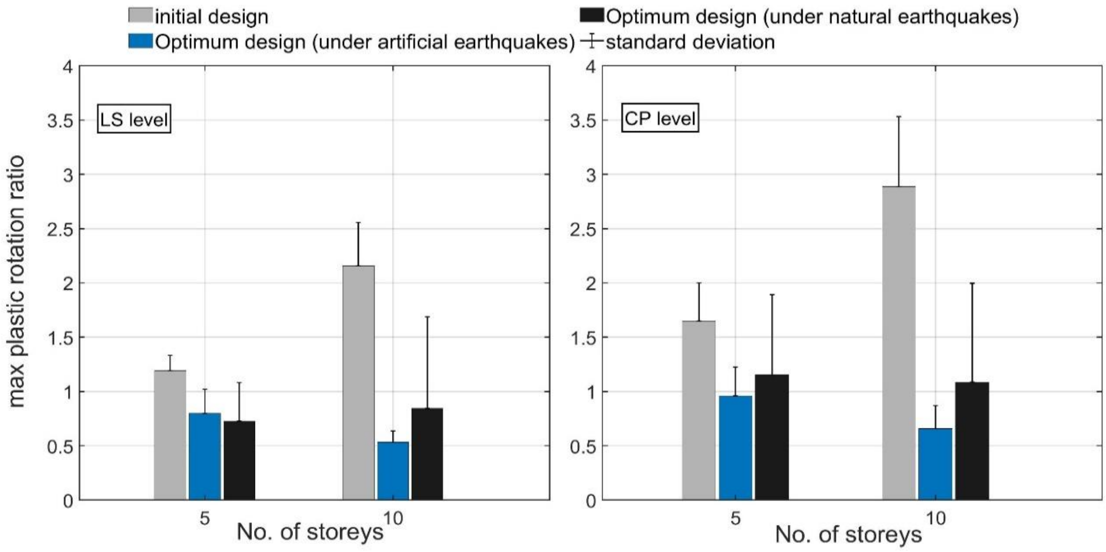

Figure 5: Max $\theta_{max,C}/\theta_{target,C}$ of 5- and 10-storey RC frames, average results (plus standard deviation) under six artificial records or fifteen natural records at LS and CP levels

As shown in the results, the proposed optimisation methodology is effective in reducing structural local damage at both LS and CP performance levels. Similar results are obtained for beam elements under a set of spectrum-compatible earthquakes (here not shown for brevity). Compared to the initial frames, the optimum design solutions reduce maximum plastic rotation ratios by up to 42% and 78% for 5- and 10-storey frames, respectively, while satisfying all specific performance targets under both artificial and natural earthquakes. A larger standard deviation is observed in the optimum designs under fifteen natural earthquake records, which is consistent with the fact that the natural records have a wider range of frequency contents and amplitudes. The results demonstrate that, the proposed UDD design optimisation can work efficiently and is reliable in achieving a unique optimum design solution for building subjected to various earthquake excitations with different intensity levels, once design spectra are specified.

Global damage index

To assess the efficiency of the UDD optimisation methodology in reducing overall structural damage under seismic loads, the damage index is calculated based on the concept of “demand versus capacity”, using the formula suggested by (Powell and Allahabadi, 1988):

$$D_i = \left(\frac{\delta_c - \delta_t}{\delta_u - \delta_t}\right)^b \quad (5)$$

where D_i is the cumulative damage index in a certain storey (at i^{th} level), δ_c, δ_t and δ_u are the calculated, threshold and ultimate values of specific damage parameter, respectively. The displacement-based ductility ratio is considered as the damage parameter, and a constant parameter b is determined based on experimental data, which is suggested as 1.5 for reinforced concrete frames (Cosenza and Manfredi, 2000). To estimate the level of damage that the entire structure is likely to experience, the global damage index is calculated as a weighted average of the cumulative damage index (D_i) at different storey levels:

$$D_g = \frac{\sum_{i=1}^{N} D_i w_i}{\sum_{i=1}^{N} w_i} \quad (6)$$

In the above equation, N represents the total number of storeys. w_i is the weightage assigned to i^{th} storey, which is taken as the dissipated energy in this study. The equation allows for consideration of not only the maximum value of a specific seismic performance parameter (i.e. inter-storey drift), but also the energy dissipation through plastic deformation. The global damage index D_g ranges from 0 (undamaged) to 1 (completely damaged). This approach is based on previous studies by Nabid et al., 2018; De Domenico and Hajirasouliha, 2021, which suggest that the amount of dissipated energy can be related to cumulative damage index (D_i) at each storey level.

Figure 6 compares the global damage indices of 5- and 10-storey RC optimum design frames with their initial designs at both LS and CP performance levels. The histograms show the average value of the damage index for the optimum design under six artificial earthquakes and fifteen natural earthquakes, along with the corresponding standard deviation of the damage index during these earthquakes indicated by error bars. The results demonstrate that the proposed optimisation methodology can significantly reduce structural damage up to 51% and 88% for 5 and 10-storey frames, respectively, under artificial earthquakes used in the design optimisation processes and a set of independent natural earthquakes. Compared to the results obtained for the artificial records, the optimum design solutions generally experience more damage, especially at CP performance level, under the selected natural earthquakes. A larger standard deviation is also observed in the damage index histograms. This is reasonable considering the independent characteristics and inherent uncertainties involved in natural earthquakes, as well as the nonlinearity of structural behaviour. Furthermore, these results are consistent with the concepts of UDD optimisation, as the structural materials in most storeys are efficiently utilised, and the specific performance parameters more closely approach the target limits at both element and structure levels, resulting in a lower global damage index.

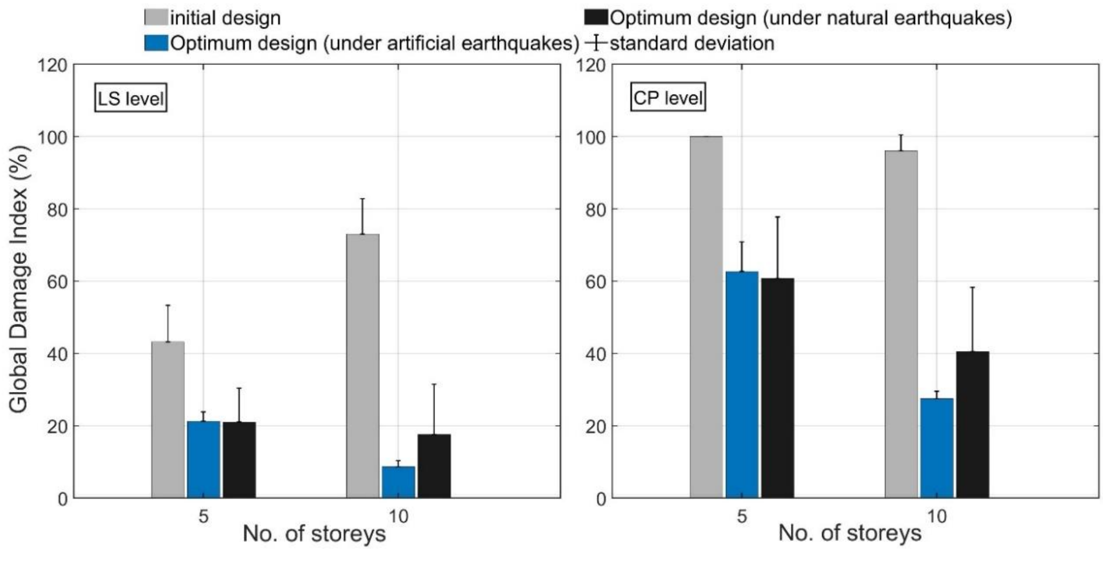

Figure 6: Global damage index (%) of 5- and 10-storey RC frame, average results (plus standard deviation) under six artificial records or fifteen natural records at LS and CP levels

Total material usage

In this section, the efficiency of the proposed optimisation methodology is investigated in terms of total material usage. It is shown in Table 4, the total concrete volume (m^3) and total reinforcement steel weights (kg) are calculated for 5- and 10- storey optimum and initial frames, respectively.

Design Alternative	Total Concrete Volume (m3)			Total Reinforcement steel weight (kg)		
	Initial design	Optimum design	Reduction	Initial design	Optimum design	Reduction
5-Storey	18.30	19.38	-5%	2480.6	1428.7	43%
10-Storey	43.95	35.14	20%	5809.0	3697.3	36%

Table 4: Total material usage for optimum and initial designs

The results demonstrate that optimum design solutions require a reduced total weight of reinforcement steel up to 43%, and less (20%) or similar total volume of concrete, compared to their code-based initial design counterparts.

6. Summary and conclusions

This study has developed a multi-level performance-based optimisation method based on the concept of Uniform Damage Distribution (UDD) for seismic design of RC frames. This design philosophy aims to iteratively modify structural materials in each storey in order to closely approach the performance target limits, while minimising structural damage by satisfying all performance and practical design constraints corresponding to multiple performance levels (i.e. IO, LS and CP). As a result, material in most stories is efficiently utilised at least at one performance level, which in turn minimises total material usage of the frames. The proposed optimisation framework considers two performance parameters in terms of plastic hinge rotation and inter-storey drift, simultaneously in the optimisation process to control both local and global structural damage. Design variables, including the cross-section size of beams and columns and longitudinal reinforcement ratio in each element, are optimised at the elastic and plastic phases, respectively. The efficiency of the proposed optimisation technique is demonstrated by assessing seismic performance of 5- and 10-storey frames under both artificial and natural earthquake records. The results indicate that the optimum designs exhibit a reduction of up to 58% and 78% for maximum inter-storey drift ratios and maximum plastic rotations, respectively, under a set of spectrum-compatible earthquakes. The optimum designs also experience a more uniform height-wise distribution of inter-storey drift ratios and plastic rotation ratios. Furthermore, the global damage index is significantly reduced (up to 88%) for both 5- and 10-storey frames, compared to the code-based initial design solution, indicating an improvement on overall seismic performance and damage. Meanwhile, the proposed optimisation method results in significant material savings, with up to 20% saving in concrete volume and up to 43% saving in steel reinforcement weight, which contributes to potential savings in initial construction costs.

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