Optimum Seismic Design of RC Frames Using Multi-Level Performance-Based Design Method

By

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Abstract

Despite much research efforts and new generations of seismic design guidelines, structures still experience extensive damage and incur significant economic losses during earthquakes (e.g. 2011 Christchurch earthquake) due to inefficient design and construction practices. This research aims to reduce the structural damage of reinforced concrete (RC) frames, particularly under major to severe earthquakes, which in turn assists to save economic losses caused by the potential structural and non-structural damage. The aim is achieved by providing a multi-level performance-based optimisation methodology.

This study develops a performance-based optimisation framework for minimising the initial material usages (costs) of multi-storey RC frames, while minimising structural damage by satisfying multiple performance objectives. The proposed methodology is characterised by computational efficiency, as optimum results can typically be achieved within a few iterative steps. This efficiency saves computational efforts particularly when non-linear time history analysis is involved in the optimisation, compared to conventional optimisation methodologies which often require thousands of analysis iterations. The optimisation method employs the concept of Uniform Damage Distribution (UDD). The novelty of the proposed optimisation framework lies in its implementation of the UDD approach to (i) simultaneously control both local structural seismic responses (plastic hinge rotations) and more global responses (inter-storey drifts); (ii) consider different hazard levels, ranging from minor (i.e. 50% probability of exceedance in 50 years) to major (i.e. 10% and 2% probabilities of exceedance in 50 years) earthquakes; and (iii) iteratively modify both section dimensions and longitudinal reinforcement ratios according to the performance results until that material capacities are fully exploited in each storey, achieving more uniform distributions of the response and satisfying multi performance targets. In addition to performance-based constraints, design constraints required in design guidelines (i.e. Eurocode) and practical design practices are also incorporated in the optimisation framework.

The efficiency of the proposed method is initially demonstrated through optimum designs of 3-, 5-, 10- and 15-storey RC frames under six spectrum-comparable artificial earthquakes. The results show that compared to conventionally designed counterparts, optimum structures exhibit lower maximum inter-storey drift and maximum plastic rotations by up to 58% and 78%, respectively, along with reduced global structural damage (up to 88%), while both responses are more uniformly distributed along storey levels. Sensitivity analysis of the optimum designs to the earthquake record selections shows that (i) using a single earthquake record may not lead to an acceptable seismic design, particularly for tall buildings, and (ii) both artificial and natural earthquakes can lead to optimum frames with similarly and satisfactory performances. Compared to code-based designs, the optimum designs reduce initial construction costs, including material and construction costs of concrete, reinforcement and formworks, by up to 15%, while reducing total life-cycle costs (the sum of initial construction costs and expected life-cycle damage losses) by up to 64%. The optimum designs consistently experience less global damage (up to 82%) when considering uncertainties in material (i.e. concrete and steel strengths) and geometry properties (i.e. area of longitudinal rebar). The effect of earthquake records variability is efficiently managed in this optimisation framework.

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$\overline{\ln()}$	Average results of the natural logarithm of structural response results, here referring to maximum floor acceleration
$\Delta_{\text{ultimate,i}}$	Ultimate drift capacity in i^{th} storey
$\Delta_{\text{yield,i}}$	Yielding drift in i^{th} storey
Δ_{k}	Structural response results at the k^{th} iteration
Δ_{max}	Maximum inter-storey drift ratio
Δ_{target}	Target value of the seismic response
ΔB	Dimension step changes in beams widths
ΔD	Dimension step changes in columns
ΔH	Dimension step changes in beams heights
A_f	Total area of formwork
A_g	Cross-section area of column
A_s	Cross-section area of longitudinal reinforcement in beams or columns
$\text{A}_{\text{s,beam}}$	Section area of longitudinal steel reinforcement in beams
A_v	Shear reinforcement area
a_{floor}	Maximum floor acceleration
b	Constant parameter describing relation between specific damage parameter and calculated damage index
B	Width of beam sections
b_w	Cross-section width of column or beam
$\text{C}^{\text{i}}_{\text{maj-inj}}$	Cost due to major injuries for the i^{th} damage state
$\text{C}^{\text{i}}_{\text{min-inj}}$	Cost due to minor injuries for the i^{th} damage state

$C^{i}_{nonstr\text{-}dam}$	Repair cost for non-structural damage for the *ith* damage state
$C^{i}_{str\text{-}dam}$	Repair cost for structural damage for the i^{th} damage state
C_0	Modification factor to relate the spectral displacement of an equivalent single-degree-of-freedom system to the roof displacement of the building (multi-degree-of-freedom system)
C_1	Modification factor to relate expected maximum inelastic displacement to displacement of a linear elastic structure system
C_2	Modification factor to represent the influences of pinched hysteresis shape, cyclic stiffness degradation and strength deterioration
C_{co}	Unit cost of concrete
C^{i}_{com}	Commercial loss cost relating to downtime of working and potential loss to the company band for the ith damage state
C^{i}_{fat}	Cost of human fatalities for the ith damage state
C_{fo}	Unit cost of formwork
C_{icon}	Contents cost
C_{idam}	Damage repair cost
C_{ifat}	Cost of human fatalities
C_{iinc}	Loss of incomes
C_{iinj}	Cost of injuries
C_{IN}	Initial construction cost
C_{iren}	Loss of rental
C_{IN}	Expected damage cost
C^{i}_{ren}	Loss of rental cost for the ith damage state
C_{so}	Unit cost of reinforcement
C_{TOT}	Total life-cycle cost
d	Distance between compression rebar to centroid of tension reinforcement

D	Design vector containing all selected design variables (Chapter 2) Dimension of column sections (Chapter 3 and 4)
$\mathrm{d_{bL}}$	Mean diameter of tension reinforcement
$\mathrm{D_g}$	Global damage index
$\mathrm{D_i}$	Structural damage index in the i^{th} storey
$\mathrm{DI^{i}_{global,target}}$	Target limit of the global damage index at the i^{th} damage state
edp	Limiting value of the engineering demand parameter (EDP)
$\mathrm{F()}$	Design objective function
$\mathrm{f_c}$	Concrete compressive strength
$\mathrm{f_c^{'}}$	Concrete compressive strength
$\mathrm{f_y}$	Steel yield strength
$\mathrm{G^i}$	Limit state function at i^{th} performance level
$\mathrm{g_i}$	Inequality design constraints
h	Depth of structural member
H	Depth of beam sections
$\mathrm{h_i}$	Storey height at i^{th} floor
im	A given earthquake intensity level
κ^{p}	Plastic curvatures in beams or columns
$\mathrm{k_e}$	Effective lateral stiffness
$\mathrm{k_i}$	Building initial stiffness
L	Length of structural member
$\mathrm{l_{pI}}$	Physical length of plastic hinge near end I

l_{pJ}	Physical length of plastic hinge near end J
L_v	Shear span at member ends
m_s	Total weight of steel reinforcement
n	Iteration step
N	Total number of storeys (Chapter 3)
	Total number of expected damage states (Chapter 4)
	Total number of independent samples utilised in Monte-Carlo method (Chapter 5)
N_{R,A_s}	Independent standard normal (Gaussian) distributed random values assigned to the uncertainty variable reinforcement section area (A_s)
$N_{R,f_c'}$	Independent standard normal (Gaussian) distributed random values assigned to the uncertainity variable concrete compressive strength (f_c')
N_{R,f_y}	Independent standard normal (Gaussian) distributed random values assigned to the uncertainty variable steel yield strength (f_y)
N_{beam}	Total number of beam elements in each storey
N_{col}	Total number of column elements in each storey
N_i	Total number of constraints
N_j	Total number of design variables
N_k	Total number of performance objectives
P	Column axial load
$P(DI_{max} > DI_{max}^i)$	Annual probability of exceedance when the maximum value of damage index (*DI*) exceeding its limit
P^i	Occurrence probability of the i^{th} damage state
$P_{np,all}^i$	Allowable limits for non-performance probability at i^{th} performance level
P_f	Failure probability
P_{np}	Non-performance probability
$PR_{drift,i}$	Performance ratio (ratio of drift demand to corresponding capacity) in i^{th} storey

$\mathrm{PR}_{\mathrm{drift,i}}^{\mathrm{critical}}$	Critical drift performance ratio in *i^th^* storey level
$\mathrm{PR}_{\mathrm{rotation,i,B}}$	Performance ratio (ratio of plastic rotation demand to corresponding capacity) in beams members in i^{th} storey
$\mathrm{PR}_{\mathrm{rotation,i,B}}^{\mathrm{critical}}$	Critical performance ratio considering rotations in beams in i^{th} storey level
$\mathrm{PR}_{\mathrm{rotation,i,C}}$	Performance ratio (ratio of plastic rotation demand to corresponding capacity) in column members in i^{th} storey
$\mathrm{PR}_{\mathrm{rotation,i,C}}^{\mathrm{critical}}$	Critical performance ratio considering rotations in column in i^{th} storey level
s	Spacing of shear reinforcement
$\mathrm{s_a}$	Spectral response acceleration
$\mathrm{Score_{TOT}}$	Overall environmental score
t	Structural lifetime
$\mathrm{T_e}$	Effective fundamental period
V	Design shear force in column
$\mathrm{V_b}$	Shear force in beam
$\mathrm{V_c}$	Total volume of concrete
$\mathrm{w_i}$	Weight factor for i^{th} storey in the calculation of global damage index
$\mathrm{W_s}$	Total weight of longitudinal reinforcement
$\mathrm{x_j}$	The j^{th} design variable considered in the design optimisation
$\mathrm{X_u}$	Uncertain variables
α	Convergence parameter
β	Reinforcement contrbution factor for beam elements (Chapter 2)
	Convergence parameter in the UDD formula which involves both local and global performance parameters (Chapter 3)
$\beta()$	Standard deviation results of the natural logarithm of structural response results, here referring to maximum floor acceleration ($\mathrm{a_{floor}}$)
β_k	Reliability index

δ_c	Calculated value of specific damage parameter
δ_i	Maximum lateral displacement at the i^{th} floor level
δ_t	Threshold value of specific damage parameter
δ_u	Ultimate value of specific damage parameter
$\theta_{max,10/50}$	Maximum inter-storey drifts under earthquake with 10% probability of exceedance in 50 years
$\theta_{max,i,B}$	Maximum beam plastic hinge rotation in the i^{th} storey
$\theta_{max,i,C}$	Maximum column plastic hinge rotation in the i^{th} storey
$\theta_{target,B}$	Target limits (capacities) of plastic rotations of beams
$\theta_{target,C}$	Target limits (capacities) of plastic rotations of columns
$\theta_{target,i,B}$	Plastic rotation capacity of beam in the i^{th} storey
$\theta_{target,i,C}$	Plastic rotation capacity of column in the i^{th} storey
θ_b	Plastic rotations of beams
θ_c	Plastic rotations of columns
θ_I	Plastic rotation at end I of an element
θ_J	Plastic rotation at end J of an element
λ	Discount rate per year in the calculation of expected life-cycle damage cost
μ	Displacement-based ductility ratio
$\mu_{\Delta u}^{N}$	Equivalent mean value for inter-storey drift Δu
μ_{d}^{N}	Equivalent mean value for allowable drift limit d
μ_{max}	Maximum ductility ratio
ρ	Tension reinforcement ratio in beam
ρ'	Compression reinforcement ratio in beam

$\rho_{b,bottom}$	Ratio of bottom longitudinal reinforcement in beams
$\rho_{b,top}$	Ratio of top longitudinal reinforcement in beams
ρ_B	Longitudinal reinforcement ratio of beams
ρ_{bal}	Reinforcement ratio producing balanced strain conditions
ρ_C	Longitudinal reinforcement ratio of columns
$\sigma_{\Delta u}^{N}$	Equivalent standard deviation for inter-storey drift Δu
σ_d^N	Equivalent standard value for allowable drift limit d
$\Phi(.)$	Standard normal probability distribution
ω_i	Reinforcements contribution factor for column elements
ω_n	n^{th} Eigen frequency of frame

List of Abbreviations

COV	Coefficient of Variation
CP	Collapse Prevention
DBE	Design-Basis Earthquake
DCM	Medium Ductility Level
DI	Damage Index
EDP	Engineering Demand Parameter
ES	Evolution Strategies
GA	Genetic Algorithm
IDA	Incremental Dynamic Analysis
IDR	Inter-Storey Drift Ratio
IM	Intensity Measure
IO	Immediate Occupancy
LCCA	Life-Cycle Cost Analysis
LS	Life Safety
MAF	Mean Annual Frequency
MCS	Monte Carlo Simulation
MC	Monte Carlo
PBD	Performance-Based Design
PBSD	Performance-Based Seismic Design
PGA	Peak Ground Acceleration
PSO	Particle Swarm Optimisation
RC	Reinforced Concrete
SSI	Soil Structure Interaction
UDD	Uniform Damage Distribution

CHAPTER 1 : Introduction

1.1. Research motivation

Reinforced concrete (RC) frames are one of the most common structural systems used worldwide for low- and medium-rise buildings, particularly in medium and high seismic regions, considering their good resistance and ductility against earthquake loads. It is well known that many RC buildings constructed prior to 1970, which are designed only to sustain gravity loads and lack of seismic detailing to provide accurate ductility under seismic loads, were generally insufficient in lateral load resistance and experienced irreparable structural damage during earthquakes. Despite developments of structural seismic designs guidelines (e.g. Eurocode 8 (CEN, 2004), Chinese code GB 50011 (National Standard of the People's Republic of China, 2010), IBC 2021 (ICC., 2020)), extreme structural seismic damage is still being observed in RC structures due to poor seismic design and construction practices, such as inefficient control of structural damage, inadequate consideration of structural nonlinearity under strong earthquakes. After the 1999 Kocaeli Earthquake, it was reported that 725 out of the observed 1215 buildings were damaged, mainly due to "weak-storey" issues (Kirac et al., 2011).. The 2008 Wenchuan earthquake has again highlighted that non-compliance with "strong-column weak-beam" principle can lead to soft-storey mechanisms and lead to substantial damage or collapse (Duan and Hueste, 2012; Lieping et al., 2008). The site survey for 2022 Luding earthquake in China reported that many observed unrepairable structural damage is mainly caused by insufficient reinforcement detailing against seismic loads and improper ductility detailing under design basis earthquake level (Qu et al., 2023). Most current seismic design guidelines utilise "force-based" principles and perform elastic analysis to determine actions for the design of structural members. These approaches can ensure overall structural capacity, but cannot directly control element and storey deformations, nor efficiently limit structural damage under earthquakes. Furthermore, these guidelines adopt equivalent static lateral forces to simulate the earthquake effects on a building, which are derived based on the dynamic behaviour of a linear elastic structure system. To account for structural nonlinearity, the forces are simply reduced by using either a behaviour factor in Eurocode 8 (CEN, 2004) or a response modification factor in IBC2006 (ICC., 2006). Previous studies have confirmed that buildings designed following the current seismic design guidelines may not supply adequate seismic performance (Magliulo et al., 2023) and fail to satisfy plastic rotation-based constraints specified in performance-based design guidelines (Mergos, 2017). Meanwhile, design solutions based on current seismic design codes do not necessarily provide adequate seismic resistances especially in inelastic range and thus do not lead to the most rational design solutions (Feng et al., 2016; Lu et al., 2016).

In general, the current guidelines mainly aim to achieve a single performance objective – life safety, under a specific hazard level (i.e. 10% probability of exceedance in 50 years). Empirical evidence shows though design requirement regarding to "life safety" can in general be achieved for the designed buildings, the extremely expensive economic losses due to structural and non-structural damage cannot be avoided in such design cases. For example, the 2016 Central Italy earthquakes resulted in the death of 297 people but the economic losses were around 11 billion euros (Perrone et al., 2019); the 2010 Canterbury earthquake (commonly known as Darfield earthquake) and its major aftershocks in 2011 caused the deaths of 185 people but an estimated NZ$40 billion costs of building recovery and reconstruction (Stevenson et al., 2014). Hence, it is essential for seismic design to provide not only structural safety but also economical solutions. However, these two objectives are conflicting, as improving structural safety generally increases the cost of construction.

To address the limitations in the current seismic design guidelines and satisfy multiple objectives, it is proposed to use structural optimisation with performance-based design (PBD). In additional to bring more resilient and economically efficient for optimum designs, the developed optimisation methodology has been identified as a feasible approach to reducing embodied carbon of structural materials in the project. This is crucial in the context of sustainable construction. In PBD, design criteria are prescribed and expressed as multiple performance objectives (i.e. immediate occupancy, life safety, collapse prevention) under various seismic hazard levels. Therefore, the proposed performance-based design optimisation method aims to provide a more direct and rational way to control structural and non-structural seismic damage during the design process. Although the seismic design of RC frames can be relatively straight forward, their optimisation is challenging considering that structural ductility and behaviour are affected by multiple design variables including section dimensions and reinforcement arrangement and amounts. Furthermore, structural seismic performance especially within the inelastic range is complex due to the yielding of steel reinforcements, and potential buckling, anchorage slipping, shear and bond failures.

Numerous optimisation techniques have been explored for seismic design of RC frames, such as optimisation based on the concept of Optimality Criteria (OC) (Bai et al., 2016; Chan and Zou, 2004), Gradient-based optimisation (Papazafeiropoulos et al., 2017), Genetic Algorithm (GA) (Seify Asghshahr, 2021; Mergos, 2018), Evolution Strategies (ES) (Lagaros and Fragiadakis, 2011), and Particle Swarm Optimisation (PSO) (Gharehbaghi, 2018). However, these existing optimisation methods tend to be computationally expensive, and their accuracy and speed can vary depending on algorithmic details. For example, OC and gradient-based optimisation demand high computational efforts to evaluate gradients of objective functions and design constraints at each optimisation step. This involves complex mathematical conversions to transform constricted design problems into unconstrainted formulations, potentially impacting accuracy and efficiency of the design optimisation. Search-based optimisations including GA and PSO generally require thousands of analysis iterations to search optimum designs, this will be computationally expensive especially for nonlinear structures under seismic loads. They also rely on predetermined search spaces for design variables in these methods, whether the optimum solution is satisfied highly depends on the choice and size of this search space. Therefore, the seismic design optimisation algorithms are still very limited for practical applications.

Most previous optimisation studies and most current design codes utilise nonlinear static (pushover) analysis with pre-defined lateral load pattern to predict structural performances under seismic loads. However, using a fixed load pattern cannot represent actual seismic effects particularly when structural inertia loads redistribute and lateral stiffnesses change after yielding. And such analyses cannot directly account for the effect of higher modes of vibrations and are likely to underestimate the seismic response especially for high-rise frames (Hajirasouliha and Pilakoutas, 2012; Moghaddam and Hajirasouliha, 2006a). Hence, such limitations in the pushover analysis can limit accuracy in performance-based optimum seismic design and may prevent engineers from using the optimisation methodology in the design process.

1.2. Scope of the research

This study is focused on developing an optimisation methodology, incorporating with concepts of “performance-based design”, for multi-storey reinforced concrete (RC) frames to minimise initial material cost of the frames, while minimising structural damage under multiple earthquake intensity levels ranging from elastic to inelastic. The concept of Uniform Damage Distribution is employed to enable a low-computational cost optimisation methodology. This approach considers multiple design variables, in terms of cross-sectional dimensions and longitudinal reinforcement ratios. The efficiency of the proposed optimisation methodology is first demonstrated through the optimisation of 3-, 5-, 10- and 15-storey RC frames, by using non-linear time history analysis under a group of generated artificial earthquakes whose spectrum compare well with the target design spectrum specified in Eurocode 8 (CEN, 2004). To assess the impact of earthquake record selections in the proposed optimisation approach, sensitivity analysis is employed. The same optimisation process is executed under two conditions: (i) a randomly selected single earthquake record, and (ii) a group of independent natural earthquakes, respectively. The economic efficiency of the provide optimisation framework is further assessed by calculating not only initial construction costs, but also expected life-cycle damage costs and total life-cycle costs for both code-based and optimum designs. Finally, this study quantifies how uncertainties related to materials, section properties and seismic ground motions affect the seismic performances of both optimally and conventionally designed RC frames.

1.3. Aims and objectives

The aim of this study is to minimise total material usages and structural damage under different seismic hazard levels in the seismic design of reinforced concrete frames, by developing a practical and computationally efficient structural optimisation methodology that integrates performance-based design concepts.

To achieve this aim, the following objectives are fulfilled:

1. Critical review of previous literatures on optimum seismic design of RC frames to identify their major achievements and to identify existing challenges and research gaps.

2. Build initial code-based seismic designs of selected 3-, 5-, 10- and 15-storey RC frames, and construct non-linear finite element models to assess their seismic performance.

3. Develop a multi-level performance-based optimisation methodology with low computational costs and suitable for practical applications.

4. Assess the efficiency of the proposed optimisation methodology by optimising the selected RC frames under a set of spectrum-compatible earthquakes with different intensity levels.

5. Study the efficiency of the optimisation framework considering the impacts of earthquake records selections.

6. Assess the cost efficiency of the obtained optimum solutions over structural effective life periods.

7. Quantify the effects of uncertainties on the seismic performances of the conventionally and optimally designed RC frames under multiple seismic hazard levels.

1.4. Methodology

The above mentioned aim and listed objectives are achieved by processing the following methodologies:

- Perform a complete review of the key steps in the structural optimisation process, including optimisation objectives, design variables, design constraints, optimisation algorithms and analysis methods for optimum seismic designs of RC frames. **(objective 1)**

- Design the selected regular multi-storey RC bare frames as typical residential buildings in high seismic regions. Begin by using Eurocode 8, the solutions also satisfy with design constraints outlined in Eurocode 2 and 8, under a seismic hazard level with PGA level of 0.4g, and with an assumption that the buildings are designed with medium ductility class (DCM). The detailing design of shear reinforcement is not involved in this work, and it assumes that the amount of the shear reinforcement is proportional to the longitudinal reinforcement quantity. **(objective 2)**

- Execute non-linear structural modelling using finite element software OpenSees. The modelling approach incorporates nonlinearities in concrete and steel reinforcement by employing specific material models with suitable stress-strain relationships. Both beam and column elements utilise a "distributed-plasticity model", enabling the simulation of nonlinear behaviour throughout the structure. **(objective 2)**

- Develop a methodology for seismic design optimisation of RC frames, based on the concept of Uniform Damage Distribution (UDD). The methodology incorporates design criteria prescribed in PBD guidelines to simultaneously control local (i.e. plastic hinge rotations of beams and columns) and global (i.e. inter-storey drift ratios) seismic responses at multiple performance levels, by iteratively modifying design variables (section sizes and longitudinal reinforcement ratios) until the responses closely approach to performance target limits. **(objective 3)**

- Implement the proposed optimisation approach for the selected RC frames, by performing non-linear time history analyses under a group of generated artificial earthquakes whose spectrum compatibility conditions have been verified to a Eurocode 8-based target design spectrum. These seismic records can be scaled to represent different seismic hazard levels. Quantify the total concrete volumes and reinforcement amounts, maximum seismic responses and structural global damage for both initial and optimum design solutions to assess the efficiency of the optimisation algorithms. **(objective 4)**

- Repeat the same optimisation framework under a randomly selected single earthquake record and a set of independent natural earthquake records, respectively. Compare results from all alterative optimisation approaches, in terms of the total material usage, maximum seismic responses at both local and global levels, and global damage index results. **(objective 5)**

- Conduct a life-cycle costs analysis to calculate the total life-cycle costs, which comprise the sum of initial construction costs and expected damage costs due to future earthquakes for the initial code-based and optimum design solutions. Utilise incremental dynamic analysis and fragility analysis in the calculation of the total life-cycle cost. **(objective 6)**

- Apply the Monte Carlo simulation method to investigate the effects of uncertainties related to concrete compressive strength, steel yielding strength and rebar area on the seismic performances of both optimum and code-based design solutions. **(objective 7)**

- Analyse the initially and optimally designed RC frames under each of selected independent natural earthquakes to investigate the impact of earthquake characteristics uncertainty on the seismic performances of both optimally and conventionally designed frames under multiple seismic hazard levels. **(objective 7)**

1.5. Thesis Layout

This thesis consists of six chapters:

1.5.1. Chapter 1

This chapter provides an overall introduction on this study, including research motivations, research aim and objectives, applied corresponding methodologies to achieve the specific objectives and thesis layout.

1.5.2. Chapter 2

A review of optimum seismic design of RC frames: state-of-the-art, challenges and future directions

Chapter 2 addresses the objective 1.

Chapter 2 provides a literature review on existing studies for structural design optimisation of RC frames under seismic loads. This chapter aims to identify research gaps in previous relevant studies. It addresses the key steps in structural size optimisation, including the design objectives, design variables and constraints involved in the optimisation formulation, the applied optimisation algorithms, and the seismic analysis methods. Based on these key aspects of design optimisation, it summarises the major achievements in previous optimum seismic designs of RC frames, identifies their research limitations, and suggests potential future directions. These may include assessing structural seismic performances at both the element and structural levels, incorporating multi-criteria performance-based seismic designs, and applying high-accuracy seismic analysis methods.

1.5.3. Chapter 3

Multi-level performance-based seismic design optimisation of RC frames

Chapter 3 addresses objective 2, 3 and 4 and part of the objective 5 and it is based on the paper: Dong, G., Hajirasouliha, I., Pilakoutas, K., Asadi, P., 2023. Multi-level performance-based seismic design optimisation of RC frames. Engineering. Structure. 293, 116591.

Chapter 3 develops a multi-level performance-based seismic design optimisation methodology based on the concept of Uniform Damage Distribution (UDD), with objectives to minimise total material usage (or initial material cost) of the RC frames while minimising structural damage. This is achieved by satisfying multiple performance objectives, including Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). The proposed optimisation framework optimises cross-section dimensions and longitudinal reinforcement ratios of RC frames in elastic and inelastic phases, respectively. The structural materials are gradually redistributed from less to heavily damaged parts until that a candidate design satisfies all pre-determined performance-based and practical design constraints, and a more uniform damage distribution is achieved under at least one seismic hazard level. The approach simultaneously controls structural seismic responses at local and global levels. The efficiency of the UDD-based optimisation methodology is demonstrated through optimum designs of 3-, 5-, 10- and 15-storey RC frames under a group of spectrum-compatible artificial earthquakes. Moreover, sensitivity analysis is conducted to assess the effect of the convergence parameter on convergence speed and optimisation accuracy. The analysis also investigates the impact of utilising a single artificial earthquake record on the efficiency of the proposed optimisation methodology.

1.5.4. Chapter 4

Life-cycle cost efficiency of RC frames optimised using multi-level performance-based methodology

Chapter 4 addresses part of the objective 5 and the objective 6.

This chapter implements life-cycle cost analysis to assess the economic efficiency of the optimum design solutions obtained in chapter 3, with an objective of minimising initial material costs. Total life-cycle costs are evaluated as the sum of initial construction costs and expected damage costs caused by future earthquakes thar may occur during the effective life period of a structure. The term “damage cost” is calculated by accounting for the costs of building repair, the costs of loss of contents and rental income, and the costs associated with human injury and death, due to structural and non-structural damage. The damage is quantified by evaluating maximum inter-storey drift and maximum floor acceleration in a probabilistic manner through fragility analysis. This approach considers the randomness of future earthquake and uncertainty in seismic demand deformations. The effect of earthquake record selections on the proposed optimisation methodology is assessed by repeating the design optimisation for a group of independent natural records, which have different dynamic characteristics from the artificial earthquakes used in chapter 3.

1.5.5. Chapter 5

Effects of uncertainties on the efficiency of multi-level performance-based seismic optimisation of RC frames

Chapter 5 addresses the objective 7.

This chapter aims to quantify the effects of uncertainties arising from material strengths, section properties and seismic ground motions on the seismic performances of both conventionally and optimally designed RC frames. The Monte Carlo simulation (MCS) method is processed to study the effects of these uncertainties. To successfully process MCS method, the first step is decided as studying the performance-sensitivity of the 5-storey frame to different uncertainty variables, including concrete strength, steel strength and cross-sectional area of reinforcement. To assess the effect of uncertainties in section and material properties on the proposed optimisation method, the global damage index is evaluated as a performance parameter for both code-based and optimum designs, under both Design Basic Earthquake and Maximum Considered Earthquake levels. This investigation considers a range of uncertainty levels, quantified by various coefficient of variations for the uncertainty variables. The effect of seismic ground motion uncertainty is investigated by analysing the optimum structures from chapter 3 under fifteen independent natural earthquake records. The seismic responses results, including maximum inter-storey drift, plastic hinge rotations and global damage index, are predicted individually under each selected natural earthquake with multiple intensity levels.

1.5.6. Chapter 6

This chapter provides conclusions of the results in the chapter 3, 4 and 5, and offers recommendations for future works on this topic.

CHAPTER 2 : A Review of Optimum Seismic Design of RC Frames: State-of-the-Art, Challenges and Future Directions

2.1. Abstract

Reinforced concrete (RC) frames are the most commonly employed structural systems globally for low and medium rise buildings. Conventional design of these systems generally relies on a "trial and error" approach with initial dimensioning and subsequent validation of the design. This makes it challenging to ascertain the potential for cost savings while maintaining structural safety. The need for efficient, safe and rational seismic design procedures has led to an increasing interest in the structural optimisation of RC frames. However, due to their complex non-linear nature, arriving at the optimum design of RC structures under earthquake excitations is challenging. Even at serviceability limit state, concrete cracking can lead to significant stiffness changes and the redistribution of inertial forces, phenomena that are normally expected after steel yielding. To address these issues, numerous combinations of section dimensions and reinforcement arrangements can be considered as design variables during the structural size optimisation process. As this requires high computational effort and high-level optimisation and analysis techniques, it is rarely attempted in RC structures. This study aims to critically review the major developments in recent seismic design optimisation studies of RC buildings. It will mainly focus on the topic "structural size optimisation", with the goal of identifying their main achievements and limitations. The first section categorises different design objectives considered in previous seismic design optimisation procedures and reviews the relevant studies along with their important conclusions. The study also addresses key steps in the structural optimisation that include design variables, design constraints, the application of optimisation methodologies and evaluation of seismic performances. Research gaps are then identified, and proposals are made

for potential future directions in this field, and the way forward for achieving safer and more efficient seismic designs in RC structures.

Keywords: Reinforced concrete frames, Seismic designs, Structural optimisation frameworks

2.2. Introduction

Substandard reinforced concrete (RC) buildings designed without considering seismic design guidelines have experienced irreparable structural damage during previous seismic events (e.g. Aycardi et al., 1992; Kunnath et al., 1995). Although new generations of structural seismic designs have emerged since the 1980s, field studies show that the seismic performances of RC frames designed according to different seismic codes are still susceptible to soft-storey failures, particularly in lower stories and under severe earthquakes (e.g. Eurocode 8-based seismic designs (Panagiotakos and Fardis, 2004), Canadian Codes-based seismic designs (NBCC) (Sadjadi et al., 2007), IBC-based seismic designs (Kim and Kim, 2009), Chinese Codes (GB50011-2010)-based seismic designs (Duan and Hueste, 2012)). This observation is also confirmed by Feng et al. (2016) and Lu et al. (2016), in which experimental and numerical studies go further to indicate that code-based structures do not exhibit uniform damage distribution especially within the inelastic range. This is because conventional seismic design generally utilises “force-based” principles, which cannot directly control element deformation and structural damage.

The primary objective in most seismic codes, such as Eurocode 8 (CEN, 2004), is to satisfy “life safety” design requirement under a design seismic hazard level (i.e. 10% probability of exceedance in 50 years). Hence, even though overall structural adequacy can be assured for that specific seismic hazard level, structural capacity is generally exhausted only in a few elements, while in most elements, it is not fully exploited. Moreover, current seismic design codes generally account for structural nonlinearities by reducing the seismic design force through a pre-determined factor (e.g. behaviour factor in Eurocode 8 (CEN, 2004)), which is decided based on judgement and empirical evidence. As a result, conventional design approach may lead to design solutions that do not satisfy rotation-based performance constraints following performance-based design criteria (Mergos, 2017). Furthermore, economic loss due to structural and non-structural damage can be unexpectedly high, even if the design solution successfully ensures life safety. This was evident in the earthquake that occurred in 2012 in Northern Italy, which resulted in 27 casualties, significant damage to public and private buildings, and an estimated overall economic loss of approximately 13 billion Euro (Meroni et al., 2017).

The increasing demand for safe and cost-efficient seismic designs for RC structures, has driven the development of structural optimisation for RC frames. The main steps required in structural optimisation are introduced in Arora (2016) and are summarised as a flowchart presented in Figure 2-1. The two main phases involved in the structural optimisation are: (i) structural analysis, which aims to evaluate structural responses under different loads (e.g. static loads, wind loads and seismic loads), and (ii) structural design, which aims to arrange structural materials, sections and elements to withstand the analysed response and satisfy code-based design constraints. As shown in the Figure 2-1, the structural optimisation process can be described as an "analysis-design" cycle that will be repeated several times (i.e. iterative steps) until an optimum design is achieved. In contrast to conventional seismic designs of RC frames, which rely on "trial-verification-modification" processes, expertise, experience, and intuition, structural optimisation provides a more systematic methodology in seismic design for solving the design problem with one or more objectives, while improving design quality. Furthermore, achieving both "cost effectiveness" and "structural safety" is considered challenging due to their conflicting nature, but structural optimisation techniques make it possible to simultaneously address these factors.

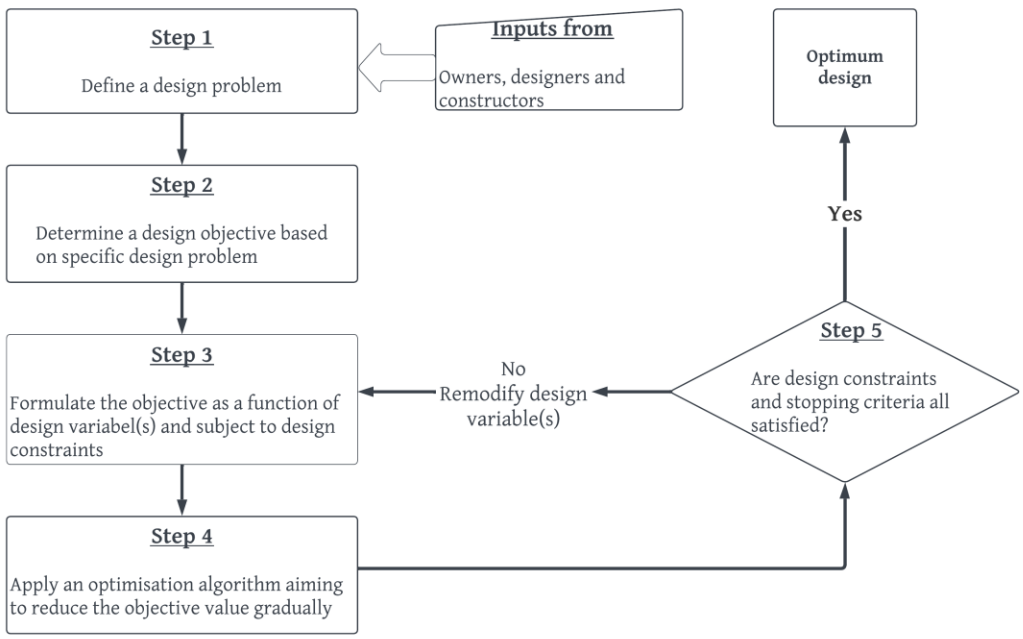

Figure 2-1: Seismic design optimisation flowchart

Structural optimisation, as a subset of mathematical design optimisation, can be further categorized based on the characteristics of the design variables into: sizing optimisation, shape optimisation and topology optimisation (Gencturk et al., 2012; Christensen and Klarbring, 2008). In sizing optimisation, the structural property of each element (e.g. dimensions of the cross sections, and longitudinal and transverse reinforcement ratios) is optimised to achieve the best solution of the optimisation objective function, while imposing geometry and boundary constraints. In this case, the location and number of structural elements are fixed. In shape optimisation, the structural boundaries are optimised to improve the performance of the structure by adopting a fixed number of boundaries for a structure, and the design variables in the objective functions are generally defined as the coordinates of the boundaries. Topology optimisation aims to find the best material layout within a selected geometrical design space through optimising the arrangement of the available materials. In such optimisation, the number, size and location of structural elements, as well as shapes of the elements and structures can be considered as design variables.

In general, both size and shape of a structure are optimised at the end of the design process to achieve a better design solution, once the layout or topology of structural members is fixed and only minor changes are allowed. Structural optimisation can vary significantly when using special devices and/or structural components (e.g. nonlinear dampers and base isolation elements), depending on the specific design objective. It should be noted that, seismic design problems are generally dealt with in the category of sizing optimisation, while the shape and topology of structural elements and the structural material properties are pre-determined and kept constant in the structural optimisation process.

Performance-based seismic design (PBSD) is a relatively new design concept adopted in several current seismic design codes (e.g. ATC 40, (1996), FEMA 227, (1992) and FEMA 356, (2000), ASCE/SEI 41-13, (2014)). Compared to the conventional "force-based" seismic design method, performance-based design can express design criteria directly relating to local (i.e. element deformations) and global (i.e. inter-storey drift) structural responses to meet specific performance requirements for buildings (e.g. immediate occupancy, collapse prevention). Hence, it offers a more rational approach to control structural and non-structural damage, satisfying different performance objectives corresponding to multiple seismic hazard levels, ranging from earthquakes with low intensity to more severe seismic events. The concept of "performance-based seismic design" ensures that structures have sufficient seismic resistance capacity with quantifiable confidence, thereby aiding structural designers in making informed decisions regarding performance criteria. The PBSD method can be further incorporated in the field of structural optimisation to produce more reliable designs that achieve specific design objectives and address multiple performance-based objectives.

However implementing performance-based structural optimisation for RC frames can be very challenging, considering that seismic behaviour of RC structures especially within non-linear range can be significantly affected by different design variables including reinforcement yielding and buckling, as well as deterioration due to shear and bond failures. Understanding the impact of different design variables on the seismic response is crucial, as it assists in determining how their adjustments contribute effectively to optimum solution in a performance-based optimisation framework. Previous studies have demonstrated that although an increase in flexural reinforcement ratio will increase the initial construction costs, it does not necessarily enhance structural safety or reduce seismic performance (Asadi and Hajirasouliha, 2020; Foraboschi, 2019).

Moreover, most optimisation methodologies employed in current studies are computationally expensive due to a large number of iterations and non-linear response analyses required during the optimisation process (Mohammadi et al., 2019; Nabid et al., 2019). To reduce the computational effort, current structural optimisation of RC frames generally employs nonlinear static analysis or even linear analysis, which limits the accuracy of the predictions for structural seismic responses especially for structures with high nonlinearity and/or for tall buildings where the effects of higher modes are more dominant. Previous studies showed that the optimum solution in this case may not be very reliable when the structure is subjected to strong earthquake events (Moghaddam and Hajirasouliha, 2006a).

In the literature, there is still a lack of a critical and comprehensive review on seismic design optimisation of RC frames, including in-depth and detailed discussions of the key steps in the structural size optimisation process.

This chapter aims to provide a critical review on recent development of structural size optimisation of RC structures in seismic regions, to better understand the current achievements in this field, identify existing challenges and major research gaps, and discuss the future developments of this topic. The chapter is organised following the structural optimisation steps in the Figure 2-1. Details of the reviewed concepts are presented in Figure 2-2. Finally, the research gaps are identified, and suggestions are provided for future research directions in the field.

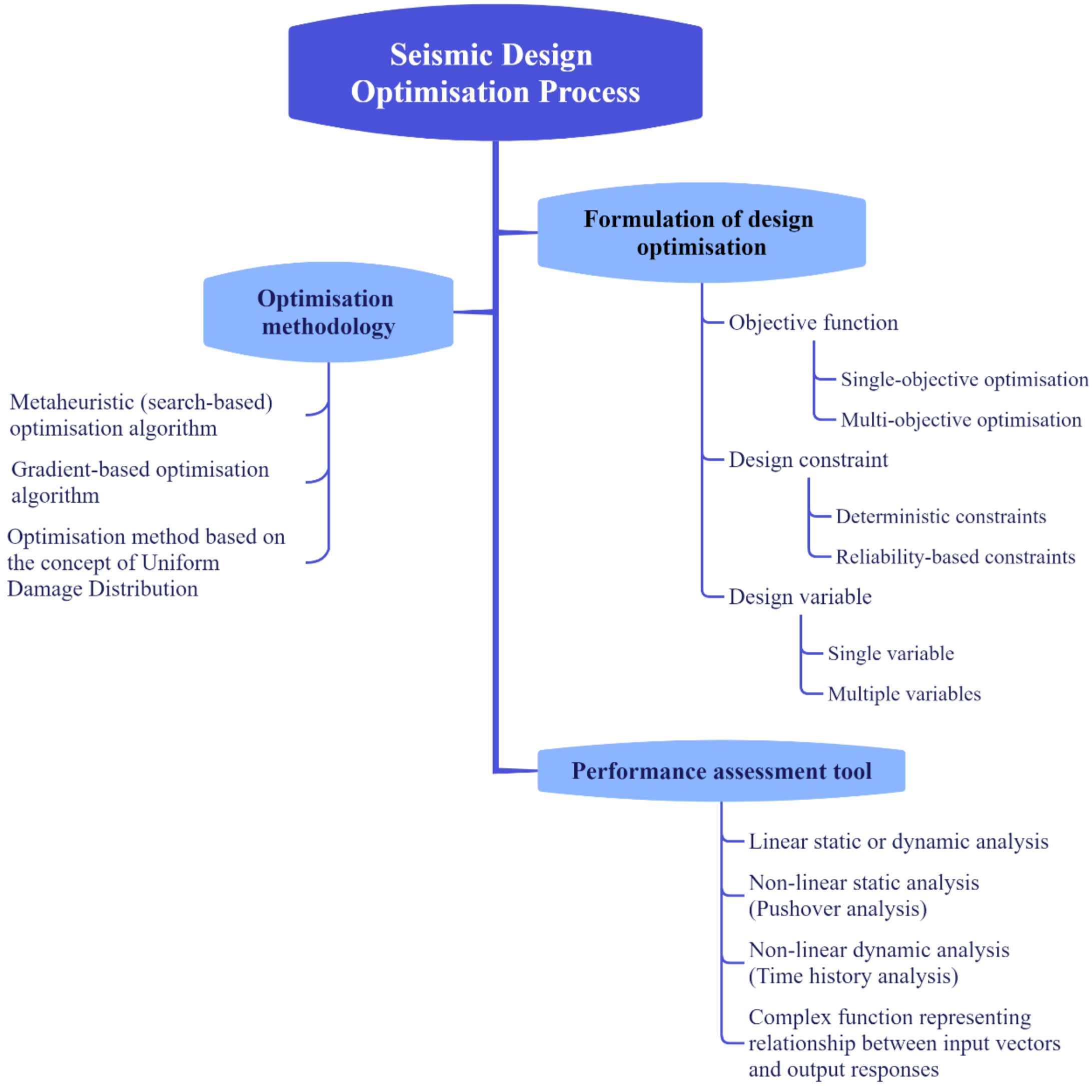

Figure 2-2: Concepts of seismic design optimisation process discussed in this study

2.3. Objective of seismic design optimisation

In structural optimisation, an optimisation formula is needed to describe the design problem, define the design objectives, and list the design constraints. The design objectives can be single or multiple, with the latter being more complex. Figure 2-3 shows the evaluation of the reviewed studies for such optimisations. It is evident that this topic has attracted increasing attention from researchers, but there is still need for the development of multi-objective optimisation procedures for seismic design of RC frame.

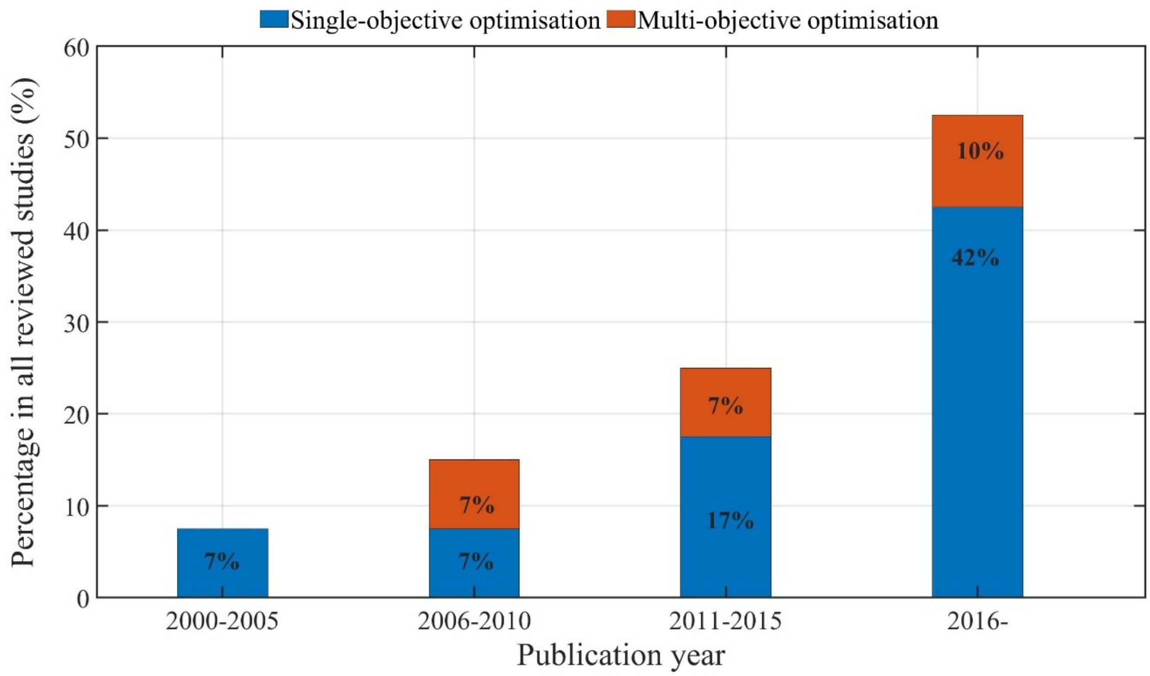

Figure 2-3: Percentage of objective function types in the past studies reviewed in this study

2.3.1. Single-objective optimisation

A general Optimisation formula in a single-objective design optimisation problem can be listed as:

$$\text{Min} \quad F = F(x_j) \;\; (j = 1, 2, \ldots N_j) \tag{2. 1}$$

subject to: $g_i(x) \geq 0 \quad i = 1,2 \ldots N_i$

$x_j \in D \quad j = 1,2 \ldots N_j$

where $F(x_j)$ represents the design objective function, and g_i denotes the inequality design constraints (more details will be provided in the next section). N_i is the total number of constraints required, x_j represents the j^{th} design variable considered in the design optimisation (for more details see section below), and N_j is the total number of variables, with *D* representing design vector containing all the selected design variables.

As mentioned above, the majority of structural optimisation studies fall into the category of "single-objective optimisation". These studies can be further classified into two categories:

(1) Minimum structural damage or seismic performance improvement: the objective of the design optimisation is to minimise structural damage at global or local level and improve structural seismic performance(s) under specific hazard level(s) in a direct or indirect manner.

(2) Minimum economic cost: the objective of the design optimisation is to minimise cost of a RC building at initial construction stage or during its effective operational period in the circumstance of earthquakes.

2.3.1.1. Minimum structural damage optimisation

The minimum structural damage optimisation can be achieved in a direct approach, with an aim to reduce the local concentration of seismic demand and obtain a more uniform distribution of damage, quantified by damage indices or other specific seismic performances such as inter-storey drift ratio.

In this case, the objective function $F(x_j)$ is generally formulated with reference to the structural seismic response as follows:

$$\text{Min} \quad F(x_j) = \frac{IDR_i}{IDR_0} \tag{2. 2}$$

where IDR_0 and IDR_i are the structural maximum response (here inter-storey drift ratio) for initial design and for design in i^{th} iterative step, respectively. The parameter IDR can also be written using alterative performance parameters, such as plastic hinge rotations, storey ductility demand, local or global damage index, and coefficient of variation (COV) of the seismic response. The structural response at the initial design stage (IDR_0) can be substituted with other variables, such as targeted response at a specific performance level. It is expected that, through a few continuous iterative steps, the optimisation process will lead to the stability of the objective function, while resulting in less structural damage and a near-uniform drift distribution across the structure.

Optimum lateral load pattern

Most seismic design guidelines that adopt force-based designs use equivalent static seismic lateral forces to account for seismic excitations. The distribution of the lateral forces along the height is derived based on elastic vibration response, and directly affect the distribution of deformation demands. However, the currently adopted lateral load patterns are not consistent with the real inertial load distribution, especially within inelastic range (Moghaddam and Hajirasouliha, 2006b). A simple and direct approach to reduce structural damage is to use an optimum lateral load pattern during the seismic design process. In this case, the optimisation only modifies the initial first mode-based lateral load pattern, but the design procedure remains unchanged.

A study by Varughese et al. (2014) aimed to minimise structural damage particularly at the top storeys of tall RC frames. It was found that RC frames designed using optimum lateral load patterns (in this case Chao load distribution) sustained more uniform damage and inter-storey drift distributions. This is because the Chao load distribution considers the contribution of higher modes that are important in high-rise buildings. To increase structural resistance capacity at collapse state under earthquakes, Li et al. (2019) presented an optimisation method for low-to-medium rise RC buildings. The shear strength in each storey was iteratively redistributed by redesigning reinforcement ratios until the storey ductility demands in all storeys were almost uniformly distributed. The optimum results were used to determine the optimum lateral load pattern. The results demonstrated that frames designed based on optimum lateral force patterns exhibited less collapse possibilities compared to code-based designs, and satisfied story limits under multiple seismic intensity levels.

More uniform damage distribution

Some seismic design codes, such as Eurocode 8 (CEN, 2004), determine the resistance of the members under the design forces derived from a linear elastic system and accounting structural nonlinearity using a behaviour factor based on empirical evidence and experimental results. And these codes typically check displacement demands at the end of the design process mainly under frequent earthquake events. It thus cannot explicitly assess the impacts of severe earthquakes on structural resilience. Meanwhile, structural damage is primarily managed by limiting elements' strengths under major earthquakes in the modern design codes, however, it cannot directly address structural and non-structural damage. To address this issue, PBD methods effectively limit structural damage by controlling structural performances under multiple seismic hazard levels, and hence can be used to attain a more uniform structural damage and better use of material to withstand seismic loads.

As an example, performance-based seismic design optimisation by Hajirasouliha et al. (2012) aimed to minimise structural damage by observing inter-storey drifts of multi-storey RC frames at Life Safety (LS) performance level. The total material usage was kept constant in the optimisation to ensures that costs immediately after construction aren't significantly affected by the optimisation framework. The results indicated that, compared to RC frames conventionally designed using IBC-2009, optimum solutions significantly reduced global damage by up to 30% and achieved near uniform inter-storey drift distribution. Similar conclusions were arrived by Bai et al. (2016), where structural damage of RC moment-resisting frames was minimised by redistributing the chosen design variable (area of flexure reinforcement) from components with low damage to the other elements experiencing more damage. The study also found that the optimum structure exhibited lower plastic hinge rotations under earthquake excitations.

Plastic hinge rotation demands were also used in (Bai et al., 2020) to redistribute material and reduce damage. It was shown that optimum design solutions can reduce maximum inter-storey drift ratio up to 35%, while also achieve more uniform deformation distributions under different seismic hazard levels, with small material costs increase (5%-10%). Similarly, Hashmi et al. (2018) presented a performance-based design optimisation that aimed to achieve more efficient use of structural members and more uniform distribution of inter-storey drifts at serviceability limit state, for both regular frames (i.e. Bare RC frames) and irregular frames (i.e. Open Ground Storey RC frames). In performance-based optimum design of irregular RC frames proposed by

Hashmi et al. (2022), the damage minimisation was achieved by controlling damage index at both storey and global levels. The optimum design had more uniform distribution of damage throughout the structure, and less global damage (up to 30%) compared to code-based design solutions.

Modified fundamental period

Reducing the fundamental (first-mode) period of an elastic structural has been used as an indirect approach to minimise structural damage (Arroyo et al., 2018; Arroyo and Gutiérrez, 2017). The objective function $F(x_j)$ here is expressed as:

$$\text{Max} \quad F(x_j) = \omega_n(x_j) \tag{2. 3}$$

where ω_n represents n^{th} Eigen frequency of selected regular and irregular RC frames.

These studies enhanced the elastic performance and overstrength of structures and in turn delayed the initiation of structural inelastic seismic responses. Consequently, they provided better designs with more uniform distributions of drifts and less susceptibility to collapse.

2.3.1.2. Minimum cost optimisation

Material optimisation

In most design optimisation studies, to save computational cost, the objective of minimising initial construction cost is simplified to minimising the of total material use in the structure in terms of concrete volumes and reinforcement weights. In this case, the objective function $F(x_j)$ can be written as:

$$F(x_j) = V_c \cdot C_{co} + m_s \cdot C_{so} = \sum_{i}^{N_i} b_i h_i L_i C_{co} + \sum_{i}^{N_i} A_{S,i} L_i C_{so} \tag{2. 4}$$

where V_c is total volume of concrete; m_s is total weight of steel reinforcement; and C_{co} and C_{so} are the unit cost of concrete and reinforcement, respectively; b_i, h_i and L_i also represent width, depth and length of structural member i, respectively, while $A_{S,i}$ is cross-section area of reinforcement in member *i*.

Ganzerli et al. (2000) proposed optimum seismic design incorporating PBD criteria (limiting plastic rotations at the ends of beam and column elements) for a simple 2D portal RC frames to minimise structural costs that were assumed to be proportional to the amounts of concrete and reinforcement. In more recent optimisation studies of multi-storey RC structures (Razmara Shooli et al., 2019; Liu et al., 2010; Fragiadakis and Papadrakakis, 2008; Zou and Chan, 2005a, 2005b; Chan and Zou, 2004), the design objective of "minimum structural material cost" was achieved by minimising total concrete volume and reinforcing steel weight. Multiple design variables in terms of the section dimensions and steel reinforcement were considered in the objective function, while subjecting to constraints to limit structural seismic responses (e.g. inter-storey drift) and ensure structural safety under selected seismic hazard levels. Consequently, the optimum solutions provided less total direct cost and achieved an improved control on maximum seismic response values.

Seify Asghshahr, (2021) developed a reliability-based optimisation framework to achieve a trade-off between minimum initial material cost and minimum reliability index for RC frames in seismic events. The performance-based design optimisation studied by Hajirasouliha et al. (2012) and Hashmi et al. (2022), also aimed to minimise total weight of longitudinal reinforcement in RC frames, while controlling inter-storey drift in each storey for the selected performance level. The dimensions of structural members were initially determined to sustain gravity loads and satisfy design requirements at serviceability limit state. It was shown that the proposed optimisation approach could reduce the amount of reinforcement steel by up to 33% and simultaneously satisfy multiple performance objectives in terms of Life Safety (LS) and Collapse Prevention (CP). To reduce computational cost, Zhang and Tian, (2019) developed a simplified version of design optimisation to minimise initial construction cost of a RC frame, by reducing the number of design variables into: overall system stiffness (factor) and overall system strength (factor). This led to a 21% reduction in overall initial cost compared to initial strength-based design, while both drift and plastic rotation-based constraints were satisfied under three seismic hazard levels (i.e. occasional, rare and very rare).

Reducing construction costs using more parameters

It is worth mentioning that, apart from total concrete volume and reinforcement weight, values assigned to section dimensions of RC frames also affect the total amount of formwork used during the construction and its relevant costs. In some optimisation studies, the construction cost involved in the design objective function was encompassed by three cost components: concrete, reinforcement steel and formwork (Gholizadeh et al., 2023; Mergos, 2020; Gholizadeh and Aligholizadeh, 2019; Mergos, 2018a, 2017; Akin and Saka, 2015; Gharehbaghi and Khatibinia, 2015; Akin and Saka, 2012). The general objective function $F(x_j)$ utilised in these studies in general were written as:

$$F(x_j) = V_c \cdot C_{co} + m_s \cdot C_{so} + A_f \cdot C_{fo} \tag{2. 5}$$

where V_c is the volume of concrete; m_s is the mass of steel reinforcement; A_f is total area of formwork; and C_{co}, C_{so} and C_{fo} are the unit costs of concrete, reinforcement and formwork, respectively. Study by Akin and Saka, (2015) has highlighted that the initial construction cost of optimum design solution is highly dependent on the unit prices of concrete, steel and formwork.

This review reveals that almost half of the previous studies simplified the optimisation from an objective “minimum economic cost” into a “minimum structural material usage”. However, using structural weight to represent initial cost is questionable as minimising total material usage in RC buildings doesn't necessarily lead to the minimum cost, particularly if the impacts of labour and fabrication costs are considered in the calculation of initial construction cost. In a study by Li et al. (2010), initial cost of RC frame-shear-wall structures was minimised by considering costs of materials (i.e. concrete and steel), fabrications, labours and formwork. To achieve a practical optimum deign that satisfies all strength and stiffness constraints in design codes, the optimisation procedure was divided into two parts: "strength optimum design" and "stiffness optimum design", while two databases were constructed for beam and column sections, respectively. The results indicated that the proposed optimisation not only minimised the total cost but also provided a design solution that could be directly adopted as practical design by engineers. A similar design optimisation was also proposed by Esfandiari et al. (2018), who established a "minimum cost" objective function with reference to costs of materials, labour and placement of concrete and reinforcement, while satisfying not only design code requirements, but also constructional, architectural and reinforcement detailing constraints. The final optimum design solution in this case can be practically applied without any further processing.

Indirectly reducing economic cost

Another indirect way to minimise structural initial construction cost is provided by Lavan and Wilkinson (2017), in which the objective of the design optimisation is to minimise total flexural moment capacity of all seismic beam and column members for 3D irregular RC structures, while satisfying constraints assigned on inter-storey drift and ductility. This is because, with an assumption of unchanged element dimensions, total volume of steel is the main component that affects the cost of the RC frame, which is directly related to element flexural strength.

Minimising total life-cycle cost

In an optimum performance-based design framework provided by Lagaros and Fragiadakis, (2011), a single objective was considered to minimise total life-cycle cost that is expressed as the sum of the initial cost and the expected limit-state cost over the life span of 3D regular and irregular RC structures. Similarly, Razavi and Gholizadeh, (2021) proposed a single-objective optimisation of RC frames with two different objective functions in terms of minimum initial cost and minimum total life-cycle cost (the sum of initial cost and expected life-cycle cost), respectively. The results indicated that optimisation considering the total cost provided a more efficient design solution in terms of economy and seismic collapse safety of the structure than minimising initial cost design.

2.3.2. Multi-objective optimisation

Several seismic design problems involve managing multiple conflicting building requirements throughout the design process. A general optimisation formula in a multi-objective design optimisation problem can be expressed as:

$$\text{Min} \quad [F_1(x_j),\ F_2(x_j)\,] \tag{2. 6}$$

subject to: $g_i(x) \geq 0 \quad i = 1,2 \dots N_i$

$x_j \in D \quad j = 1,2 \dots N_j$

where $F_1(x_j)$, $F_2(x_j)$ are multiple design objectives relating to design problems in the optimisation study. g_i denotes design constraints, N_i is the total number of constraints required, and N_j is the total number of the selected design variables (x_j).

It should be noticed that, in general, there is not any unique solution that achieves optimum answers for all specific design objectives simultaneously. Thus, a set of optimum solutions are obtained as trade-off answers among all design criteria and are generally presented as a Pareto front. The Pareto curve is a useful tool to display all multi-objective optimum solutions and to help engineers choose a compromise solution that achieves a balance between conflicting objectives and satisfies practical design purposes and restraints.

2.3.2.1. Reducing structural damage and saving costs

"Minimum cost" and "damage control" can be considered as two conflicting design objectives in seismic design of RC frames. In general, reducing the total amount of material usage can minimise the initial construction cost. However, blindly reducing material usages may compromise the capacity of certain structural elements, thereby increasing structural seismic response (e.g. floor acceleration, inter-storey drift) and possibilities of structural failure during strong seismic events. To strike a balance between these conflicting objectives, Lagaros and Papadrakakis (2007) expressed a multi-objective function in the optimisation framework for a 3D RC frame as:

$$\text{Min} \quad [F_1(x_j) = C_{IN}(x_j),\ F_2(x_j) = \theta_{max,10/50}(x_j)\,] \tag{2. 7}$$

where C_{IN} is initial construction cost, which consists of costs of materials, labours and non-structural components; and $\theta_{max,10/50}$ is maximum inter-storey drifts under earthquakes with 10% probability of exceedance in 50 years.

As mentioned above, for multi-objective optimisation problems, a unique solution that simultaneously achieves multiple predetermined objectives generally doesn't exist. In a study by Lagaros and Papadrakakis (2007), a set of solutions that are acceptable answers for the optimisation problem were presented as "optimum designs", expressed as points on a Pareto front curve. Here, the x and y axis represent two specific design objectives, namely initial construction cost and maximum inter-storey drift. The curve indicates the locus of all "optimum designs" across different values of the objectives. The results on limit-state fragility curves showed that, using the same initial cost, optimum designs obtained through the Eurocode-based design method were more vulnerable to future earthquakes, compared to design solutions obtained following PBD procedures (Lagaros and Papadrakakis, 2007).

Gharehbaghi, (2018) proposed a uniform damage-based optimisation approach for the seismic design of RC frames that led to optimum solutions with lower construction costs and structural damage. The modified Park-Ang damage index was adopted as the performance parameter to observe structural damage at components as well as storey and global levels. It was found that code-based design solutions were slightly more expensive (up to 4%) and suffered more damage (30% on average) especially under severe earthquakes. Asadi and Hajirasouliha, (2020) introduced a practical performance-based optimisation methodology based on the concept of Uniform Damage Distribution (UDD) for RC frames, aiming to minimise both structural and non-structural damage, quantified in terms of inter-storey drift, and total life-cycle cost. The results from incremental dynamic analysis (IDA) indicated that, compared to frames that were initially designed based on seismic design codes ASCE 07-16 and ACI 318-14, the optimum design solutions significantly reduced total life-cycle cost (up to 45%) and exhibited up to 50% less maximum inter-storey drift ratios at LS performance level. It also highlighted the fact that the optimisation with an aim to minimise initial cost does not necessarily lead to the optimum solution when the life-cycle cost is considered in the objective function. In another relevant studies, Möller et al. (2009) and Möller et al. (2015) proposed optimisation frameworks for RC frames with an aim to minimise both life-cycle cost and structural failure probability under earthquake excitations. The failure probabilities of the optimum solutions were limited by applying reliability constraints at each selected performance level. Apart from initial construction cost and damage repair cost involved in the life-cycle cost, Möller et al. (2015) also introduced social cost as additional component in the cost objective function, which was associated with costs of human injuries and fatalities, as well as economic loss after earthquakes.

2.3.2.2. Minimising initial construction cost and total life-cycle cost

During seismic design optimisation , the term "economic cost" can extend to a boarder definition that consists of: (i) the initial costs which comprise material and fabrication costs required during the construction, and (ii) the expected damage costs due to possible structural and non-structural damage under random seismic events occurring over time. The total life-cycle cost, defined as the cost required to maintain the structural conditions over the structural operational lifetime, is calculated as sum of initial construction cost and expected damage cost:

$$C_{TOT}(t, x_j) = C_{IN}(x_j) + C_{LS}(t, x_j) \tag{2. 8}$$

where C_{IN} is the initial cost and C_{LS} is the expected damage cost under different levels of earthquake intensity, x_j relates to selected j^{th} design variables, and t is the pre-decided structural lifetime. The expected damage cost is then calculated as:

$$C_{LS} = C^{i}_{dam} + C^{i}_{con} + C^{i}_{ren} + C^{i}_{inc} + C^{i}_{inj} + C^{i}_{fat} \quad (2.9)$$

where C_{LS} is the sum of damage repair (C^{i}_{dam}), contents cost (C^{i}_{con}) due to structural damages which are generally quantified by maximum inter-story drift and floor acceleration, loss of rental (C^{i}_{ren}), loss of incomes (C^{i}_{inc}), cost of injuries (C^{i}_{inj}) and cost of human fatalities (C^{i}_{fat}).

During an optimisation process, the parameters "initial construction cost (C_{IN})" and "expected damage cost (C_{LS})" in the calculation of life-cycle cost conflict with each other. Considering that C_{LS} is technically determined based on both structural and non-structural damage, a reduction in structural damage can result in lower overall expected damage cost. However, this reduction is generally achieved by utilising additional materials, which in turn increases the initial construction cost (C_{IN}). Therefore, Zou et al. (2007) developed a multi-objective function in seismic design optimisation of RC frames as:

$$\text{Min} \quad [F_1(x_j) = C_{IN}(x_j),\ F_2(x_j) = C_{LS}(t, x_j)\ ] \quad (2.10)$$

In such a design optimisation framework, the abovementioned objectives can be used to minimise the total life-cycle cost ($C_{TOT}(t, x_j)$). Both section dimensions of RC members and reinforcements quantities were considered as design variables to minimise concrete cost and steel reinforcement cost, respectively. The proposed multi-objective optimisation function was solved by first transferring it into a single-objective function through ε-constraint method. Consequently, a Pareto optimal set that contained a set of no-dominated solutions of the optimisation problem was provided. Optimisers directly selected the best compromise solution which achieved a balance between the initial cost and expected damage cost.

Similar multi-objective functions were utilised in studies by Khatibinia et al. (2013) and Yazdani et al. (2017), where annual probabilities of non-performance (failure) were also limited by subjecting reliability-based constraints to the objective function. In study by Mitropoulou et al. (2011), a multi-objective performance-based seismic optimisation was processed to simultaneously minimise $C_{IN}(x_j)$ and $C_{TOT}(t, x_j)$ in 3D regular and irregular RC buildings. Compared to single-objective design optimisation where $C_{IN}(x_j)$ was an objective to be minimised, the solutions of the multi-objective optimisation problem required more material usage but led to less life-cycle cost. This reduced structural vulnerabilities to future earthquakes especially when the initial cost was considered as dominant factor in choosing optimum solutions. The results also highlighted that neglecting the effects of uncertainties in material properties, section dimensions and record-incident angle can significantly underestimate the values of seismic damage indices and total life-cycle costs (up to 30%).

2.3.2.3. Minimising total life-cycle cost and overall environmental impacts

Additional optimisation objectives can include minimising environmental impacts caused by material and energy consumption, greenhouse gas emissions and carbon dioxide (CO_2) emissions, during both construction and operation periods. In a study by Nouri et al. (2020), an optimisation approach was developed for sustainable seismic design of RC frames, focusing on both cost saving and environmental impacts for the entire structural life- cycle period. Optimum designs were achieved by considering three distinct objective functions: (i) minimising the sum of initial construction cost (C_{IN}) and expected damage cost (C_{LS}), (ii) minimising the total life-cycle cost (C_{TOT}) and overall environmental score ($Score_{TOT}$) (quantifying the environmental impact), and (iii) minimising the sum of life-cycle cost (C_{TOT}) and environmental scores at initial construction and operational stages. The results showed that, compared to code-based designs: (i) optimum design obtained considering the first objective reduced total life-cycle cost by up to 9%, but had a slight increase in initial cost; (ii) when parameter "environmental impact" was considered in the objective function, seismic responses of the optimum designs slightly violated target limits but within an acceptable range, and while it led to a slightly higher life-cycle costs (up to 5.5%), it considerably reduced the environmental score (up to 22%). Optimisation studies on RC frames developed by Mergos (2018b) also confirmed that seismic design increased the CO_2 emissions of RC frames. When using high ratios of unit environmental impacts of concrete and steel, minimum environmental impact designs were closely related to optimum cost designs.

Figure 2-4 summarises design objectives selected in the reviewed studies on optimum designs of RC structures, categorising them into single-objective and multi-objective optimisation. The figure also provides details on the procedures used to achieve each design objective in each case. Figure 2-5 shows the percentage of the relevant optimisation studies reviewed in this chapter for each optimisation objective. A comprehensive overview of research developments in design optimisation problems of RC frames, including the names of researchers, publication years, targeted structures, applied optimisation methodologies, implemented seismic analysis methods, and modified design variables, is provided in the Appendix A.

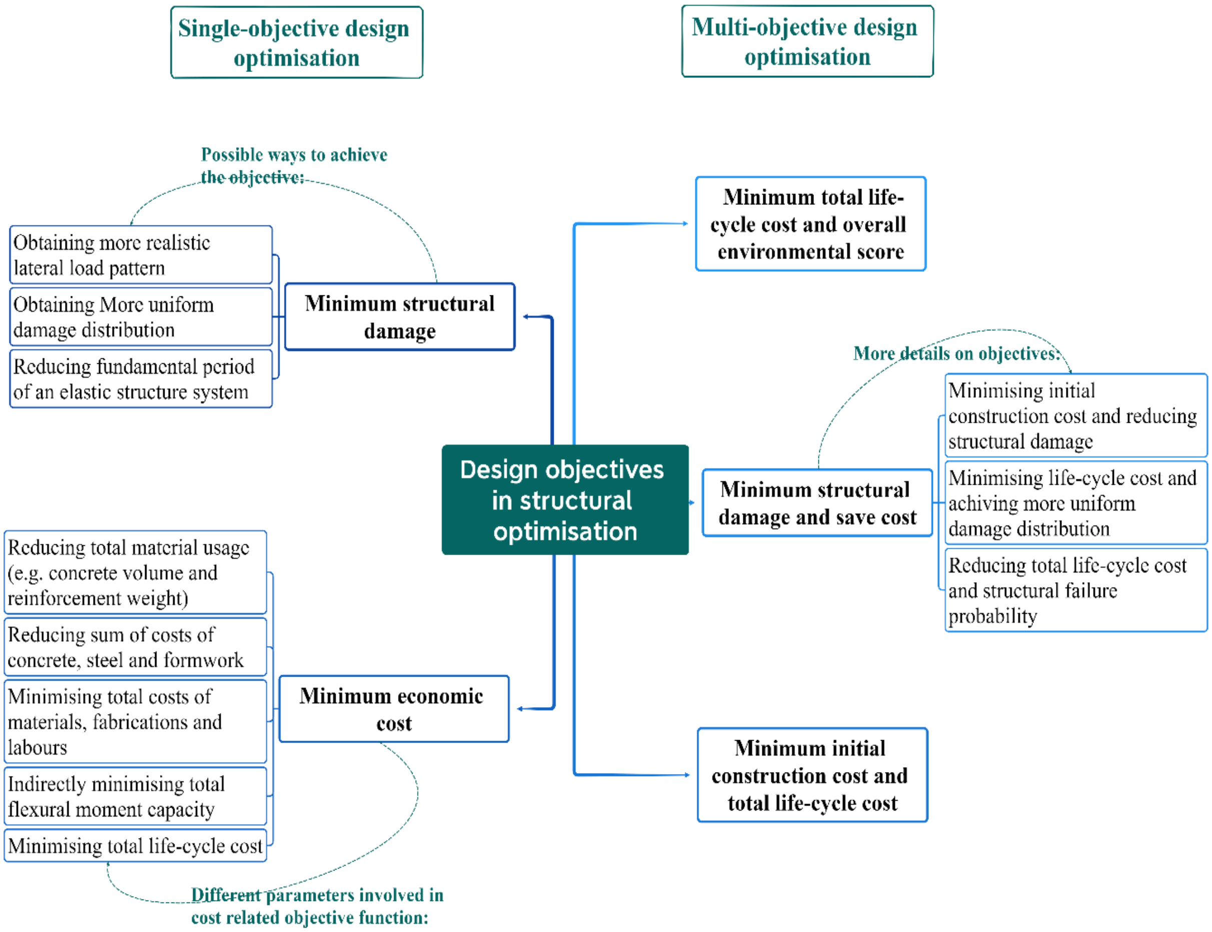

Figure 2-4: Details on different design objective(s) selected in the optimisation frameworks reviewed in this study

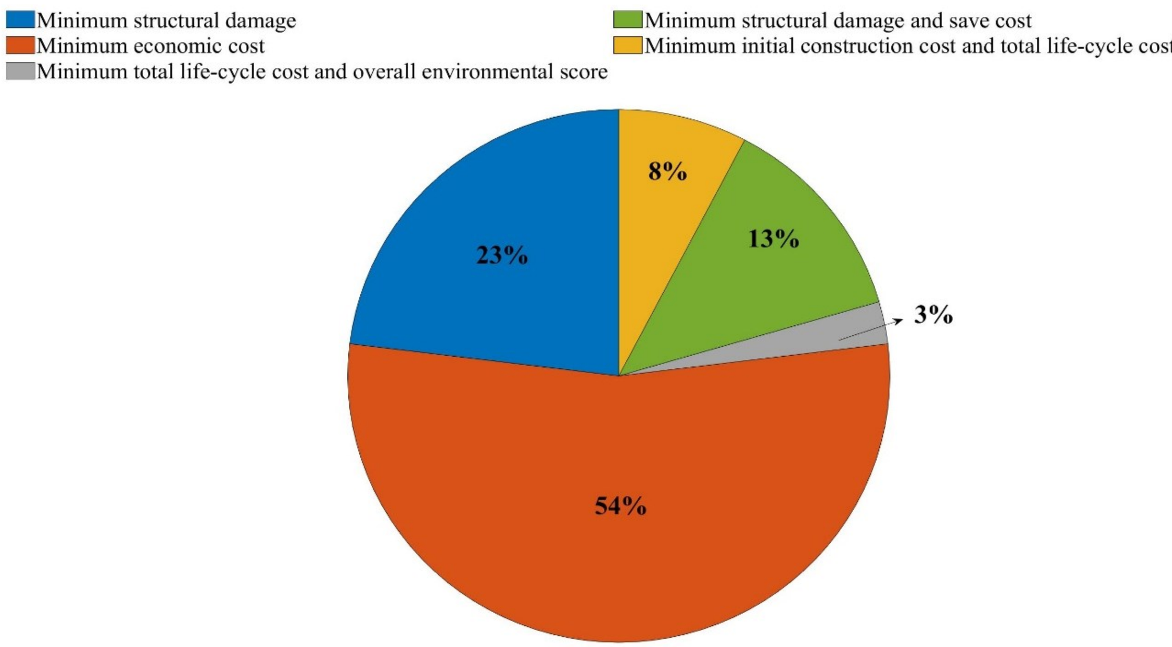

Figure 2-5: Proportion of different design objectives in all reviewed studies

2.4. Optimisation formulations: design variables and constraints

As shown in the above-mentioned optimisation formulations for both single objective and multiple objectives design problems, the three fundamental components involved in the formulations are objective function(s) ($F_1(x_j)$,), "j" dimensional design variables (x_j) and design constraints.

2.4.1. Design Variables

Design variables are the parameters that are modified in the optimisation process to achieve the optimisation objective. In terms of size optimisation in the field of seismic design, the building geometry is generally pre-determined by the architects, and while structural materials are also pre-determined by engineers. Thus, in most previous seismic design optimisation of RC buildings, cross-section dimensions and amount of flexural reinforcement of RC elements are considered as design variables since their values are directly related to the total weight of the structures and their optimisation can reduce structural damage during seismic events. In accordance with the number of design variables modified in the optimisation formulation, the design variables can be divided into two categories: single and multiple design variables. The values of the variables are generally classified as discrete or continuous.

2.4.1.1. Single design variable

Longitudinal steel reinforcement ratio is widely used as a single design variable in previous optimum designs, especially in inelastic designs, since it can be considered as the key parameter to control structural inelastic responses and provide structural ductility capacity. Bai et al. (2016) highlighted that the longitudinal reinforcements of elements in a specific story i can be also influenced by the reinforcement designs in adjacent stories:

$$[(A_{scol})_i^j]_k = \omega_i^1 * [(A_{scol,1})_i^j]_k + \omega_i^2 * [(A_{scol,2})_i^j]_k + \omega_i^3 * [(A_{scol,3})_i^j]_k \tag{2. 11}$$

$$[(A_{sbeam})_i^j]_k = (1-\beta) * [(A_{sbeam,1})_i^j]_k + \beta * [(A_{sbeam,2})_i^j]_k \tag{2. 12}$$

where $\omega_i^1, \omega_i^2, \omega_i^3$ are reinforcements contribution factors; subscripts "1" "2" and "3" denote longitudinal reinforcement of column in adjacent lower storey, current storey and adjacent upper storey, respectively; β is the reinforcement contribution factor for beam elements, and is normally assumed as a constant value (i.e. β = 0.5), $[(A_{sbeam})_i^j]_k$ is new modified steel reinforcement areas for the j^{th} beam element in the i^{th} storey and at the k$^{\text{th}}$ iteration; $A_{sbeam,1}$ and $A_{sbeam,2}$ represent longitudinal reinforcement in beams in the i^{th} storey and adjacent upper storey, respectively.

In the case where quantity of longitudinal steel reinforcement is considered as the only section design variable, sectional dimension is another important parameter affecting the structural performance of the RC buildings. The dimension variable is normally determined at the initial stage of seismic design in accordance with gravity loads information and serviceability requirements in design codes (Bai et al., 2020; Lavan and Wilkinson, 2017; Hajirasouliha et al., 2012). Previous optimisation studies generally kept the section dimensions constant during the entire design optimisation procedure, and these were only enlarged if the modified reinforcement exceeded the limiting values specified in design guidelines or due to practical limitations. However, these design variables are not independent, as structural ductility and deformability under seismic excitations are affected by both section size and reinforcement amount. In such optimisation approaches, it is also assumed that adequate transverse reinforcement is available to prevent shear failure and buckling of the longitudinal reinforcement, and that their amount is approximately proportional to the amount of longitudinal reinforcement.

In some optimisation studies, sectional dimension of column and beam elements is used as the only independent design variable. This single design variable is generally utilised in optimisation problems dealing with the elastic behaviours under minor earthquakes or for stiffness optimisation. This is because concrete sectional size dominates structural lateral stiffness and deformations within the elastic range. Thus detailing on the steel reinforcement is performed after optimising section dimensions and following conventional design procedures (Arroyo et al., 2018; Arroyo and Gutiérrez, 2017; Li et al., 2010).

2.4.1.2. Multiple design variables

For "minimum structural cost", multiple design variables are generally used in the objective function that accounts for the costs of both concrete and steel. Similarly, if the optimum design aims to improve structural safety under multiple seismic hazard levels, multiple design variables are needed, as concrete and steel influence stiffness and strength, respectively.

In structural optimisation, one of the ways to utilise multiple design variables is by using databases that contain pre-determined beam and column elements with various cross-sectional sizes and amounts of longitudinal and transverse reinforcements. The boundaries of each database are decided by subjecting design constraints on the objective function. A search-based optimisation method is then used to find optimum answers of design variables from pre-determined values in the database. Therefore, the proper selection of search domain (database) affects the accuracy of optimum design solutions. To provide a more suitable search domain for seismic design optimisation and save computational costs, Razmara Shooli et al. (2019) proposed combining non-linear static analysis and non-linear dynamic analysis to obtain an optimum search domain first. The optimum design variables were then determined from this optimum domain. Furthermore, to achieve an accurate and practical design solution, Mergos, (2018a, 2017) divided the vector of design variables, namely section dimensions, longitudinal and transverse reinforcement steels, into three independent sub-vectors. Each sub-vector was optimised independently by searching answers from the pre-determined database. Consequently, two structural members designed with the same dimensions could have different reinforcement details. Mergos (2020) categorised the selected design variables into primary and secondary variables. Cross-section dimensions and longitudinal reinforcement amounts were considered as primary variables and were selected from a pre-determined search space. Transverse reinforcements, as a secondary variable, was decided to fulfil design requirements with respect to performance, serviceability and construction practices after the optimiser derived the primary variables in each iteration.

Some optimisations dealing with multiple design variables divide the entire optimisation process into: (i) elastic design optimisation, in which structures behave mainly elastically under minor earthquakes, and (ii) plastic design optimisation involves ensuring that structural seismic responses remain within an inelastic range when buildings are subjected to major earthquakes (Zou et al., 2007; Zou and Chan, 2005a, 2005b; Chan and Zou, 2004). Element cross section sizes were considered in (Zou et al., 2007; Zou and Chan, 2005b, 2005a; Chan and Zou, 2004) as the only design variables in the elastic phase to ensure structural serviceable or immediate occupancy under minor earthquakes. Once the optimum section dimensions were decided at end of the elastic optimisation, they were kept unchanged during plastic phase. The design with optimum section sizes was considered as initial design in plastic design optimisation, where cross-section area or arrangement of longitudinal reinforcement was optimised as primary design variable under more rare earthquakes.

2.4.2. Design Constraints

A set of design constraints or "checks" can be used to ensure that each candidate design is acceptable and satisfies design code requirements as well as practical limitations. Design code requirements may include deformation demands, structural geometry and detailing. Structural behaviours constraints in terms of strength, ductility and displacement can also be adopted to ensure structural safety under seismic loads. For example, Akin and Saka, (2015, 2012) adopted design constraints in terms of shear and flexural strength, ductility, serviceability and seismic performances requirements according to design provisions in ACI 318-05.

The concept of performance-based design (PBD) can be incorporated in design optimisation so that structural seismic performance is more directly and efficiently controlled under different earthquake intensity levels. The design criteria are expressed in terms of performance objectives that are statements for acceptable damage levels of structural and non-structural components under different seismic hazard levels, namely Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). In PBD, performance-based target limits are used to quantitatively describe the desired structural safety at each chosen performance level. A structure that satisfies the pre-determined performance-based constraints will maintain the specific performance objective or structural damage level.

Design constraints used in structural optimisation under seismic loads can be categorised into: deterministic and reliability-based constraints.

2.4.2.1. Deterministic Design

A typical deterministic design constraint can be of the format:

$$g_i(x_j) \geq 0 \quad i = 1,2 \ldots N_i \tag{2. 13}$$

where $g_i(x)$ is the constraint relating to design variable x_j, N_i represent the total number of design constraints that should be satisfied in the optimisation process. Apart from inequality constraints, equality constraints can also be used:

$$g_i(x_j) = 0 \;\; i = 1,2 \ldots N_i \tag{2. 14}$$

Previous optimisations used the inter-storey drift ratio to describe structural damage, while limited the response using deterministic constraints (Gholizadeh et al., 2023; Gharehbaghi, 2018; Hajirasouliha et al., 2012; Zou and Chan, 2005a, 2005b; Chan and Zou, 2004). In these studies, the target limits for inter-storey drift were generally considered as 1%, 2% and 4% at IO, LS and CP levels as recommended by ASCE/SEI 41-06, (2007). Moreover, some studies used plastic hinge rotation at each structural element as performance constraints, for which a target limiting value at each specific performance level is decided following load information and section properties at each optimisation iteration, as recommended in ASCE/SEI 41-13, (2014) (Gholizadeh et al., 2023; Zhang and Tian, 2019).

2.4.2.2. Reliability-based Design

In current seismic design codes, seismic uncertainty is commonly addressed by applying a series of factors when deciding seismic design loads, which reflect the influence of site soil conditions, loading characteristics, the importance of structure, and seismic nonlinear behaviours factors. These factors are, in general, decided based on expert judgements and empirical evidence but are not always realistic and rational (Wen, 2001). Meanwhile, uncertainties in other parameters such as structural properties, material properties and numerical modelling can also have significant impact on seismic response. Therefore, a deterministic-based optimisation approach without considering the effects of sources of uncertainties may lead to unreliable evaluation of seismic responses and hence unsafe design solutions.

To address this issue, Möller et al. (2015, 2009) developed seismic design optimisation frameworks for RC frames, by taking into account uncertainties from structural capacities and seismic demands. It was shown that their proposed methods lead to more accurate predictions of seismic response and avoid unexpected failure probabilities. Moreover, in studies by Khatibinia et al. (2013) and Yazdani et al. (2017), sources of uncertainties in material properties of concrete, steel and soil, and in earthquake characteristics as well as their effects on seismic responses were considered in the optimisation process. In a more recent study, Seify Asghshahr (2021) considered the modulus of elasticity and seismic loads as two uncertain parameters.

In reliability-based optimisation, structural and non-structural damage are quantified in a probabilistic manner, while inherent uncertainties and their effects on structural responses are introduced as additional reliability-based constraints. When the concept of performance-based design is implemented, structural reliability is referred to as “failure probability” or “limit-state probability of exceedance”, which reflects levels of confidence that a structural design can perform successfully, without violating target limits under a specific seismic hazard level. Minimum levels of reliabilities associated with chosen performance levels are checked for each candidate design in optimisation to prevent infeasible design solutions. In previous optimisation objective functions, the reliability-based constraints were generally expressed in terms of reliability index, annual failure probability, or mean annual frequencies (rate) of exceedance. Several reliability analysis methods were introduced to calculate failure probability directly or indirectly, including “First-order second-moment” method, Monte-Carlo simulation method, and direct calculation of “limit-state probability of exceedance”, as will be explained in the following sections.

First-order reliability-based method

When FOSM (first-order second-moment) is adopted as the reliability analysis method, the reliability constraint is expressed as a reliability index β_k (Seify Asghshahr, 2021; Zou et al., 2018). The design constraint referring to structural reliability index can be expressed as:

$$\beta_k \geq \overline{\beta_k} \; (k = 1,2,3, \dots N_k) \tag{2. 16}$$

where N_k is total number of performance objectives specified in the optimisation problem, $\overline{\beta_k}$ is minimum target value on reliability index, and k represents k^{th} considered design objective. In addition, the corresponding failure probability (P_f) can be calculated as:

$$P_f = 1 - \Phi(\beta_k) \tag{2. 17}$$

This equation calculates the failure probability using the assumptions that all design variables are normally distributed, and the failure criterion is expressed linearly. $\Phi(.)$ represents standard normal probability distribution.

It should be noted that, in this method, it is necessary to have an exact expression on limit-state function or limit-state surface. To calculate the reliability index (β_k), a simplified way is proposed based on the seismic responses and corresponding target limits at chosen performance levels:

$$\beta_k = \frac{\mu_d^N - \mu_{\Delta\bar{u}}^N}{\sqrt{(\sigma_d^N)^2 + (\sigma_{\Delta\bar{u}}^N)^2}} \tag{2. 18}$$

where $\mu_{\Delta\bar{u}}^N$ and $\sigma_{\Delta\bar{u}}^N$ represent equivalent mean and equivalent standard deviation for inter-storey drift ($\Delta\bar{u}$), respectively; μ_d^N and σ_d^N are equivalent mean and equivalent standard value for allowable drift limit (d) specified in design criteria of PBD, respectively. $\sigma_{\Delta\bar{u}}^N$ and $\mu_{\Delta\bar{u}}^N$ can be then calculated as:

$$\sigma_{\Delta\bar{u}}^N = \frac{\phi\{\Phi^{-1}[F_{II}(\Delta\bar{u})]\}}{f_{II}(\Delta\bar{u})} \tag{2. 19}$$

$$\mu_{\Delta\bar{u}}^N = \Delta\bar{u} - \sigma_{\Delta\bar{u}}^N \times \Phi^{-1}[F_{II}(\Delta\bar{u})] \tag{2. 20}$$

where $\Phi^{-1}(.)$ is inverse of standard normal probability distribution, $\phi(.)$ represents standard normal density distribution, $F_{II}(\Delta\bar{u})$ is cumulative distribution function for seismic response variable (e.g. $\Delta\bar{u}$), and $f_{II}(\Delta\bar{u})$ is corresponding probability density function.

Monte-Carlo simulation method

The Monte-Carlo simulation method is employed to calculate the non-performance probability (P_{np}) especially when a large number of design variables are involved in a complex optimisation problem or when other reliability analysis methods are not suitable (Yazdani et al., 2017; Khatibinia et al., 2013). This method can simultaneously consider limit state functions introduced in PBD guidelines at all performance levels. Equations utilized in the Monte-Carlo method are as follows:

$$P_{np} = \frac{1}{N}\sum_{i=1}^{N} I_i\,(X_u) \tag{2. 21}$$

$$I_i(X_u) = \begin{cases} 1 & if\ G^i(X_u) \leq 0 \\ 0 & if\ G^i(X_u) > 0 \end{cases} \tag{2. 22}$$

where N is total number of independent samples utilized in the method, which are generated according to probability distribution for the uncertain variables (X_u) (e.g. normal distribution), limit state function $G^i(X_u)$ at i^{th} performance level is calculated as:

$$G^i(X_u) = R^i_{limit} - R^i(x_u) \tag{2. 23}$$

where $R^i(x_u)$ represents probabilistic structural seismic response (e.g. maximum inter-storey drift) and R^i_{limit} represents corresponding limiting values. As a result, the reliability constraint ($g^i_R(X_u)$) in the optimisation formula are expressed as:

$$g^i_R(X_u) = \frac{P^i_{np}}{P^i_{np,all}} - 1 \leq 0 \tag{2. 24}$$

where $P^i_{np,all}$ is allowable limiting value for non-performance probability, and i represents the selected performance level (i.e. IO, LS, CP) in performance-based optimum design.

Since the Monte-Carlo simulation method needs to be performed for each sample at each iterative step of the optimisation process, it generally requires very high computational efforts. Therefore, previous studies predicted relevant structural seismic performance mathematically using metamodels, instead of nonlinear time history analysis, to save on computational time (Gholizadeh and Aligholizadeh, 2019).

Direct calculation of limit-state probability of exceedance

The reliability constraint considered in structural optimisation can be also expressed as mean annual frequency (MAF) of exceeding pre-determined limit states (damage states) (Fragiadakis and Papadrakakis, 2008). The MAF ($\nu(EDP > edp)$) is defined as the annual rate that the predicted value of Engineering Demand Parameter (EDP) exceeds its limiting value (edp)

corresponding to a given damage state under a selected earthquake intensity level which is quantified in terms of intensity measure (IM), and it can be calculated as follows (Fragiadakis and Papadrakakis, 2008):

$$\nu(EDP > edp) = \int_0^\infty \left[1 - P\big(EDP > edp(IM = im)\big)\right] \left|\frac{dv}{dIM}\right| dIM \qquad (2.25)$$

where $P\big(EDP > edp(IM = im)\big)$ is limit-state probability (or exceedance probability on a condition of target damage state) that the engineering demand exceeds its threshold value under a given earthquake intensity level $IM = im$, and $\left|\frac{dv}{dIM}\right|$ represents the mean annual rate of the earthquake intensity or the slope of site seismic hazard curve. The EDP is generally written as inter-storey drift ratio, local or global damage index and maximum floor acceleration that can represent either local or global structural damage. The limit-state probability $P\big(EDP > edp(IM = im)\big)$ is further calculated as:

$$P\big(EDP > edp(IM = im)\big) = \Phi\left[\frac{\ln(edp) - \ln(\widehat{\vartheta_{max}})}{\hat{\delta}}\right] \qquad (2.26)$$

where $ln\ (\widehat{\vartheta_{max}})$ and δ represent the logarithmic mean and standard deviation of the response (ϑ_{max}); $ln\ (edp)$ is the logarithmic mean of the pre-determined target limit of the response. Here the probability distribution of the seismic response variable was assumed as logarithmic. Table 2-1 summarises the corresponding target limiting values of several engineering demand parameters (EDP), namely inter-storey drift ratio, floor acceleration and global damage index, at given performance levels which are expressed as expected damage states for a RC structure (Fragiadakis et al., 2006).

Table 2-1: Classification of levels of damage states and corresponding limit-state parameters

Damage state (performance level)	Floor acceleration (a_{floor}) (g)	Inter-storey drift ratio ($\Delta\mu_{max}$) (%)	Mean Damage Index (%)
None	0.15 > a_{floor}	$\Delta\mu_{max}$ < 0.2	0
Slight	0.15 < a_{floor} < 0.30	0.2 < $\Delta\mu_{max}$ < 0.4	0.50
Light	0.30 < a_{floor} < 0.60	0.4 < $\Delta\mu_{max}$ < 0.7	5
Moderate	0.60 < a_{floor} < 1.20	0.7 < $\Delta\mu_{max}$ < 1.5	20
Heavy	1.20 < a_{floor} < 1.80	1.5< $\Delta\mu_{max}$ < 2.5	45
Major	1.80 < a_{floor} < 2.40	2.5< $\Delta\mu_{max}$ < 5.0	80
Destroyed	2.40 < a_{floor}	5.0 < $\Delta\mu_{max}$	100

It should be noted that, to reduce computational costs, reliability-based design optimisation generally introduces source of uncertainties that broadly affect structural seismic performances and design solutions into reliability constraints. Other uncertain parameters are assumed in a deterministic form. For example, most previous reliability-based optimisations used reliability constraints referring to randomness in seismic actions and uncertainties in mechanical characteristics of materials. However, other design parameters, such as the parameter describing characteristic of plastic hinge rotation, the mass of the structure and the detailing of structural geometry, were decided deterministically, meaning their inherent uncertainties and corresponding influences in seismic design were ignored. This simplification may reduce the accuracy in calculation of structural failure probability and affect design checks in accordance with reliability-based constraints. Furthermore, this review found that when the reliability index or failure probabilities were utilized in expressing reliability constraints, there are currently no design criteria on target limiting values for the reliability index. Previous studies utilized user defined limiting values in reliability constraints that do not necessarily lead to rational and reliable design solution whose structural damage is accurately quantified and limited under earthquake excitations.

2.5. Optimisation methodologies

In the field of structural optimisation under seismic loads, three categorises of the optimisation methodology are commonly adopted:

(i) Search-based optimisation,

(ii) Gradient-based optimisation,

(iii) Optimisation using optimality criteria such as Uniform Damage Distribution (UDD) optimisation.

This section summarises the main characteristics of each methodology. The optimisation methodologies adopted by different studies are listed in the Appendix A, while their relative usage is illustrated in Figure 2-6. It can be seen that the search-based optimisation is the most popular method, followed by UDD optimisation.

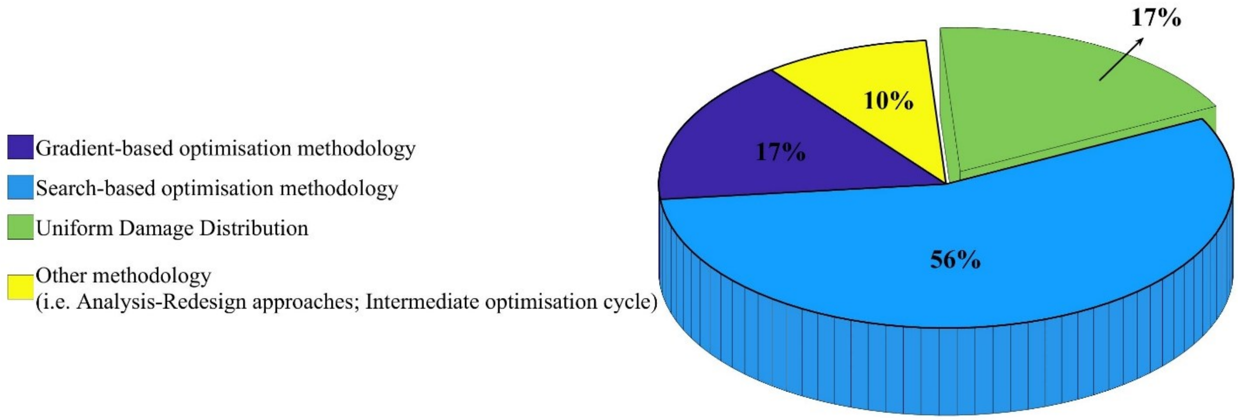

Figure 2-6: Percentage of optimisation methodologies utilised in the previous studies

2.5.1. Search-based optimisation methodology

Search-based optimisation algorithms, also called metaheuristic optimisation algorithms, are generally inspired by natural phenomena such as movement of individuals in a bird flock, or process of natural selection. They include: Genetic Algorithm (GA) (Seify Asghshahr, 2021; Mergos, 2020; Razmara Shooli et al., 2019; Arroyo et al., 2018; Mergos, 2018; Arroyo and Gutiérrez, 2017; Mergos, 2017; Li et al., 2010), Evolution Strategies (ES) (Lagaros and Fragiadakis, 2011; Mitropoulou et al., 2011; Fragiadakis and Papadrakakis, 2008; Lagaros and Papadrakakis, 2007), Chaotic Enhanced Colliding Bodies Optimisation (CECBO) (Gholizadeh and Aligholizadeh, 2019), Harmony Search (HS) (Akin and Saka, 2015, 2012), Particle Swarm Optimisation (PSO) (Razmara Shooli et al., 2019; Esfandiari et al., 2018; Gharehbaghi and Khatibinia, 2015), Gravitational Search Algorithm (Yazdani et al., 2017; Khatibinia et al., 2013), improve muti-verse (IMV), improved black hole (IBH) and modified newton metaheuristic algorithm (MNMA) (Gholizadeh et al., 2023; Razavi and Gholizadeh, 2021). Such algorithms are widely used in structural optimisation and mainly aim to improve objective values through iterations.

As a representative example of search-based methodologies, Figure 2-7 shows a flowchart of the optimisation procedure adopted in the ES algorithm. It consists of details on generating new population and deciding stopping criteria (termination criterion). Generating new individuals in each generation is an essential component in the search-based optimisations. In ES optimisation, when several individuals (potential optimum solutions) are formed in one population, genetic operators in terms of recombination, mutation and selections are processed to denote parent and offspring populations for individuals in the next generation. GA utilises crossover and mutation operators to generate populations for the next iterative step, which is inspired by Darwin's theory of natural selection and evaluation. PSO iteratively adjusts position and velocity of each particle (individual) to search for its best position within the search space. In such a search-based optimisation, multiple candidate designs are generally generated at each iterative step. These designs are evaluated and compared, and the design with best value of the objective function is considered as the “best solution” for that step. The global optimum answer is eventually obtained by exploring the pre-determined search space and comparing the best results across numerous iterative steps.

Search-based optimisation methods have several advantages, including: (i) the optimisation algorithms are capable of handling both continuous and discrete design variables; (ii) multiple design variables can be modified in the optimisation approach; (iii) there is no need of gradient information or exact relationship for objective functions and design constraints, hence they can be easily implemented when obtaining the gradient of objective functions would be difficult; and (iv) the algorithms generally lead to a relatively more global optimum answer, when the pre-determined search space is fully explored.

While there is no apparent relation between the optimisation design objective and the selected methodology, it is noticeable that studies employ search-based algorithms when an optimum design with a goal of initial construction cost minimisation is of interest. This is because costs of RC structures are affected by multiple parameters, including material costs of both reinforcement and concrete. It should be acknowledged that most search-based techniques provide several optimum answers rather than a single result, offering designers choices in finding optimum solutions.

However, search-based optimisation generally requires a pre-determined search space (i.e. database) for design variables. This does not necessarily lead to the best design solution if there are still possibilities out of the search space considered for variable modification. The convergence speed and accuracy of the optimisation are highly depended on the size and the selection of the search space. Such optimisation methods are normally not suitable for complex structures under many load cases and when using time-consuming analyse method, such as non-linear time history analysis, since high dimensional design variables and large sizes of search spaces can result in extremely expensive computational costs. Mahdavi et al. (2015) pointed that the performances of standard metaheuristic algorithms deteriorate for dealing with high dimensional problems mainly due to the landscape complexity and the exponentially increased search space. A large gap is thus found between cases of theoretical optimum designs and practical applications. Moreover, in accordance with “No free lunch” theorems studied by Wolpert and Macready (1997), there is no unique metaheuristic optimisation approach that can provide best answers for all optimisation problems.

To reduce the computational costs, Li et al. (2010) used a hybrid GA and Optimal Criteria (OC) optimisation method that combines the advantages of both methods, and aimed to solve a practical design problem of RC frames with a large number of design variables. Strength and stiffness design optimisation were processed by GA and OC algorithm, respectively, and consequently, an optimum design which satisfies both strength and stiffness constraints was achieved. In a study by Esfandiari et al. (2018), the optimisation implemented a hybrid of multi-criterion decision-making (DM) and PSO, which is helpful to accelerate the optimisation convergence and simplify the optimisation process. Razmara Shooli et al. (2019) also implemented a hybrid GA and PSO optimisation technique to improve populations which were initially generated by GA. The hybrid optimisation helped to achieve an optimum solution for a complex design problem with less computational cost.

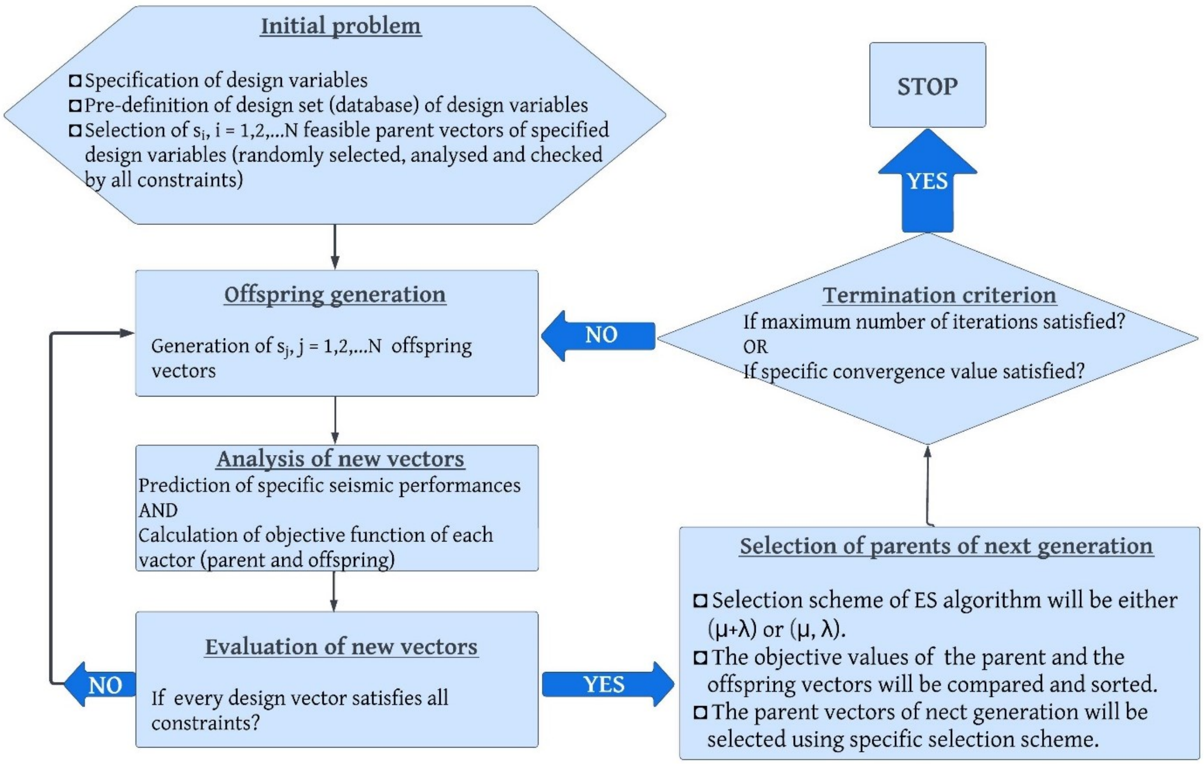

Figure 2-7: Optimisation procedure with Evolution strategies (ES) algorithm

2.5.2. Gradient-based optimisation methodology

Gradient-based optimisation requires gradient information as a pre-determined search direction to search for optimum solutions. These algorithms can be classed into: (i) first-order methods that only require first derivatives of seismic response with a function of design variables, and (ii) second-order methods (gradient-Hessian matrix-based algorithms) which require both first and second derivatives information.

Both gradients of the objective functions and gradients of specific constraints are required in these algorithms. To reduce computational cost and obtain gradient information for a complex optimisation formula, gradient-based optimisation algorithms generally convert the objective functions subjected inequality time-dependent performance constraints into approximately formed unconstrainted functions. The approaches generally involved in the conversion are Lagrange multiplier method (Chan and Zou, 2004), or exterior Penalty function (Razavi and Gholizadeh, 2021; Liu et al., 2010).

For example, the Lagrange multiplier method utilized a Lagrangian function in a previous study by Zou and Chan (2005a), the Lagrangian function is formed by adding the product of each specific constraint function ($F(x_j)$) and its corresponding Lagrangian multiplier (λ_i) to the objective function ($g_i(x_j)$). It transformed a general optimisation formula objective as illustrated in Equation (2.1), into an unconstrained optimisation formula ($L(x_j, \lambda_i)$):

$$L(x_j, \lambda_i) = F(x_j) \pm \sum_{i=1}^{N_i} \lambda_i g_i(x_j) \tag{2. 27}$$

where x_j represents j^{th} design variable of RC building, N_i is total number of performance-based constraints (e.g. inter-storey drift constraints, plastic rotation constraints) subjected in the optimisation problem; a series of λ_i are the Lagrangian multiplier corresponding to the i^{th} design constraint. The Lagrangian multiplier (λ_i), as a crucial component in the Lagrangian function, can be considered as a mathematical concept used to incorporate constraints into the objective function. It enables the identification of optimum solutions that satisfy these constraints while minimising objective $F(x_j)$. It also serves as a factor controlling the impact of specific constraints on the design objective in an optimisation problem.

Various values of the λ_i are decided when the gradient of the Lagrangian function ($L(x_j, \lambda_i)$) with respect to the design variables (x_j) equals to zero (Chan and Zou, 2004), such times points can be considered as stationary conditions of the Lagrangian function or critical points of the optimisation problem, as express as follows:

$$\frac{\partial F(x_j)}{\partial x_j} \pm \sum_{i=1}^{N_i} \lambda_i \frac{\partial g_i(x_j)}{\partial x_j} = 0 \tag{2. 28}$$

In the optimisation framework incorporating with the Lagrange multiplier method, the design variable x_j can be indirectly modified by finding these stationary conditions:

$$S_i = \frac{\sum_{i=1}^{N_i} \lambda_i \frac{\partial g_i(x_j)}{\partial x_j}}{\frac{\partial F(x_j)}{\partial x_j}} - 1 \tag{2. 29}$$

$$x_j^{v+1} = x_j^{v} \times \left[1 + \frac{1}{\eta} S_i\right] \tag{2. 30}$$

where $\frac{\partial F(x_j)}{\partial x_j}$ is the derivative of objective function ($F(x_j)$) with respect to design variable (x_j), λ_j is a parameter that is utilised to convert constrained problem into unconstrained one, $\frac{\partial g_i(x_j)}{\partial x_j}$ is the derivative of i^{h} constraint ($g_i(x_j)$), N_i is total number of performance-based constraints, η is the parameter controlling convergence speed, v represents the iterative step, and S_i is the search direction in the optimisation.

The major advantage of gradient-based optimisation is that it avoids random searching within the identified search domain. As the optimisation approach ensures that any design variable violating design constraints is not involved in the optimisation, the search space of design variables can be reduced and more directed. In general, the gradient-based algorithm leads to a smooth convergence solution since convergence rate is commonly controlled by a parameter in the gradient calculation.

Despite better convergence rates, gradient-based methodologies may lead to local optimum designs if the search direction is not well defined, and it is not easy to assess if a global optimum answer has been reached. Another limitation for the gradient-based methodology is that it is still expensive in computational demand due to complex mathematical models and difficult gradient calculations at each iteration. This issue becomes even more challenging if several design variables of RC frames (e.g. section dimension and reinforcement ratio) are needed to be simultaneously optimised in optimisation approach. Due to their high computational effort, previous gradient-based optimisation approaches avoided utilizing time-consuming seismic analysis method such as nonlinear dynamic analysis, and this can affect the accuracy of the final results.

In order to calculate first- and second-order derivatives of performance-based constraints with respect to design variables, past studies applied principle virtual work (Chan and Zou, 2004) or Newmark direct time integration method (Liu et al., 2010) to express seismic performance as an explicit function of design variables (e.g. section dimensions). However, such explicit functions are normally approximate since any slight changes in member sizes can result in a redistribution of inertia forces and change the natural frequency characteristics. as more details will be explained in the following section.

2.5.3. Uniform Damage Distribution (UDD)

To address the limitations in metaheuristic and gradient-based optimisation methodologies, a new type of optimisation methodology based on concept of uniform damage distribution (UDD) has been developed and widely utilized to solve different design problems in the optimisation field. UDD optimisation utilises an iterative analysis-redesign process, where design variables (e.g. section dimension, reinforcement area, damping coefficient in damper) are redistributed from heavily damaged components to slightly damaged components of a structure until a status of uniform damage distribution is achieved. Such optimisation algorithm is also available to optimise lateral load patterns that simulates the seismic effect in each storey in a structure during seismic analysis (Moghaddam and Hajirasouliha, 2006b). A representative example of UDD formula used in the optimisation is presented as follows:

$$[(A_{rein})_i^j]_{k+1} = \left(\frac{\Delta_i}{\Delta_{target}}\right)^{\alpha} * [(A_{rein})_i^j]_k \tag{2. 31}$$

where $[(A_{rein})_i^j]_{k+1}$ represents specific design variable (here area of reinforcement in j^{th} element) of the i^{th} story at $(k+1)^{th}$ iteration; Δ_i is seismic response result (here maximum lateral inter-storey drift) at i^{th} storey level; Δ_{target} is target value of the selected response parameters; α mainly controls convergence speed in the optimisation process and is ranging from 0 to 1.

In the above formula (2.31), the parameter describing structural damage (Δ) can be considered as any performance parameter, such as: local Park and Ang damage index, storey ductility demand, maximum inter-storey drift and plastic hinge rotations of elements. Using the UDD concept, specified structural damage can be directly controlled and managed based on target limiting values and at specific performance levels (e.g. IO, LS, CP).

The UDD optimisation can be easily implemented for practical design purposes to achieve different multiple objectives including minimum structural damage and minimum total life-cycle cost (Asadi and Hajirasouliha, 2020). It was found to result in safer designs for RC frames, with less concentrated maximum seismic response, and since material capacities in most of stories are fully exploited, it potentially leads to more uniform damage distribution (Hashmi et al., 2022; Hajirasouliha et al., 2012). Previous studies have also demonstrated that the UDD method can speed up the optimisation process, and convergence speed can be directly controlled by choosing a suitable value of the convergence parameter (α) in the UDD formula. The α values around 0.1 were shown to generally lead to good convergence rate without major fluctuations in the optimisation of RC structures. Compared to other optimisation methodologies, such as GA, UDD optimisation required up to 300 times less number of non-linear dynamic analysis (Nabid et al., 2019). Thus, the more accurate but computationally more expensive nonlinear time history analysis can be efficiently used in this optimisation approach.

However, most previous UDD-based optimisation studies only considered maximum inter-storey drift as the performance parameter to control damage, and only limited research monitored structural damage at global level. It should be noted that satisfying lateral drift constraints in a structure does not necessarily control localised damage in structural members. Furthermore, most of previous studies only considered a single performance level, such as LS under earthquakes with an exceedance rate of 10% in 50 years. This does not guarantee safety of the optimal structure in future rarer earthquakes with higher intensities levels.

2.6. Seismic performance evaluation

In general, high-level analysis procedures are required when adopting performance-based design criteria in the optimisation framework. The current seismic design guidelines, such as ASCE/SEI 41-13, (2014), introduced four alternative seismic analysis methods ranging from linear to nonlinear, as summarised in Table 2-2 below. The choice of seismic analysis method depends on several factors: target performance level, seismic hazard level, the importance of structure, structural characteristics (e.g. regularity, complexity, frequencies and donation of mode shapes) and properties of structural modelling (Zameeruddin and Sangle, 2016). It should be noticed that there is no direct correlation between the section on seismic analysis methods and the application of optimisation approaches. However, it can be observed from the details of the reviewed literatures summarised in Tables A.1 – A.3 in Appendix A that: (i) time-consuming optimisation methods, such as most of the search-based and gradient-based optimisation approaches listed above, do not utilise nonlinear time history analysis, considering the factor of computational efficiency involved in the optimisation framework; (ii) low-computational cost UDD optimisation methods generally adopt nonlinear time history analysis to improve the accuracy of the optimum design solutions; and (iii) since gradient-based optimisation methods require derivative information of performance-based constraints as a function of selected design variables, they generally use pushover analysis or even mathematical-based response analysis functions to predict structural response.

Table 2-2: Alternative seismic analysis procedures introduced in code ASCE/SEI 41-13, (2014)

Category	Analysis procedure	Analysis method	Seismic load
Linear	Linear static	Equivalent static analysis	Distributed lateral base shear
	Linear dynamic	Response spectrum analysis	Response spectrum or seismic ground motion record
Non-linear	Non-linear static	Pushover analysis	Response spectrum
	Non-linear dynamic	Time History analysis	Seismic ground motion record

From Figure 2-8, the frequency of different seismic analyses used in optimisation studies are in the following order: nonlinear static (or push-over) analysis, nonlinear dynamic (or time history) analysis, predicting responses using mathematical model and linear analysis. The main characteristic as well as advantage and disadvantages for each of widely used seismic analysis methods are summarised in this section.

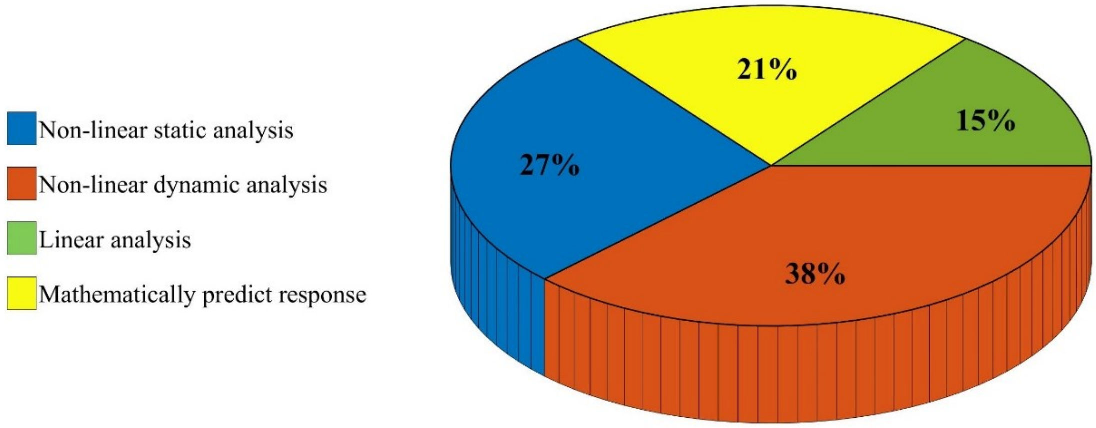

Figure 2-8: Percentage of seismic analysis methods utilized in the previous reviewed studies

2.6.1. Linear static or dynamic analysis

In linear static or dynamic analysis methods, the equivalent static lateral force used to simulate seismic effects on a structure system is derived based on the expected structural behaviours of a linear elastic system. Non-linear ductile behaviours as well as hysteretic energy dissipation capacity are accounted indirectly by simply applying a response modification factor R in ASCE/SEI 7-10, (2010) or a behaviour factor q in Eurocode 8 (CEN, 2004). They are the simplest methods to predict structural responses in seismic events, however, they cannot accurately evaluate seismic performances especially when structures become non-linear (Moghaddam and Hajirasouliha, 2006b). The distributions of equivalent static loads along the height of structures become unrealistic when the lateral inertia forces redistribute after the occurrence of yielding. Only few previous optimisation studies reviewed in this review adopted linear analysis methods, and the analyses were only processed to evaluate seismic performances in elastic phase under minor earthquakes (Seify Asghshahr, 2021; Ontiveros-Pérez et al., 2019; Hashmi et al., 2018).

2.6.2. Non-linear static analysis

Nonlinear static analysis, or pushover analysis, applies a monotonically increasing horizontal lateral load at each storey to push a structure until a collapse mechanism or target displacement at a control point is reached. The increasement of the lateral load can be controlled by either force or displacement. In the previous design optimisation frameworks that used performed pushover analysis, the expected target displacements and maximum seismic responses were evaluated using either Displacement Coefficient Method (Razavi and Gholizadeh, 2021; Razmara Shooli et al., 2019) or Capacity Spectrum Method (Zhang and Tian, 2019; Mergos, 2018a; Bai et al., 2016; Mitropoulou et al., 2011; Zou et al., 2007; Zou and Chan, 2005a; Chan and Zou, 2004).

Displacement Coefficient method

ASCE/SEI 41-13, (2014) design guideline adopts the displacement coefficient method to evaluate the target displacement that refers to the maximum displacement of a characteristic node on the roof, which is calculated using the following formula:

$$\delta_t = C_0 C_1 C_2 S_a \frac{{T_e}^2}{4\pi^2} g \tag{2. 32}$$

where C_0 is used to scale the elastic displacement of a single degree of freedom (SDOF) system to building roof displacement; C_1 is the modification factor reflecting ratio of maximum inelastic displacement to the displacement of a linear elastic structure system with the same unyielding period of vibration; C_2 reflects influences of pinched hysteresis shape, strength deterioration and stiffness degradation on the maximum displacement response, and S_a is calculated by using the design acceleration response spectrum, corresponding to the effective fundamental period T_e. As suggested in code ASCE/SEI 41-17, (2017), T_e in the direction under consideration is evaluated by modifying the fundamental period of a structure (T) as follows:

$$T_e = T\sqrt{\frac{k_i}{k_e}} \tag{2. 33}$$

where k_i is building initial stiffness, and k_e is effective lateral stiffness, which is evaluated by using the idealised force-displacement curve, as shown in Figure 2-9.

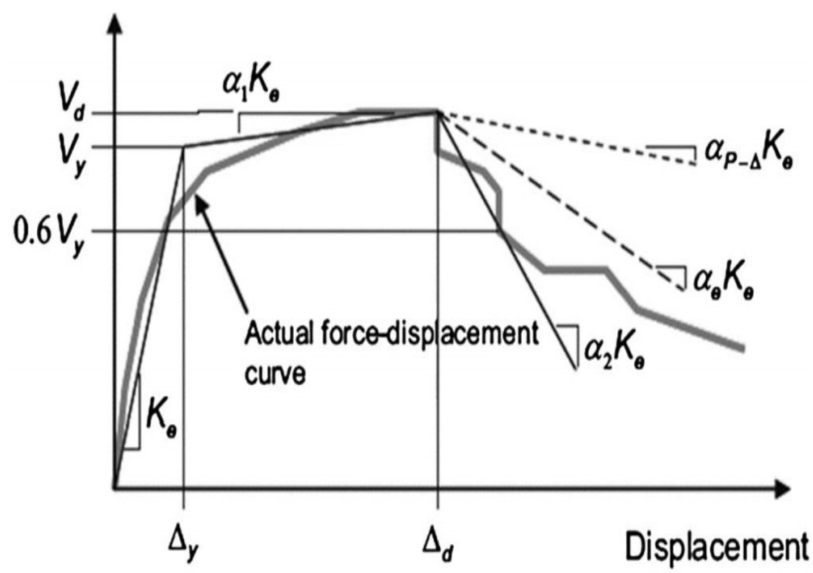

Figure 2-9 Idealized and actual force-displacement curve ASCE/SEI 41-17, (2017)

Capacity Spectrum method

The capacity spectrum method requires to first convert a pushover curve expressed in force-displacement relation to an acceleration-displacement response spectra (ADRS) format that is called capacity spectrum. The capacity spectrum is generally processed based on the assumption that the first vibration mode plays a dominant role in seismic response. The demand curve is then obtained by reducing elastic response spectra to a demand spectrum. The intersection of these two curves is defined as the performance point. At the performance point, seismic responses at both local and global levels are checked against acceptability limits specified in design criteria to ensure structural safety. As describing in code ATC 40, (1996), the target displacement can be evaluated based on deformation demand at the performance point. Zameeruddin and Sangle, (2016) summarised the key procedures utilised in the capacity spectrum method, as shown in Figure 2-10.

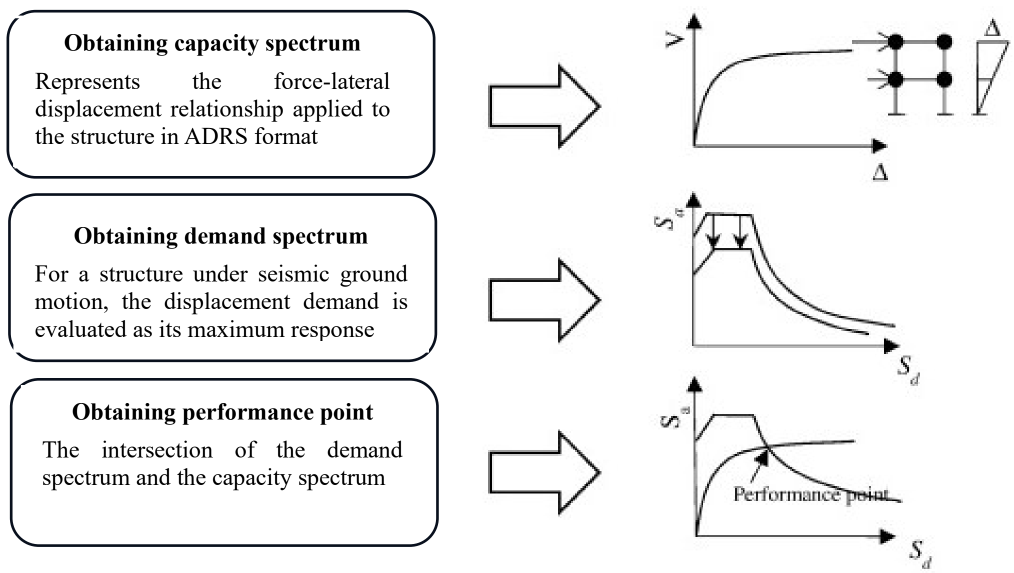

Figure 2-10: Procedures in capacity spectrum method in nonlinear static analysis (adopted from Zameeruddin and Sangle, 2016)

The study by Lagaros and Fragiadakis (2011) implemented different pushover analysis methods as recommended in seismic design codes, namely the displacement coefficient method of ASCE-41, the capacity spectrum method of ATC-40, and the N2 method of Eurocode 8, in the design optimisation of RC frames. The results showed that the ATC-40 method overestimated the seismic demand deformation (i.e. maximum inter-storey drift) and led to higher initial costs, whilst the N2 method and ASCE 41-method provided relatively similar response results especially under low to medium earthquake intensity levels.

The pushover analysis is considered as a powerful tool to not only evaluate structural capacity under seismic loads, but also predict deformation demands (i.e. plastic hinge rotation) of structural elements and structural response at a more global level (i.e. inter-storey drift ratio).

The analysis can also provide information regarding the sequence of element yielding. This analysis method offers advantages in terms of ease of use and speed of implementation.

The pushover analysis was generally utilized in previous optimisation problems when computationally expensive optimisation techniques (e.g. Genetic Algorithm) were used, as it can significantly reduce computation costs compared to the case that non-linear dynamic analyses are utilised. Besides, in gradient-based optimisation, seismic performance parameters (e.g. inter-storey drifts) are needed to be explicitly formulated as a function of structural design variables so that derivatives of the design constraints can be easily calculated. This can be achieved by utilising pushover analysis.

However, most previous pushover analyses utilized an invariant lateral load pattern (e.g. a triangular shape for short- to medium-rise buildings and a unform shape for high-rise buildings) that is approximately proportional to the structural fundamental mode shape or floor mass. This invariant load pattern is generally not consistent with actual conditions as the structural inertia force is redistributed after some yielding in the structure. On the other hand, using invariant load pattern may neglect the effects of higher modes, and hence lead to an underestimation of the seismic responses in the upper stories of high-rise buildings. To address this issue, a previous study Bai et al. (2016) utilized an improved pushover analysis, namely consecutive modal pushover analysis, which can help account for higher-mode effects. Another limitation of the non-linear static analysis is that it can only deal with regular buildings (Mitropoulou et al., 2011). The results of a stud by Mergos (2020) demonstrated that optimum designs obtained using pushover analysis with either unform or first mode-based lateral load patter may not satisfy all performance requirements especially at the local level, when non-linear time history analysis was used to check the final designs.

2.6.3. Non-linear dynamic analysis

Nonlinear dynamic analysis calculates the structural seismic responses at each discrete time step by numerically solving direct integration of equations of input ground motion within a given time domain (Bathe, 2006). Inelastic behaviour can be easily included in structural modelling to deal with geometric nonlinearities and material inelasticity (Pinho, 2007). Seismic ground motion records reflecting realistic acceleration time-histories should be utilised as seismic inputs in the nonlinear dynamic analysis. Selected records may contain different characteristics such as frequency contents, period duration and maximum acceleration. To address uncertainties involved in seismic actions, a group of earthquakes records can be used in optimisation studies, rather than a single earthquake, to capture record-to-record variability. To ensure selected earthquake records are representative at a specific seismic hazard level, the earthquake records are generally scaled so that their mean spectrum compares well with the code-specified response spectra in the range covering the fundamental periods of the referenced frames. Artificial earthquake records, mathematically derived from design response spectrum, are also widely utilized for time history analysis in optimisation problems.

Non-linear dynamic analysis is currently considered as the most powerful and accurate tool to predict structural behaviours especially under strong earthquake events. It is capable of predicting deformation demands under cyclic loads and considering influences of stiffness and strength degradation. It also provides information regarding the amount of structural energy dissipation due to the hysteretic behaviour and cyclic responses (Fragiadakis and Lagaros, 2011). It should be noted that, nonlinear dynamic analysis is also appropriate to evaluate the seismic responses of irregular RC buildings, as demonstrated in previous optimisation studies (Lavan and Wilkinson, 2017; Mitropoulou et al., 2011). However, the analysis increases considerably the analytical complexity and computational costs especially in optimisation frameworks where a larger number of analyses are required.

2.6.4. Evaluate seismic response using mathematical equations

Seismic responses can also be evaluated by using the displacement matrix method, which uses the stiffness matrix and its linear relationship with load to predict seismic performances (Akin and Saka, 2015, 2012). However, this method is based on the structural elastic behaviour and thus cannot accurately predict inelastic responses. When dealing with nonlinear responses in the design optimisation framework, several past studies employed a metamodel involving neural network (NN), based on the concept of machine learning, to produce predictions of seismic responses (Gholizadeh and Aligholizadeh, 2019). The metamodel was trained to evaluate output vectors in terms of structural seismic responses by adopting suitable input vectors with reference to selected design variables (e.g. first natural period of structure, section properties). Complex mathematical functions in terms of wavelet function were then established to represent the relationship between inputs and outputs. A large database of the design variables was required to ensure acceptable accuracy of the prediction results. In addition, a metamodel that was constructed based on the concept of artificial neural network and combined weighted least squares support vector machine with the wavelet kernel theory was trained as an additional optimisation framework to predict reliable average inelastic responses under earthquake records. In this additional optimisation, the responses were predicted by minimising the objective function, while a gravitational search algorithm was applied to increase the accuracy of the output response (Yazdani et al., 2017; Khatibinia et al., 2013). However, proposing a suitable metamodel can be challenging since several controlling parameters are required in the evaluation of seismic performance. Furthermore, the computational effort needed to develop the databases of the input variables for a specific structure, are likely to exceed the efforts of a conventional optimisation of the same sophistication level.

2.7. Summary and Research gaps

Conventional seismic design methods that follow a “trial-verification-modification” may not necessarily lead to safe and economic efficient design solutions for multi-storey RC buildings especially under strong seismic events. To address potential issues in current seismic design methods, this review confirms that whilst different methods have been proposed for structural size optimisation problems, overall the main steps involved in all previously reviewed optimisation methodology generally imply: (i) defining optimisation objective(s), (ii) selecting design variables and design constraints that are involved in the objective formulation, (iii) analyse seismic response and calculate the objective function(s) (iv) applying optimisation methodology. This chapter critically reviews these steps in above sections, respectively.

Different objectives functions proposed previously to solve the design problems were discussed here, mainly including minimum structural damage and minimum economic cost. They were further categorised into single-objective optimisation and multi-objective. The most common design variables considered in the structural size optimisation, as well as deterministic and reliability-based design constraints were reviewed. Subsequently, different optimisation methodologies and their advantages and limitations were discussed in detail. Based on the critical review conducted in this study, the following research gaps are identified:

- **Formulation of design optimisation problem**

In structural optimisation, a design objective is quantified using a mathematical function with reference to selected design variables and is subjected to a set of design constraints. However, most of current optimisation studies simplify the objective "minimum initial construction cost" to "minimum total concrete volume and reinforcement weight". Such a simplified quantification cannot represent real construction costs, as it excludes labour, management and storage fees and their relative costs. For example, in a country where market prices of concretes are extremely low, minimising total concrete volume cannot necessarily reduce total economic cost, as the optimum design may add to the cost of formwork, labour and fabrication.

Most optimisations studies only consider a single design variable as either "reinforcement ratio" or "section dimension". However, the structural seismic behaviours are affected by both variables, and they are also affected by detailing of the steel reinforcement (e.g. laps and anchorage of longitudinal reinforcement, spacing of transverse reinforcement, volumetric ratio of transverse reinforcement, amount of steel reinforcement within critical region such as element ends). This may appreciably affect the outcome of the optimum answer and may result in unexpected failure modes such as brittle shear failures.

Another limitation is the use of deterministic design constraints, without accounting for the sources of uncertainties and their influences on seismic designs. Ignoring uncertainties may also lead to a design solution with unacceptable failure probabilities and expensive economic losses after earthquakes. In previous reliability-based optimisation studies, only limited sources of uncertainties were accounted, and there are no design criteria specifying target limiting values for reliability constraints.

- **Seismic performance assessment**

As recommended in current design codes, nonlinear static analysis is widely adopted for the seismic performance assessment of RC structures due to its computational efficiency. However, the conventional pushover analysis with invariant load pattern cannot accurately predict the inelastic responses especially for tall buildings, where their structural behaviour is affected by higher modes. Similarly, other simplified structural models such as shear building model can overestimate lateral deformations, they are difficult in evaluating force and deformation demands at local level, and can underestimate structural localised damage (Hajirasouliha and Doostan, 2010).

For structural optimisation of structures under seismic loads, the variability in earthquake inputs is generally considered as the main source of uncertainty that may significantly affect evaluation of structural seismic performance. However, current optimisation frameworks mainly employed deterministic analysis, and the performance parameters were directly checked against the corresponding target limits by subjecting to the deterministic constraints, without considering the failure probability.

Most current performance-based design optimisations aimed to satisfy a single performance objective under a specific seismic hazard level. Therefore, they do not necessarily ensure “structural safety” in future earthquake events with higher intensity levels. Furthermore, the maximum inter-storey drift, commonly used as a single performance parameter, cannot identify structural damage at both local and global levels.

- **Application of optimisation methodology**

Although many metaheuristic algorithms have been used in structural optimisation, these may lead to different answers on a specific design objective even for the same design case. Therefore, each suggested methodology may only perform well for a specific design problem, and it is hard to determine if the obtained optimum design is the “best” design solution.

Most previous optimisation studies utilised the metaheuristic algorithms due to their simplicity, easily implementing without completed mathematical formulations and no need for gradient information. However, these methods generally require higher computational effort with increasing the number of design variables, especially when high-rise non-linear buildings are considered. Their computational effort is also significantly increased when combing with a computationally expensive seismic analysis method (i.e. non-linear dynamic analysis) to increase the accuracy of evaluation for seismic responses especially within inelastic range, as a large number of iterative steps and nonlinear analyses are generally required to obtain the optimum solution.

2.8. Future directions

In accordance with the identified research gaps, the proposed future directions are summarised below:

- Cost minimisation should be achieved by considering all relevant aspects in the cost evaluation. This will include current material prices and labour costs depending on place of construction, transport and storage costs influenced by urban environment, and fabrication cost relating to different construction methods (e.g. pre-cast and cast in place concrete).

- Life-cycle cost should be considered as another necessary aspect in estimating the economic cost of optimum buildings under unpredictable earthquake loads, as it accounts for not only potential repair costs due to structural and non-structural damage, but also relevant indirect losses caused by future earthquakes.

- Seismic design of RC frames is generally required to satisfy several conflicting design requirements in terms of economy, safety and sustainable. Hence, multi-objective optimisation is necessary in future development.

- For an optimisation framework incorporating the concept of performance-based design, there is a need to define multiple performance objectives under various seismic hazard levels ranging from earthquakes with low intensity to more severe seismic events. Structural performances at both element and structural levels should be simultaneously controlled to satisfy these performance requirements and to minimise structural damage.

- To assess a more reliable and accurate seismic performance, high-level seismic analysis, such as non-linear dynamic analysis, is recommended especially for structures located in a seismic region with high earthquake intensity levels. Variability in seismic ground motion records utilised in the analysis is needed to be considered and managed to reduce their impacts on optimum solutions.

- The optimisation framework should provide a practical and rational design solution that balances structural economy and safety by addressing inherent uncertainties in material properties, geometrical details and modelling assumptions. It is suggested to improve the control of seismic performance by calculating the probability of exceedance conditional on specific limit states, rather than checking against performance-based limiting value at each performance level in a deterministic manner.

- Environmental impacts, such as CO_2 emissions and greenhouse gas emissions, can be considered as an additional design objective to be minimised for RC frames, to achieve a more sustainable design solution.

- The effect of plan irregularities should be considered in the optimisation process by utilising three-dimensional model. However, this will significantly increase the computational costs of the optimisation especially in the case of non-linear structures.

CHAPTER 3 : Multi-level Performance-based Seismic Design Optimisation of RC Frames

3.1. Abstract

Conventional structural optimisation techniques often result in unconventional structural configurations, unrealistic structural elements and ignore actual construction costs. This study presents an effective performance-based optimisation framework for minimising initial material costs of realistic multi-storey reinforced concrete (RC) frames, while satisfying pre-determined performance targets under multiple seismic hazard levels as well as a set of practical design and construction constraints. A new low computational-cost optimisation method is proposed to directly control specific response parameters at both the element and structural levels (i.e. plastic rotation and inter-storey drift). For the first time, the concept of Uniform Damage Distribution (UDD) is adopted to simplify the complex design optimisation problem of RC buildings with multiple design variables in terms of section sizes and reinforcement ratios. The optimum design solution is achieved by gradually redistributing materials from strong to weak parts of the structure, aiming to fully exploit the material capacity. The efficiency of the proposed optimisation framework is then demonstrated in the optimum designs of 3-, 5-, 10- and 15-storey RC frames under a set of six spectrum-compatible earthquake records. The results indicate that compared to structures designed by current codes, optimum solutions required up to 20% and 43% less concrete volume and steel reinforcement weight, respectively. It is also noted that due to more efficient use of materials, optimum structures exhibited less maximum inter-storey drift (up to 58%), less maximum plastic rotations (up to 78%). The structures also significantly reduced structural global damage (up to 88%), which is computed in a quantitative and deterministic manner using a damage index parameter. Sensitivity analysis on earthquake record selection shows that using a single earthquake record may not lead to reliable design solutions, in

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particular for tall buildings, and hence a set of spectrum-compatible records should be used in the optimisation process. This research will lead to more economical and safe design of multi-storey RC structures in seismic regions by developing a practical multi-level optimisation method with low computational costs.

Keywords Structural optimisation; Reinforced concrete frames; Multi-level performance based design; Global damage index; Nonlinear dynamic analyses

3.2. Introduction

Current seismic design guidelines (e.g. Eurocode 8 (CEN, 2004a), Chinese code GB 50011 (National Standard of the People's Republic of China, 2010), IBC 2021 (ICC., 2020)) generally adopt "strength-based" or "force-based" design principles. While these methods mainly ensure overall structural capacity, they cannot directly control member deformations and lateral drifts and in turn efficiently limit structural and non-structural damage under earthquakes. Moreover, most seismic design codes, such as Eurocode 8, mainly aim to satisfy "life safety" requirements by preventing local or global structural collapse under design level seismic action (10% probability of exceedance in 50 years) and, hence, may not satisfy target performance objectives under other seismic hazard levels. It should be noted that, structures designed using modern codes successfully protected occupants' lives in recent major earthquake events (e.g. Christchurch 2010-2011, Northern Italy 2012, Kumamoto 2016); however, economic losses due to repairable and non-repairable damage were extremely large in some cases (Takeda and Inaba, 2022; Meroni et al., 2017; Stevenson et al., 2014).

In current seismic design codes, the equivalent static lateral force determined to simulate seismic loads is based on the dynamic behaviour of linear elastic systems. However, typical RC structures do not generally remain elastic under severe earthquake events. In conventional seismic design approaches, structural nonlinearity and hysteretic energy dissipation capacity are generally taken into account by using a response modification factor (e.g. ASCE/SEI 41-13, (2014)) or a behaviour factor (e.g. Eurocode8 (CEN, 2004a)). These factors are normally decided based on judgment and empirical evidence, and do not necessarily lead to most suitable design solutions, which is also confirmed by results from previous experimental and numerical studies (Feng et al., 2016; Lu et al., 2016). On the other hand, while push-over analyses suggested in current seismic design guidelines aim to provide better predictions of structural seismic responses, previous studies have identified that: (i) the fixed load patter used in the pushover analysis may be unrealistic when lateral inertia loads and storey stiffness change due to the occurrence of yielding and nonlinear structural behaviour; and (ii) the push-over analysis does not directly account the contribution of higher modes on structural behaviour, which can be especially important for high-rise buildings (Hajirasouliha and Pilakoutas, 2012; Moghaddam and Hajirasouliha, 2006a). The study by Elnashai, (2001) indicated that using a single pushover analysis cannot duplicate the interaction between the continuously changing dynamic characteristics of inelastic structural system with the various frequencies of earthquake records.

Performance-based design (PBD) has been introduced in more recent seismic resistant design guidelines (e.g. ATC 40, (1996), FEMA 356, (2000), ASCE/SEI 41-17, (2017)) and is intended to address some of the limitations of the conventional "force-based" design methods (Krawinkler and Miranda, 2004; Ghobarah, 2001). In PBD, a set of design criteria are expressed in terms of performance objectives that directly correspond to specific requirements for the building (i.e. immediate occupancy, life safety, collapse prevention) under different seismic hazard levels. This provides a more direct and rational approach for controlling structural and non-structural damage during the seismic design process. Mergos (2017) compared to conventional design solutions (e.g. Eurocode 8 force-based design methods) subjected to earthquakes with different hazard levels. It was found that in several cases they failed to satisfy performance constraints on element plastic hinge rotations set by performance-based design criteria (e.g. MC2010). Though it is accepted that PBD can provide better control of structural damage during seismic events, it still does not necessarily guarantee the most efficient design.

The concept of "optimal design" has been widely utilised for different structural systems. For instance, Foraboschi, (2014) searched for the best thickness of the glass layers and the stillness of the interlayer to optimise the cost of plates made of glass, while fulfilling strength and deflection design requirements. In another study (De Domenico and Hajirasouliha, 2021) aimed to minimise structural damage of steel frames with nonlinear viscous dampers by remodifying damping coefficients of the dampers. RC frames are the most common structural system used worldwide for low and medium rise buildings. Even though, their overall structural design is relatively straight forward, obtaining their optimum design solution can be very challenging due to cracking of concrete affecting the lateral stiffness and inertia load distribution, as well as the non-linear behaviour of the structure mainly caused by the yielding of reinforcement.

Several design optimisation studies on RC frames have been published in the past 20 years. Chan and Zou, (2004) and Zou et al. (2007) aimed to obtain the optimum design of RC frames by employing an Optimality Criteria (OC) performance-based methodology. Both section sizes and steel reinforcement quantities were considered as design variables and optimised based on the performance results of elastic and inelastic (push-over) analyses, respectively. To achieve the optimum solution, objective function(s) and the subjected design constraints were first converted into an unconstrained formula using the Lagrange multiplier method. This involves creating a Lagrangian function by combining the constraint functions, the corresponding Lagrange multipliers and the objective functions. Then the stationary condition of the Lagrangian function was evaluated to iteratively modify the specific design variables. In another study, Bai et al. (2016) developed an optimisation technique based on the concept of Optimality Criteria (OC) to achieve more uniform distribution of storey drifts. Inelastic response demands were evaluated through consecutive pushover analysis. Reinforcement areas of beams and columns were iteratively modified in accordance with storey lateral drifts and element hinge rotations, simultaneously. Furthermore, Liu et al. (2010) used a second-order optimisation method for elastic seismic drift design of RC frames. This required to first transfer a constrained problem into an unconstrained optimisation formulation through an interior penalty function. Subsequently, the first and second derivatives of the penalty function were calculated to achieve seismic design with minimum structural weight. Dimensions of beams and columns were considered as the only design variables in the objective function. In a more recent study, Papazafeiropoulos et al. (2017) used a gradient-based first-order optimisation methodology to achieve uniform distribution of dissipated energy for RC frames by optimising the distribution of structural stiffness.

Recently, evolutionary algorithms such as Genetic Algorithm (GA) and Evolution Strategies (ES) are also used in seismic design optimisation of RC frames (Arroyo and Gutiérrez, 2017; Gholizadeh and Salajegheh, 2010; Fragiadakis and Papadrakakis, 2008; Lagaros and Papadrakakis, 2007). Mergos (2018a, 2017) utilised GA in the optimum seismic design of RC frames in accordance with both force-based and performance-based design methods to minimise material costs. Section dimensions, diameter and number of longitudinal reinforcement bars, and diameter and number of transverse reinforcement bars were considered as design variables and were independently remodified according to a set of design constraints. In another study, Mitropoulou et al. (2011) applied ES for multi-objective optimisation. To assess structural performance, both nonlinear static and dynamic analyses were conducted, while discrete design variables including dimensions of members, longitudinal and transverse steel reinforcements were considered. Razmara Shooli et al. (2019) also adopted a mixed GA and Particle Swarm Optimisation (PSO) in conjunction with nonlinear static and dynamic analysis methods for PBD optimisation of RC frames. Their optimisation method was first processed with nonlinear static analyses to obtain the optimum search domain involving specific design variables, and then nonlinear dynamic analyses were performed to find the optimum result with minimum material cost within the identified search domain. Gholizadeh and Aligholizadeh, (2019) performed a reliability-based design optimisation of RC frames, where a Chaotic Enhanced Colliding Bodies Optimisation (CECBO) metaheuristic algorithm in conjunction with a metamodel was adopted to search for optimum solutions in a specific design space. In another relevant study, Razavi and Gholizadeh, (2021) utilised the Improved Black Hole (IBH) metaheuristic algorithm to minimise initial cost and total life-cycle cost, as two different independent optimisation objectives. Specific design variables were optimised by using pre-determined databases and based on seismic responses obtained from pushover analysis.

In general, the above-mentioned optimisation methodologies can be classified into two categories: (i) mathematical programming algorithms, such as OC method and gradient-based algorithms, incorporate mathematical concepts into the optimisation method, and (ii) search-based optimisation methodologies, including GA, ES and PSO, aim to obtain the best solution by searching for satisfactory values of specific design variables within a predetermined design space. Most previous optimisation studies on RC frames adopting math-based algorithms required complex mathematical formulas to transfer inequality constraints and objective functions into unconstrained problems. They also required a high computational effort to calculate the derivatives of the objective functions at each optimisation step, particularly in the case of nonlinear systems under dynamic loads. Search-based design optimisation methods are also computationally expensive (i.e. require thousands of analysis iterations), while their optimisation speed and accuracy depend on the pre-determined search domain. Moreover, more than half of previous optimisation studies adopted nonlinear static (pushover) analysis with predefined lateral load patterns to predict the seismic behaviour of structures. However, as mentioned before, using the fixed load patterns may not represent the actual seismic effects in a non-linear structural system. These limitations increase costs and limit accuracy, hence, may prevent engineers from using these optimisation methods in practical applications.

To reduce computational costs, Hajirasouliha et al. (2012) developed a practical optimisation methodology for the seismic design of RC frames, based on the concept of Uniform Damage Distribution (UDD). Previous study demonstrated that this approach can significantly reduce the computational costs (up to 300 times less number of non-linear dynamic analysis) of the optimisation process of complex non-linear systems compared to the metaheuristics optimisation methods such as GA and PSO (Mohammadi et al., 2019; Nabid et al., 2019). According to the design philosophy of UDD, structural materials are redistributed iteratively from low- to high-damaged areas until a state of nearly uniform height-wise distribution of structural damage is achieved. In follow-up studies, the UDD concept was adopted for the optimum seismic design of RC frames (Asadi and Hajirasouliha, 2020; Bai et al., 2020). However, these studies mainly considered maximum inter-storey drift as the single performance index, which cannot comprehensively identify structural damage at both local and global levels. It should be noted that, the selection of performance levels plays an important role in performance-based optimisation methods. Previous optimisation studies using UDD, mainly considered a single performance objective (i.e. life safety) under a certain seismic hazard level (i.e. 10% probability of exceedance in 50 years). This may not necessarily lead to a safe design solution in rarer earthquakes with higher intensity levels when high localised damage may develop (De Domenico and Hajirasouliha, 2021).

In general, optimisation of RC frames is a complex problem since both reinforcement arrangement and weight, and concrete volume can significantly affect seismic responses of structures. Foraboschi, (2019) studied the bending load-carrying capacity of RC beams allowing for ductility and concluded that a blending increasement in amount of reinforcement of the RC element does not compensate a reduction in its cross-section size. Indeed, many of the previous studies on the optimum design of RC frames only optimised the reinforcement ratios of the elements, while the initial dimensions were obtained based on existing seismic design guidelines and then kept unchanged during the optimisation process (Li et al., 2019; Bai et al., 2016; Hajirasouliha et al., 2012). However, these two design variables are not independent, as structural ductility and deformability are affected by both parameters. On the other hand, Arroyo and Gutiérrez, (2017) and Arroyo et al. (2018) mainly aimed to improve the elastic performance of the structural system by optimising the dimensions of the structural members, while the reinforcement bars were designed using current design codes after the optimisation stage. However, the optimum solution in this case may not guarantee the structural safety in future rare earthquakes, when structures are loaded beyond the elastic range.

The objective of this study is to develop an efficient multi-objective performance-based optimisation framework for seismic design of RC frames based on the concept of Uniform Damage Distribution (UDD).

In this UDD-based approach, the specific design variables change iteratively to closely approach the performance target limits. In more details, cross-section sizes are first optimised in elastic phases to control drift in each storey, and their optimum answers are considered as initial design in the second (plastic) phase of optimisation, where longitudinal reinforcement ratios are considered as main design variable, modifying based on performance results of plastic rotations and drifts. Consequently, material capacities in most storeys are to be fully exploited at least under one of the three seismic hazard levels, selected in the optimisation procedure, and hence a design solution with minimum material usage is obtained by satisfying all the performance targets corresponding to multiple design objectives. The novelty of the proposed framework is that, for the first time, the UDD approach is implemented to accommodate: (i) multi-level performance objectives under different seismic intensity levels ranging from elastic to inelastic states; (ii) optimising both section sizes and reinforcement ratios; and (iii) controlling both local (element plastic rotation) and global (inter-storey drift) performance indices simultaneously with low computational costs. The efficiency of the framework is demonstrated in the design of 3-, 5-, 10- and 15-storey RC frames under a set of spectrum-compatible earthquakes. The framework is further developed to investigate the effect of variability in the selected earthquake input records, here the effect of earthquake record selections is assessed by repeating the same optimisation process under a randomly selected single artificial earthquake record.

3.3. Performance-based optimisation framework

In this study, the optimisation objective is to minimise the total material usage (both concrete and reinforcement), while satisfying a set of performance constraints to control local and global structural damage under different earthquake intensity levels. This is achieved based on the concept of Uniform Damage Distribution (UDD), and by incorporating the design criteria in PBD. In accordance with ASCE/SEI 41-13, (2014) recommendations and seismic hazard studies in several seismic source areas, Table 3-1 shows the performance objectives that should be satisfied in the multilevel performance-based optimisation, their corresponding seismic hazard levels (here expressed as occurrence probability of the earthquakes), their relationship with magnitude of peak ground acceleration (PGA) for the design earthquakes is decided based on seismic hazard maps results in the previous case studies (Cheng et al., 2007; Yahya et al., 2016).

Table 3-1: Relationship between performance objectives, corresponding seismic hazard level and peak ground acceleration

Performance objective	Earthquake excitation	Occurrence Probability	Return Period (year)	PGA (g)
Immediate Occupancy (IO)	Frequent earthquake	50% in 50 years	72	0.1
Life Safety (LS)	Design basis earthquake (DBE)	10% in 50 years	475	0.4
Collapse Prevention (CP)	Maximum considered earthquake (MCE)	2% in 50 years	2475	0.65

3.3.1. Formulation of multi-objective optimum design problem

The design constraints on geometry and reinforcement detailing of beam and column elements are based on Eurocode 2 and 8 (CEN, 2004a, 2004b) recommendations for medium ductility level (DCM). Key practical design considerations adopted in common practice were also considered in the optimisation process. The overall optimisation problem can be expressed as:

Minimise:

$$V_c, W_s \tag{3.1}$$

Subject to:

$$\theta_c \leq \theta_{target,c}, \theta_b \leq \theta_{target,b}$$

$$\Delta_{max} \leq \Delta_{target}$$

$$\rho_{c,min} \leq \rho_c \leq \rho_{c,max}$$

$$\rho_{b,min} \leq \rho_{b,top} \leq \rho_{b,max}$$

$$\rho_{b,min} \leq \rho_{b,bottom} \leq \rho_{b,max}$$

$$D_{min} \leq D,\ B_{min} \leq B,\ H_{min} \leq H$$

where V_c is the total concrete volume in the frame, W_s is the total longitudinal reinforcement weights. θ_c and θ_b are plastic rotations of columns and beams, respectively. Δ_{max} denotes maximum inter-storey drift ratio. ρ_c is longitudinal reinforcement ratio of columns, while $\rho_{b,top}$ and $\rho_{b,bottom}$ are the ratio of beam top and bottom reinforcement, respectively. D is the dimension of column sections assumed to be square, and B and H are the width and depth of beam sections, respectively. The subscript "min" represents the minimum values of cross-section dimensions suggested by Eurocode. It should be noted that the optimisation framework proposed in this study is general and able to be adopted for any seismic design codes to obtain the most suitable design solution.

3.3.2. Design constraints

To achieve DCM in Eurocode 8 (CEN, 2004a), the minimum dimension of concrete sections is limited to 250 mm, and minimum and maximum reinforcement ratios in columns are 1% and 4%, respectively. Upper and lower reinforcement limits in beams are also imposed. To promote the "strong column/weak beam" design principle, at each beam-column joint the sum of the flexural stiffnesses of beams is designed to be less than the flexural stiffness of columns. Additionally, according to practical considerations from engineering experience, it is recommended that the width of beams should be always less than the dimensions of columns, and dimensions of columns shouldn't be less than the ones in upper storeys.

3.3.3. Design variables in UDD optimisation

As mentioned in the above problem formulation, this study aims to minimise both rebar weights (kg) and concrete volume (m^3) of RC frames. Column dimensions (*D*) as well as beam width (*B*) and depth (*H*) are accounted as discrete design variables, since their adjustments in practical designs typically occur in increasements (decrements) of 50 cm, which is not continuous modification. While longitudinal reinforcement ratios in columns (ρ_c) and beams ($\rho_{b,top}$, $\rho_{b,bottom}$) are considered as continuous variables in the optimisation process, these continuous ratios can be further converted into various combinations of rebar numbers and diameters. The proposed study assumes that each RC member has adequate amount of transverse reinforcement that is approximately proportional to the longitudinal reinforcement quantity. The values of all the selected design variables also satisfy the practical and code-based design constraints in each iterative step during the optimisation procedure.

3.3.4. Performance parameters and design targets

The current method considers plastic hinge rotations in beams (θ_b) and columns (θ_c) as primary performance parameters to measure local element response quantities under medium to severe earthquakes, while the inter-storey drifts (Δ_{max}), as more global performance parameters, are also simultaneously controlled in the optimisation process. It should be noted that the performance parameters in the proposed optimization methodology are selected based on the suggestions in ASCE 41. However, the adopted UDD method is general, and can be efficiently applied to any other performance parameters such as floor acceleration and velocity.

Axial load ratios and transverse reinforcements ratios are important parameters that affect the rotation capacity of RC elements under earthquake loads, while the flexural capacity is also influenced by shear loads (Belkacem et al., 2019). Yuen et al. (2017) shows that increasing axial load ratios reduces ductility and energy dissipation capacity of flexure-dominated columns. Therefore, the plastic hinge rotation target limit (capacity) cannot be pre-determined and should be updated following the section properties and loading information (i.e. shear load, axial load) at each iterative step of the optimisation procedure. This increases the complexity of the optimisation problem for RC frames under multiple earthquake intensity levels.

Once the target performance level is decided (e.g. IO, LS, CP), target limits (capacities) of plastic hinge rotation of beams $\theta_{target,B}$ and columns $\theta_{target,C}$ can be calculated following ASCE/SEI 41-13, (2014) guidelines as presented in Table 3-2 and Table 3-3, respectively. In these tables: 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.

Table 3-2: Column plastic rotation capacity ($\theta_{target,C}$) (unit: rad) (ASCE/SEI 41-13, (2014))

$\frac{P}{A_g f_c'}$	$\frac{V}{b_w d\sqrt{f_c'}}$	$\rho = \frac{A_v}{b_w s}$	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 3-3: Beam plastic rotation capacity ($\theta_{target,B}$) (unit: rad) (ASCE/SEI 41-13, (2014))

$\frac{\rho - \rho'}{\rho_{bal}}$	$\frac{V_b}{b_w d\sqrt{f_c'}}$	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

To constrain performance at structural level, the target inter-storey drift ratios (Δ_{target}) are defined as 1%, 2% and 4% at performance levels IO, LS and CP, respectively, in accordance with ASCE/SEI 41-06, (2007). It should be noted that, the proposed methodology is general and other drift limits can be easily adopted. To provide more accurate results, structural response parameters are obtained through non-linear time history analysis (NTHA) using OpenSees software (McKenna et al., 2006). The iterative optimisation process based on UDD concept is performed by a specifically designed subroutine in MATLAB (MATLAB R, 2020).

3.3.5. Multi-level UDD optimisation

The seismic design of RC structures is commonly based on the assumption that the buildings experience nearly elastic response under frequent earthquakes and mainly behave inelastically under moderate to severe earthquakes. The entire design optimisation procedure can thus be categorized into two phases: “elastic phase” and “inelastic phase”. The proposed multi-level UDD optimisation methodology aims to improve the design solutions based on current design standard. The optimisation is implemented after the initial code-based design is achieved. It is important to note that the methodology is general, and any design guidelines, such as Chinese code GB 50011 or American code IBC 2021, can be selected to process the initial designs. The optimum solutions won't be significantly affected by the initial design results.

3.3.5.1. Elastic Phase: Element Size Optimisation

The performance objective at this stage is to satisfy Immediate Occupancy (IO) criteria under frequent earthquakes in accordance with 50% probability of exceedance in 50 years (here PGA= 0.1g as shown in Table 3-1). The key performance parameter for controlling the structural response of elastic (or near-elastic) systems is considered to be inter-storey drift ratio (IDR). The first phase considers elements sizes as a single design variable, considering that the concrete section size plays a more dominant role in providing lateral stiffness and hence controlling inter-storey drift ratios. The element size optimisation algorithm is briefly summarised as follows:

1. The RC frame is initially designed in accordance with a conventional code-based design method. In this study, Eurocode 8 is used for preliminary designs of the selected frames. The details of cross-section dimensions and reinforcement ratios of the initial design solutions are provided in Appendix B.
2. The designed structure is subjected to a set of spectrum-compatible frequent earthquake records (scaled to PGA = 0.1g) corresponding to IO performance level. Maximum IDR at each storey ($\Delta_{max,i}$) is obtained as average of the maximum values under the selected earthquakes to capture record-to-record variability, through non-linear time-history analysis, using the following Equation (3. 2):

$$\Delta_{max,i} = \frac{\delta_i - \delta_{i-1}}{h_i} \tag{3.2}$$

   where, δ_i and δ_{i-1} are 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.
3. When the IDR in a certain storey is higher than the target limit, the specific performance objective is in turn violated, and hence structural capacity should be increased by adding more material. On the other hand, in storeys where IDR is less than the target value, structural materials are not fully utilised. Therefore, the concrete section dimensions (here size of column square cross-sections (*D*), width (*B*) and depth (*H*) of beam rectangular cross-sections) are reduced or increased accordingly as discrete design variables by using the Equations (3. 3)-(3. 8):

   If $\Delta_{max,i} \leq \Delta_{target,i}$

$$[B_i]_{n+1} = [B_i]_n - \Delta B \tag{3.3}$$
$$[H_i]_{n+1} = [H_i]_n - \Delta H \tag{3.4}$$
$$[D_i]_{n+1} = [D_i]_n - \Delta D \tag{3.5}$$

   If $\Delta_{max,i} > \Delta_{target,i}$

$$[B_i]_{n+1} = [B_i]_n + \Delta B \tag{3.6}$$

$$[H_i]_{n+1} = [H_i]_n + \Delta H \tag{3.7}$$
$$[D_i]_{n+1} = [D_i]_n + \Delta D \tag{3.8}$$

where $\Delta_{target,i}$ is the pre-decided target drift value of i^{th} storey corresponding to IO (i.e. 1%); $[D_i]_n$ represents dimensions of columns of each storey (i denotes i^{th} storey) at n^{th} iteration; $[B_i]_n$ and $[H_i]_n$ are beams widths and heights at i^{th} storey in n^{th} iterative step, respectively; ΔD, ΔB and ΔH denote small dimension step changes in columns, beams widths and beams heights, respectively. Based on practical considerations, the cross-section dimension changes are set at 50 mm. It should be noted that the reinforcement ratio of each beam and column element is initially designed and kept unchanged during the entire element size optimisation when the structure behaves nearly elastically.

4. The coefficient of variation (COV) of inter-storey drifts (calculated as standard deviation of IDRs divided by the average of IDRs across all storeys) is calculated at each iterative step. Steps 2 and 3 are repeated iteratively until the COV is lower than a given value, maximum IDR in each storey is satisfied with target limit at IO level. It should be noticed that since the optimum design solution is also required to sustain gravity loads, section sizes in lower stories are usually larger than the codified minimum value, and hence it is very unlikely to achieve a very uniform inter-storey drift distribution due to practical applications.

3.3.6. Inelastic Phase: Reinforcement Ratios Optimisation

Once optimum concrete section sizes are obtained at the end of the elastic design optimisation phase, they are used as the initial design in the second phase of optimisation. At this stage, longitudinal reinforcement ratios are used as primary design variables. This can be justified as steel reinforcement plays a dominant role in controlling inelastic responses beyond the occurrence of first yielding and providing structural ductility. In accordance to ASCE/SEI 41-17, (2017), multiple performance objectives including Life Safety (LS) and Collapse Prevention (CP) should be satisfied in the plastic phase, under earthquake excitations with different hazard levels (PGA = 0.4g; 0.65g) as mentioned in Table 3-1. Performance parameters at structural (i.e. inter-storey drift) and element (i.e. plastic hinge rotation) levels are simultaneously considered in the optimisation process at both specific performance levels.

The steps for UDD optimisation in the plastic phase are as follows:

1. Optimum structural design obtained at the end of elastic phase is regarded as initial design in the plastic design optimisation phase.

2. Plastic hinge rotations at both ends of beam and column elements are calculated based on plastic curvatures (k^p) and in accordance with "Modified Gauss-Radau" plastic hinge integration method as suggested by (Scott and Fenves, 2006):

$$\begin{bmatrix} \theta_I \\ \theta_J \end{bmatrix} = \begin{bmatrix} -k^p|_{x=0} \times l_{pI} \\ k^p|_{x=L} \times l_{pJ} \end{bmatrix} \tag{3.9}$$

where: θ_I and θ_J are plastic rotations at ends I and J of an element, respectively; $x = 0$ and $x = L$ describe the locations of integration points (both ends of an element); l_{pI} and l_{pJ} are physical length of plastic hinge near ends *I* and *J*, respectively; $k^p|_{x=0}$ and $k^p|_{x=L}$ are plastic curvatures at both ends of the element. Average maximum plastic rotations under a set of earthquakes in a certain storey can in turn be obtained for column and beam elements using the Equations (3.10)-(3.11):

$$\theta_{max,i,C} = \max\left[\theta_{n=1,I}, \theta_{n=1,J}, \theta_{n=2,I}, \theta_{n=2,J} \dots \theta_{n=N_{col},I}, \theta_{n=N_{col},J}\right] \tag{3.10}$$

$$\theta_{max,i,B} = \max\left[\theta_{n=1,I}, \theta_{n=1,J}, \theta_{n=2,I}, \theta_{n=2,J} \dots \theta_{n=N_{beam},I}, \theta_{n=N_{beam},J}\right] \tag{3.11}$$

where: $\theta_{max,i,C}$ and $\theta_{max,i,B}$ are the peak column plastic hinge rotation and peak beam plastic hinge rotation in i^{th} storey, respectively; N_{col} is total number of columns elements in each storey (i.e. $N_{col} = 4$); N_{beam} is total number of beam elements in each storey (i.e. $N_{beam} = 3$).

3. The performance ratios $PR_{drift,i}$, $PR_{rotation,i,C}$ and $PR_{rotation,i,B}$ are calculated as ratios of deformation demands to corresponding capacity in each storey, column member and beam member, respectively.

$$PR_{drift,i} = \frac{\Delta_{max,i}}{\Delta_{target}} \tag{3.12}$$

$$PR_{rotation,i,C} = \frac{\theta_{max,i,C}}{\theta_{target,i,C}} \tag{3.13}$$

$$PR_{rotation,i,B} = \frac{\theta_{max,i,B}}{\theta_{target,i,B}} \tag{3.14}$$

where: Δ_{target} equals to 2% at LS level and 4% at CP level; $\theta_{target,i,C}$ and $\theta_{target,i,B}$ are plastic rotation capacity of column and beam elements, respectively, determined in accordance with ASCE/SEI 41-06, (2007).

Multi performance levels (here LS and CP) are concurrently considered, and design variables are remodified based on the most critical performance ratio ($PR^{critical}$) chosen as the largest value from the ratios relating to all specific performance levels:

$$PR^{critical}_{drift,i} = \max\left[PR^{K=I}_{drift,i}, PR^{K=II}_{drift,i}, \dots PR^{K=n}_{drift,i}\right] \tag{3.15}$$

$$PR^{critical}_{rotation,i,C} = \max\left[PR^{K=I}_{rotation,i,C}, PR^{K=II}_{rotation,i,C}, \dots PR^{K=n}_{rotation,i,C}\right] \tag{3.16}$$

$$PR^{critical}_{rotation,i,B} = \max\left[PR^{K=I}_{rotation,i,B}, PR^{K=II}_{rotation,i,B}, \dots PR^{K=n}_{rotation,i,B}\right] \tag{3.17}$$

where: K = I, II, n denotes pre-decided performance objectives (here I = LS performance level under DBE, II = CP performance level under MCE); $PR_{drift,i}^{critical}$ is the critical drift performance ratio considering drift responses in i^{th} storey level; and $PR_{rotation,i,C}^{critical}$, $PR_{rotation,i,B}^{critical}$ are critical performance ratios considering rotations in columns and beams, respectively.

The proposed method considers plastic hinge rotation as the main performance parameter, while IDR is only accounted in UDD optimisation when the target IDR is violated (i.e. $PR_{drift,i}^{critical}>1$). To provide more practical design solutions, it is assumed that in each storey, both interior and exterior columns have the same reinforcement ratio. Furthermore, in the selected models, all beam elements in one storey were designed to have similar reinforcement detailing as the span lengths are identical. The longitudinal reinforcement ratios of beams and columns in each storey are modified using the Equations (3. 18) - (3. 21):

If $PR_{drift,i}^{critical} > 1$ and $max\ [PR_{rotation,i,C}^{critical}, PR_{rotation,i,B}^{critical}] < 1$:

$$[(\rho_C)_i]_{n+1} = [(\rho_C)_i]_n \times (PR_{drift,i}^{critical})^{\beta} \times (1-(PR_{rotation,i,C}^{critical})^2)^{(-\beta)} \tag{3.18}$$

$$[(\rho_B)_i]_{n+1} = [(\rho_B)_i]_n \times (PR_{drift,i}^{critical})^{\beta} \times (1-(PR_{rotation,i,B}^{critical})^2)^{(-\beta)} \tag{3.19}$$

In the other conditions:

$$[(\rho_C)_i]_{n+1} = [(\rho_C)_i]_n \times (PR_{rotation,i,C}^{critical})^{(\alpha)} \tag{3.20}$$

$$[(\rho_B)_i]_{n+1} = [(\rho_B)_i]_n \times (PR_{rotation,i,B}^{critical})^{(\alpha)} \tag{3.21}$$

where: $[(\rho_C)_i]_n$ and $[(\rho_B)_i]_n$ represent reinforcement ratios in columns and beam in i^{th} storey at n^{th} iteration, respectively; β is a convergence parameter in the UDD formula which involves both local and global performance parameters, while α is a convergence parameter that controls optimisation speed when only one design parameter is considered. Based on results on the effect of convergence parameters on optimum solutions in the following section, α was considered to be equal to 0.2, while β was assumed to be half of α.

4. When the modified reinforcement is below the minimum allowable values, the minimum values are used. However, if the reinforcement ratio exceeds the maximum allowable value, the element size (*D, B* and *H*) in i^{th} storey at $(n+1)^{th}$ iteration is incrementally increased using the following Equations (3. 22)-(3. 25):

   If reinforcement ratio in a column reaches the maximum allowable value:

$$[D_i]_{n+1} = [D_i]_n + \Delta D \tag{3.22}$$

   If tension or compression reinforcement ratio in a beam reaches the upper limit:

$$[B_i]_{n+1} = [B_i]_n + \Delta B \tag{3.23}$$

$$[H_i]_{n+1} = [H_i]_n + \Delta H \tag{3.24}$$

$$[D_i]_{n+1} = [D_i]_n + \Delta D \tag{3.25}$$

where: D and ΔD are dimensions of the column cross section and corresponding dimension increment, respectively; B and ΔB are beam widths and corresponding size increment, respectively; H and ΔH are beam height and corresponding size increment, respectively. ΔD, ΔB and ΔH are all taken as 50 mm to provide practical solutions and avoid a sudden increase in structural stiffness.

5. The COV(%) of IDR and plastic hinge rotations in each storey are calculated at both LS and CP performance levels. The UDD algorithm iterates from step 2 until the following conditions are met: (i) the calculated COVs are decreased to an acceptable value (e.g. less than 0.2), (ii) the given performance parameters in each storey are fully satisfied with the target limits corresponding to both LS and CP levels, and their changes remain small at a few subsequent iterations. As discussed before, it is also checked that the final optimum design can sustain the gravity loads.

3.4. Modelling and assumptions

3.4.1. Reference reinforced concrete frames

To assess the efficiency of the proposed optimisation framework, four different 3-bay regular RC frames with 3-, 5-, 10- and 15-storey were selected, with a uniform height of 3 m as shown in Figure 3-1. The buildings were selected to represent typical residential buildings, covering both medium and high-rise buildings, in high seismic regions. They were considered with importance class I and medium ductility class (DCM). The seismic loads were calculated using the Eurocode 8 design response spectrum for medium seismic regions (peak ground acceleration (PGA) 0.4g). 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 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 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 (CEN, 2004a, 2004b).

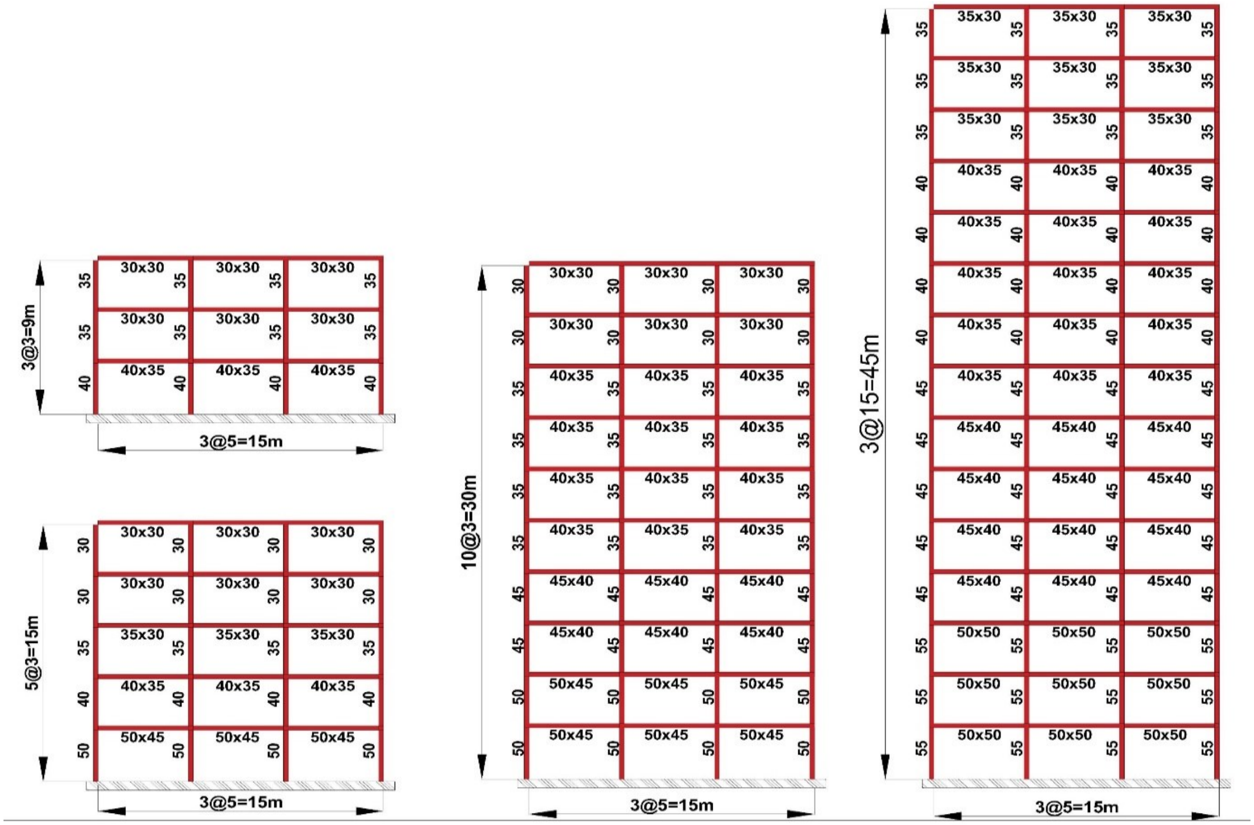

Figure 3-1: Geometry and dimensions of beam and column members of 3-, 5-, 10- and 15-storey RC frames (Beams: "height × width"; Columns: "square dimension")

The frames were modelled and analysed using the finite element software OpenSees (McKenna et al., 2006). "Concrete02" model was utilised to express the material properties of concrete, considering stress-strain relationships of both confined and unconfined concrete by taking into account tension cracking and compressive crushing failure mechanisms (Mohd Yassin, 1994). "Steel02" (or Giuffre-Menegotto-Pinto) model was considered to simulate the bilinear stress-strain relationship of reinforcement steel (Filippou et al., 1983). Beam and column elements were modelled using "distributed-plasticity models", in which the occurrence of nonlinearity is allowed at any location within a specific range of the element (plastic hinge region) instead of concentrated at both ends of an elastic element (Neuenhofer and Filippou, 1997). The nonlinear behaviour was analysed using "force-based" nonlinear finite element models ("*forceBeamColumn*"). To obtain more accurate structural inelastic responses (i.e. plastic hinge rotations), the "Modified Gauss-Radau" integration method derived from Gauss-Radau quadrature rule was used. Two integration points are located at the two ends of the element, where the bending moment is largest in the absence of member loads, and four integration points along the element length (six integration points in total). The method integrates the deformation over the estimated plastic region using a force-based flexibility formulation, and the element deformation is calculated as the sum of deformations within two plastic hinge regions and one interior section (Scott and Fenves, 2006). In this study, the physical length of the plastic hinge region (l_{pl}) was updated in each iterative step using the following formula from Eurocode 8, part 3 (CEN, 2005).

$$l_{pl} = \frac{L_v}{30} + 0.17h + 0.24\frac{d_{bL} f_y\ (MPa)}{\sqrt{f_c}\ (MPa)} \tag{3.26}$$

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

The above formulation indicates that the plastic hinge length depends on the variation of stiffness (cross-section size, diameter of reinforcement) and material properties. It should be noted that these material and element models are extensively used, as they simulate well the nonlinear behaviour of RC structures under seismic and cyclic lateral loads (Calledda et al., 2021; Attarchian et al., 2014).

Rayleigh damping with a constant ratio of 5% was assigned to the first mode and any mode whose cumulative mass participation exceeds 95%. P-Delta effects were also considered for both the design and analyses of the frames. Soil-structure interaction (SSI) effects were not taken into account. To consider the effect of concrete cracking on overall element stiffness, the effective flexural and shear stiffnesses of beam and column elements were taken as half of the gross section stiffness values, as recommended by Eurocode 8 (CEN, 2004a).

It is important to note that, the frames referenced in this study are assumed to be modelling in 2D, mainly to capture their translational behaviour under earthquakes. When implementing 3D modelling in the case study, additional structural behaviour, such as torsional effects, shear deformation and out-of-plane bending, should also be considered in deciding performance parameters. Meanwhile, in the optimisation process, more design variables, such as shear reinforcement ratio, should be carefully modified in a 3D RC building. As a result, the computational costs generally increase with 3D modelling.

3.4.2. Selected earthquake records and code-based design spectrum

In this study, a set of six seismic ground motion records fully compatible with the target spectrum were synthesised using target acceleration spectra compatible time histories (TARSCTHS) (Papageorgiou et al., 2002). The elastic response spectrum of each of the generated records as well as their mean response spectrum are compared with the Eurocode 8-based elastic design response spectrum in Figure 3-2. It can be seen that the mean response spectrum of the artificial records provides a close approximation to the Eurocode 8 design spectrum within a wide range of periods that cover the fundamental periods of the four selected RC frames. Therefore, the selected artificial earthquakes can be considered as suitable representatives of the chosen design spectrum. For different hazards levels, the generated records are simply scaled to reach the target PGA level.

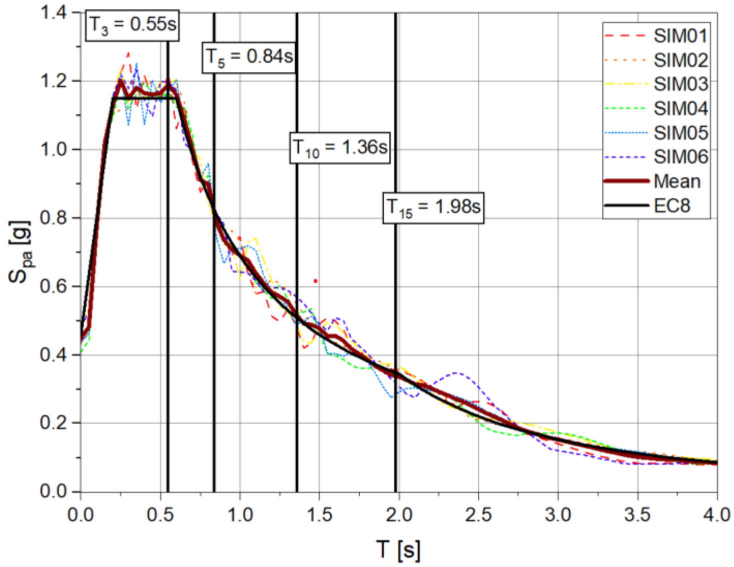

Figure 3-2: Eurocode 8 design response spectrum and acceleration spectra of artificial records

3.5. Optimum design for the design earthquakes

3.5.1. Seismic performance assessment

For each referenced frame, the average seismic responses (i.e. IDR and plastic rotations ratios) under all six records are calculated and compared between the optimum solution (named as "optimum design") and the initial design codified by Eurocode 8 (named as "initial design").

Figure 3-3 compares the height-wise distributions of maximum inter-storey drift ratios (Δ_{max}) for the 3-, 5-, 10- and 15-storey RC frames at IO, LS, CP performance levels. Compared to the initial frames, the frames optimised based on the concept of UDD exhibited more uniform inter-storey drift distribution, and less concentrated maximum inter-storey drifts, while they closely approached the performance targets at each specific seismic hazard level. The optimum designs also reduce global damage, as details will be explained in the following sections.

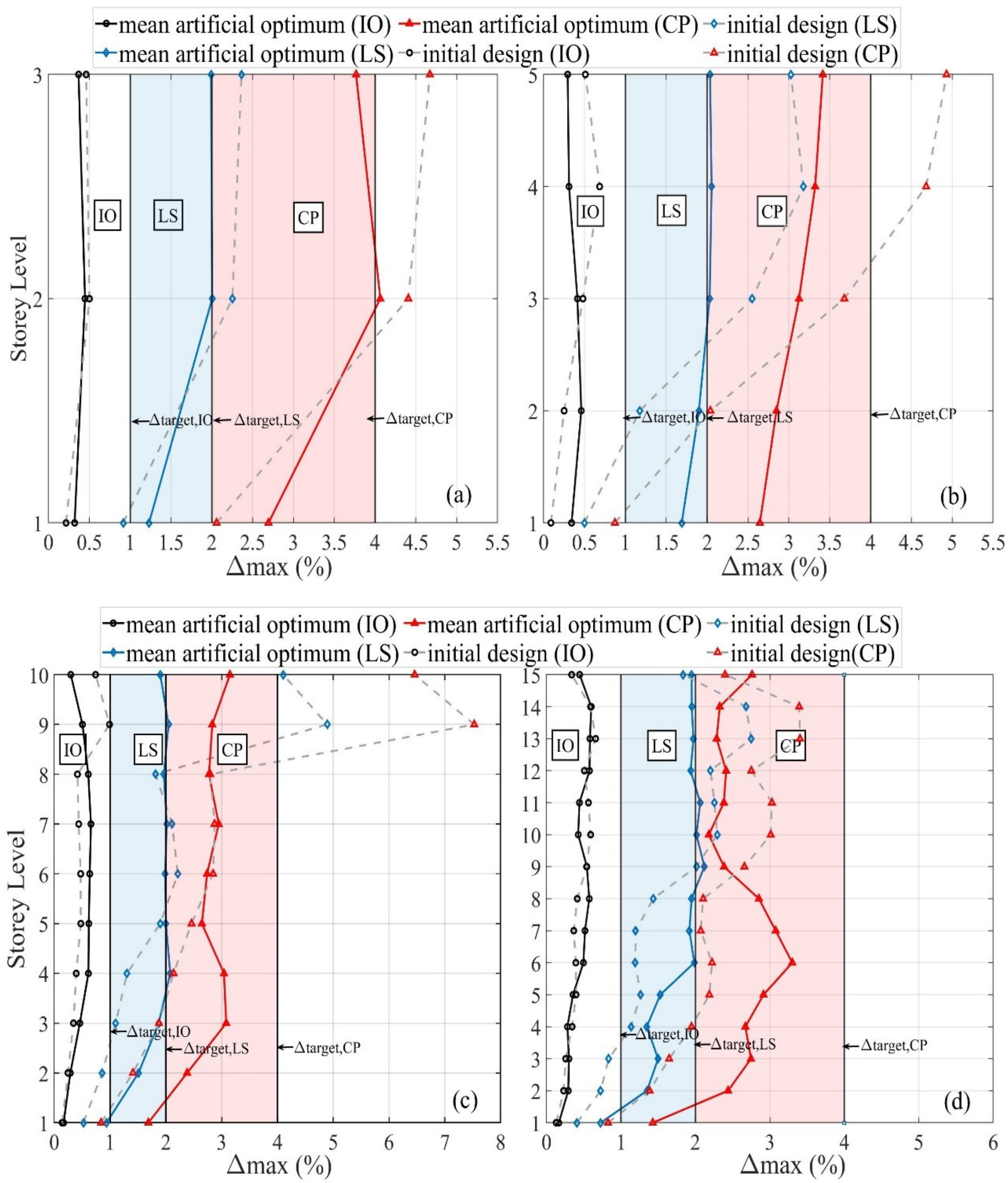

Figure 3-3: Height-wise distribution of Δ_{max} for optimum and initial design solutions for (a) 3-, (b) 5-, (c) 10- and (d) 15-storey frames, average results under six artificial records at IO, LS, CP performance levels

Figures 3-4, 3-5, 3-6 and 3-7 illustrate the height-wise distributions of maximum plastic rotation ratios in columns ($\theta_{max,C}/\theta_{target,C}$) for 3-, 5-, 10- and 15-storey optimum and initial designs, respectively. The results represent the average values under the selected six artificial records corresponding to LS and CP performance levels subjected to DBE and MCE records, respectively. It can be seen that the optimum solutions generally experienced less maximum plastic rotation ratios and localised damage concentration. It should be noted that in some cases the optimum designs exhibited larger plastic rotation ratios than their initial design counterparts (e.g. first storey in 3-storey frame, and first three storeys in 15-storey frame). This is because the optimum solutions use smaller column cross sections in these storeys to achieve a more efficient use of material capacity. This leads to higher axial load ratios in these elements, which also affects their rotational capacity.

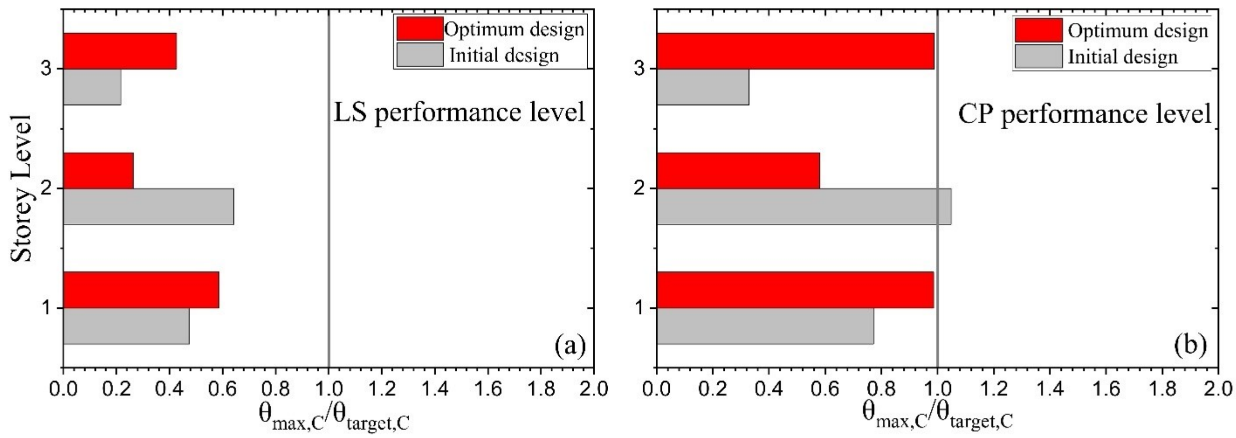

Figure 3-4: Height-wise distribution of $\theta_{max,C}$ to $\theta_{target,C}$ ratios for optimum and initial design 3-storey frame, average results under six artificial records at (a) LS and (b) CP performance levels

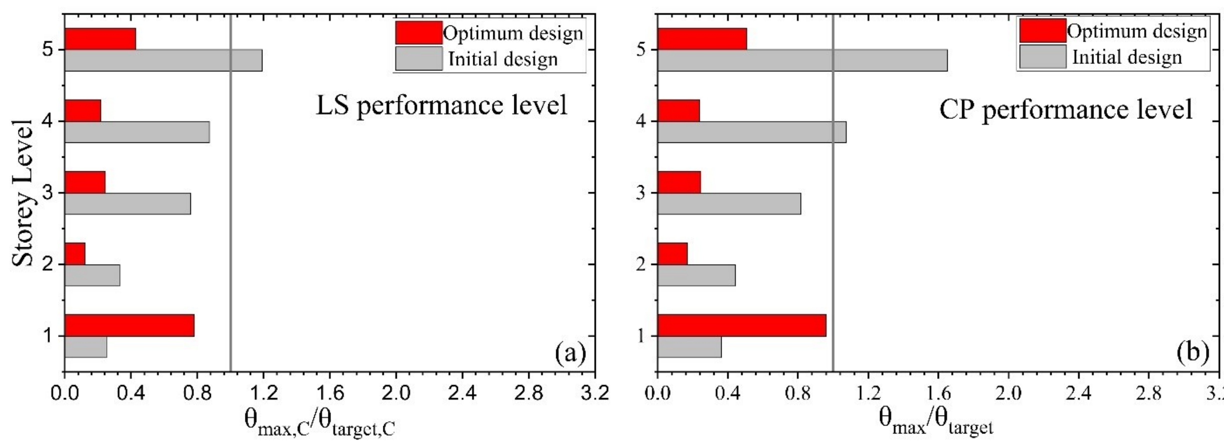

Figure 3-5: Height-wise distribution of $\theta_{max,C}$ to $\theta_{target,C}$ ratios for optimum and initial design 5-storey frame, average results under six artificial records at (a) LS and (b) CP performance levels

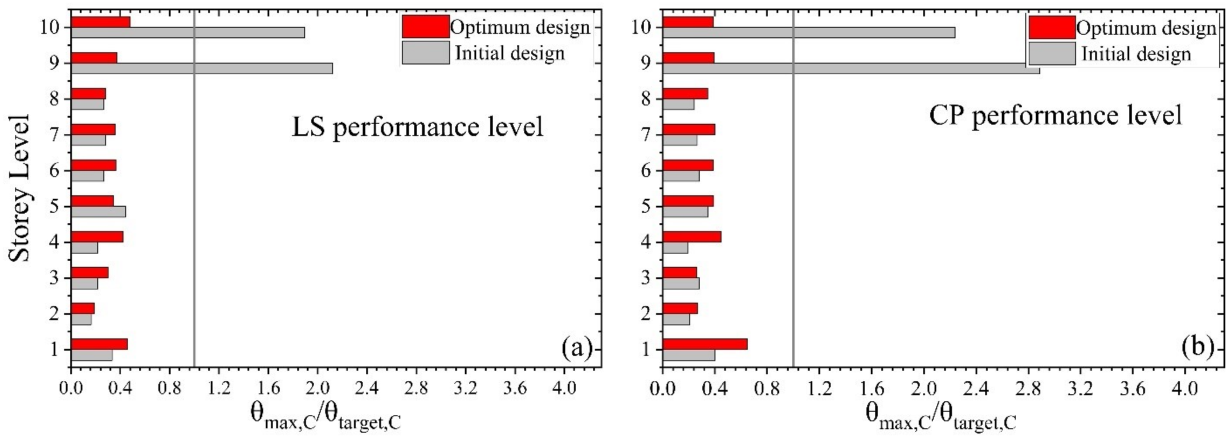

Figure 3-6: Height-wise distribution of $\theta_{max,C}$ to $\theta_{target,C}$ ratios for optimum and initial design 10-storey frame, average results under six artificial records at (a) LS and (b) CP performance levels

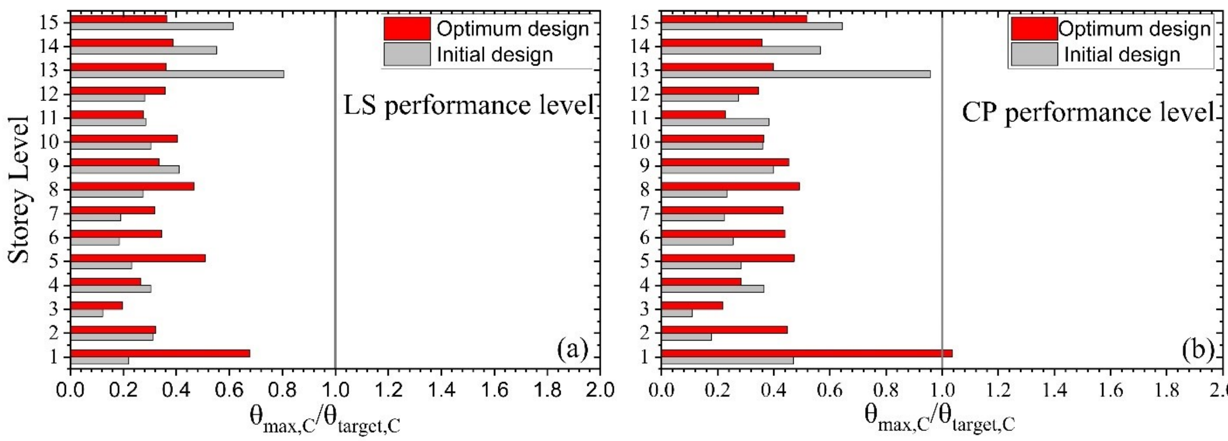

Figure 3-7: Height-wise distribution of $\theta_{max,C}$ to $\theta_{target,C}$ ratios for optimum and initial design 15-storey frame, average results under six artificial records at (a) LS and (b) CP performance levels

Tables 3-4, 3-5 and 3-6 present the average maximum responses in terms of inter-storey drift, and plastic rotation in columns and beams, respectively, for optimum and initial designs at different performance levels. The results demonstrate that the proposed optimum design framework reduces maximum inter-storey drift ratios up to 15%, 36%, 58% and 23% for 3-, 5-, 10- and 15-storey frames, respectively. When structures are subjected to DBE and MCE earthquake records, optimum design solutions also result in significant reductions in maximum plastic rotation ratios in beams and columns up to 13%, 42%, 78% and 20% for 3-, 5-, 10- and 15-storey frames, respectively. It can be concluded that, compared to the initial designs, the proposed optimisation technique is helpful in improving the structural safety of all the selected frames at IO, LS and CP performance levels by reducing localised damage and preventing soft storey failures. In optimum design solutions, material capacities in most of storeys are fully exploited, which in turn leads to a more uniform performance distribution aligned with the concept of UDD.

Table 3-4: Maximum Δ_{max} (%) of 3-, 5-, 10- and 15-storey frame with optimal and initial design solutions

Performance Level	Immediate Occupancy			Life Safety			Collapse Prevention		
RC frames	Optimum	Initial	Gain	Optimum	Initial	Gain	Optimum	Initial	Gain
3-storey	0.45	0.50	10.7%	2.00	2.36	15.4%	4.04	4.68	13.2%
5-storey	0.46	0.68	32.8%	2.04	3.18	35.6%	3.40	4.96	31.2%
10-storey	0.65	1.01	35.3%	2.08	4.88	57.5%	3.16	7.48	57.9%
15-storey	0.60	0.66	9.0%	2.12	2.74	22.9%	3.28	3.30	3.1%

Table 3-5: Maximum Plastic Rotations Demand to Capacity Ratios in Columns of 3-, 5-, 10- and 15-storey frame with optimum and initial design solutions

Performance Level	Life Safety			Collapse Prevention		
RC frames	Optimum	Initial	Gain	Optimum	Initial	Gain
3-storey	0.59	0.64	8.6%	0.99	1.05	5.8%
5-storey	0.78	1.19	34.4%	0.96	1.65	42.0%
10-storey	0.46	2.12	78.2%	0.66	2.89	77.2%
15-storey	0.68	0.80	16.0%	1.04	0.96	-8.4%

Table 3-6: Maximum Plastic Rotations Demand to Capacity Ratios in Beams of 3-, 5-, 10- and 15-storey frame with optimum and initial design solutions

Performance Level	Life Safety			Collapse Prevention		
RC frames	Optimum	Initial	Gain	Optimum	Initial	Gain
3-storey	0.80	0.90	11.3%	0.87	0.99	12.6%
5-storey	0.68	1.04	34.9%	0.64	0.90	29.3%
10-storey	0.54	1.10	50.9%	0.40	1.05	61.7%
15-storey	0.52	0.66	20.4%	0.42	0.44	4.8%

As an example, Table 3-7 presents the details on section dimensions of the 10-storey RC frame for both optimum and initial designs. In the optimum design, the element cross section dimensions were only increased in the top storeys, while the dimensions of most sections were reduced to achieve more efficient material usage. The difference in total volume of concrete used in the beams and columns in the 10-storey frames is found to be 20%.

Table 3-7: Initial and Optimum Member Sizes (unit: mm) of 10-storey RC frame

Element	Storey No.	Dimension (Initial)	Dimension (Optimum)	Element	Storey No.	Width (Initial)	Depth (Initial)	Width (Optimum)	Depth (Optimum)
Column	10	300	350	*Beam*	10	300	300	350	300
	9	300	350		9	300	300	350	300
	8	400	350		8	400	350	300	300
	7	400	350		7	400	350	350	300
	6	400	350		6	400	350	350	300
	5	400	350		5	400	350	350	300
	4	450	350		4	450	400	350	300
	3	450	400		3	450	400	400	350
	2	500	450		2	500	450	450	400
	1	500	450		1	500	450	450	400
Total Volume		Initial 43.95 (m^3)					Optimum 35.37 (m^3)		

In general, the optimum design solutions required considerably lower reinforcement ratios to satisfy the selected performance target. As an example, Table 3-8 presents the longitudinal steel reinforcement design details of beams and columns of the 5-storey RC frame before and after optimisation. It can be noted that in the optimum design the ratios of columns and beams in several storeys tend to the minimum allowable limits in Eurocode 8 so that material can be more efficiently used. The dimensions of such sections cannot be reduced otherwise the IO design targets will be violated. The difference in total weight of reinforcement steels used in the beams and columns in the 5-storey frames is found to be 43%. More information about cross-section dimensions and longitudinal reinforcement ratio of initial and optimum design solutions for 3-, 5-, 10- and 15-storey RC frames are presented in the tables in Appendix B and C.

Table 3-8: Initial and Optimum Component Reinforcement Ratios (in %) of 5-storey RC frame

Storey No.	Element	Rein. (Initial)	Rein. (optimum)	Element	Top rein. (Initial)	Bottom rein. (Initial)	Top rein. (optimum)	Bottom rein. (Optimum)
5	*Column*	2.68%	2.13%	*Beam*	1.12%	0.67%	0.72%	0.43%
4		2.68%	1.01%		1.12%	0.67%	0.35%	0.35%
3		1.97%	1.00%		0.96%	0.57%	0.33%	0.33%
2		2.36%	1.00%		0.86%	0.57%	0.33%	0.33%
1		2.01%	1.02%		0.36%	0.27%	0.33%	0.33%
Total		Initial				Optimum		

Weight	2480.6 (kg)	1428.7 (kg)

3.5.2. Global damage index

Previous studies indicated that structural seismic demand parameter based on a single maximum value (i.e. maximum inter-storey drift ratios) may not always accurately predict the damage state of a structure. This is particularly evident when considering the effects of structural energy dissipation through large plastic deformations and corresponding capacity of the structure under a number of cycle forces (Park and Ang, 1985; Kappos, 1997). To investigate the efficiency of the proposed optimisation method on reducing overall structural damage during an earthquake event, this study quantifies the damage by using the damage index which is evaluated as a function of displacement ductility and dissipated energy. To achieve this, the structural damage index in the i^{th} storey (D_i) is first estimated, using the "demand versus capacity" concept as suggested by (Powell and Allahabadi, 1988):

$$D_i = \left(\frac{\delta_c - \delta_t}{\delta_u - \delta_t}\right)^b \tag{3.27}$$

where δ_c , δ_t and δ_u are the calculated, threshold and ultimate values of specific damage parameter, respectively. Constant parameter b is determined based on experimental data, which is suggested as 1.5 for reinforced concrete frames (Cosenza and Manfredi, 2000).

In this study, the displacement-based ductility ratio (μ) is considered as the damage parameter to evaluate the structural ability to deform within the inelastic range before failure. The maximum ductility ratio (μ_{max}) can be obtained using the Equation (3. 28):

$$\mu_{max} = \frac{\Delta_{max,i}}{\Delta_{yield,i}} \tag{3.28}$$

And the corresponding ultimate ductility ($\mu_{ultimate}$) is calculated as:

$$\mu_{ultimate} = \frac{\Delta_{ultimate,i}}{\Delta_{yield,i}} \tag{3.29}$$

where $\Delta_{max,i}$, $\Delta_{yield,i}$ and $\Delta_{ultimate,i}$ represent peak inter-storey drift, yielding drift and ultimate drift capacity in i^{th} storey, respectively. $\Delta_{yield,i}$ can be estimated through pushover analysis by applying a monotonically increasing lateral load and adopting a bilinear representation of the capacity diagram based on equal energy principle as suggested by ASCE/SEI 41-13, (2014). In this study, $\Delta_{ultimate,i}$ is assumed to be the target limiting value at CP performance level in accordance with ASCE/SEI 41-06, (2007), and the value is for structural failure is assumed to occur. Using the defined ductility parameters, Equation (3. 27) can be written as follows to calculate the storey damage index corresponding to i^{th} storey (D_i):

$$D_i = \left(\frac{\Delta_{max,i} - \Delta_{yield,i}}{\Delta_{ultimate,i} - \Delta_{yield,i}} \right)^b \tag{3.30}$$

This damage index formula assumes that the largest damage in each storey is expected to happen where inter-storey drift ratio at i^{th} level reaches the maximum value during the earthquake.

Subsequently, the global damage index D_g is defined as weighted average of damage index D_i at individual storey levels, with weights (w_i) for i^{th} storey represented by dissipated energy:

$$D_g = \frac{\sum_{i=1}^{N} D_i w_i}{\sum_{i=1}^{N} w_i} \tag{3.31}$$

In the above equation, N represents the total number of storeys. Here it is assumed that the amount of energy dissipation corresponding in each storey is proportional to its damage index at i^{th} storey level (D_i), as suggest by previous studies (Nabid et al., 2018; De Domenico and Hajirasouliha, 2021). Thus, the global damage index D_g in Equation (3.31) can be simplified as:

$$D_g = \frac{\sum_{i=1}^{N} D_i^2}{\sum_{i=1}^{N} D_i} \tag{3.32}$$

The value of the D_g ranges from 0 (undamaged) to 1 (completely damaged).

Figure 3-8 shows the mean global damage indices of 3, 5, 10 and 15-storey RC optimum design frames compared to their code-based design counterparts under the six selected earthquakes. The results indicate that the UDD optimisation led to less overall damage at both LS and CP performance levels. It is shown that, compared to the initial frames, the optimum design solutions experience less damage up to 64%, 51%, 88% and 52% for 3, 5, 10 and 15-storey frames, respectively. This is because the proposed optimisation methodology significantly reduces maximum inter-storey drift ratios and prevents “soft storey” failures in the storeys where the response violates the drift limits. Indeed, the drift profiles of all optimum frames tend to be close to the target limits at all specific performance levels in most storeys. Although this cannot be completely achieved due to the influence of axial loads on columns in lower storeys, structural materials at most storey levels are efficiently utilised, which results in a better seismic performance and hence a lower global damage index.

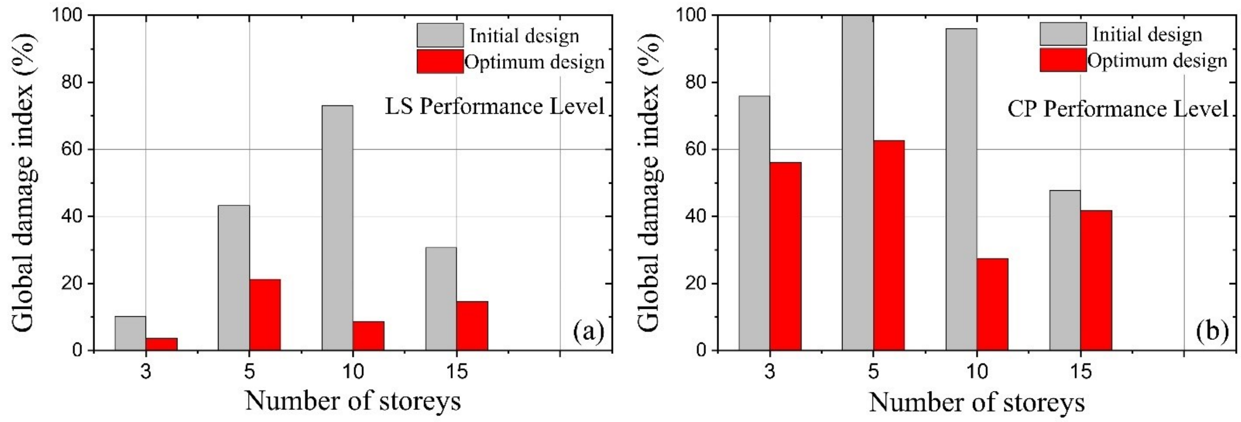

Figure 3-8: Global damage index for 3, 5, 10 and 15-storeyy RC frames at (a) LS and (b) CP performance levels, average results under six artificial earthquakes

3.6. Sensitivity analysis

3.6.1. Effect of convergence parameter

In a nonlinear system, previous studies indicate that changes in the design variables during the iterative process should be made gradually to avoid divergence (De Domenico and Hajirasouliha, 2021; Nabid et al., 2019, 2017). In the proposed UDD optimisation method, the convergence speed is governed by the value of convergence parameter α. A small value of α increases the chance of convergence, but at the expense of increasing computational costs, whilst a larger α reduces computational cost, but may generate significant fluctuations and divergence. This highlights the importance of selecting a suitable value of convergence parameter by considering a balance between computational efficiency and convergence. Figures 3-9 and 3-10 show the variation of maximum inter-storey drift ratios (Δ_{max}) and maximum plastic rotation ratios ($\theta_{max,C}$) of the 5-storey frame versus the iterative steps, for α values equal to 0.005, 0.02 and 0.07, respectively. The results are shown for a single spectrum-compatible artificial earthquake (SIM01) and corresponding to DBE level, but a similar trend was observed for the other earthquake records. It should be noted that the sensitivity analysis started with the optimum solution obtained at the end of the elastic design optimisation.

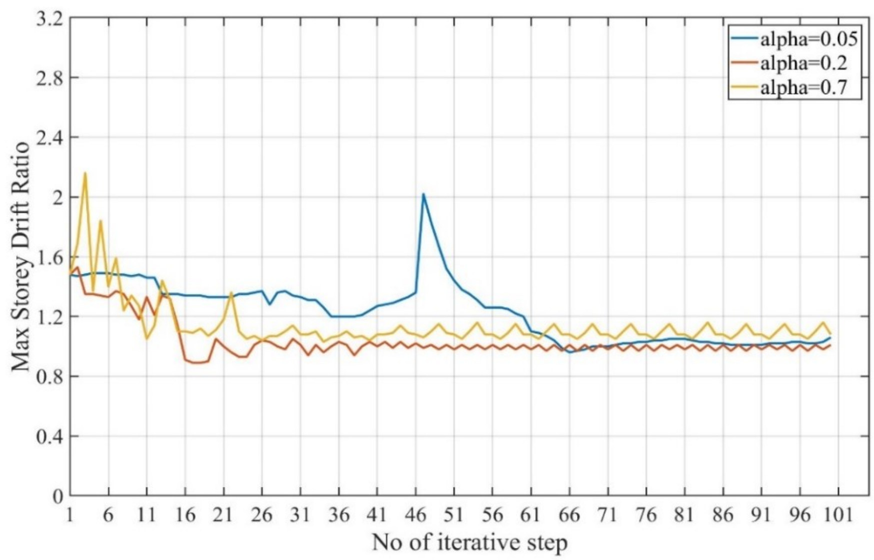

Figure 3-9: Maximum Δ_{max} as iterations processed of 5-storey RC frame under, SIM01

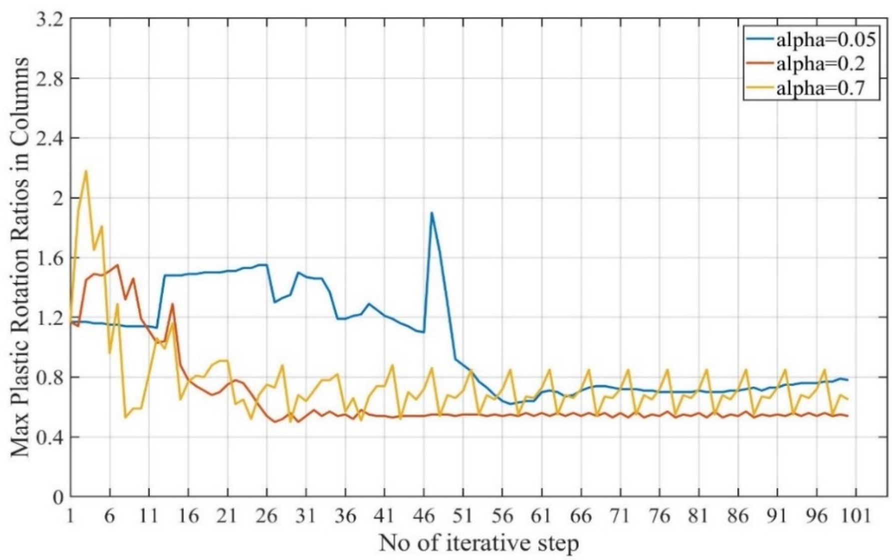

Figure 3-10: Maximum $\theta_{max,C}$ as iterations processed of 5-storey RC frame under, SIM01

The results show that the convergence speed is very slow when the convergence parameter α is 0.05. On the other hand, using α equals to 0.7 leads to divergence, which is especially evident in the case of maximum plastic rotations. The α with value of 0.2 provided steady convergence without any major fluctuations, and the final design was practically obtained in less than 40 steps.

It should be noted that Hajirasouliha et al. (2012) also suggested using convergence values between 0.1 and 0.2 for single level performance-based optimisation of RC frames. In another study, Nabid et al. (2018) demonstrated that reasonable convergent solutions of RC frames with friction dampers can be obtained when the convergence parameter ranges from 0.2 to 0.5.

Therefore, the convergence parameter α equal to 0.2 was used for the optimisation in the plastic phase in this study.

3.6.2. Effect of earthquake record variability on various design approaches

Earthquakes are random excitations in nature and their frequency contents and amplitudes in future events cannot be accurately predicted. Consequently, the optimum design solution may be affected by uncertainty associated with different characteristics of the design earthquakes and may change due to the variability of the earthquakes. This section investigates how the uncertainty in the selection of earthquake records affects the design optimisation. Three alternative design approaches are examined: (i), Initial design obtained by following Eurocode 8 regulations (named as “Initial design” in previous sections); (ii), Optimum solution obtained based on the average responses under six chosen earthquake records (named as “Optimum design” in previous sections), (iii), Optimum solutions obtained based on a randomly selected single spectrum-compatible earthquake record, here the optimisation is processed individually under SIM01 and SIM03, respectively (named as “SIM01 optimum” and “SIM03 optimum”). In the third alterative design approaches, the same multi-level optimisation methodology was applied but under a single chosen earthquake record. Nonetheless, for comparison purposes, the performance of the three design approaches is assessed using all six spectrum-compatible artificial earthquakes.

3.6.2.1. Structural Performance of three design approaches

Figures 3-11 - 3-14 show the effect of earthquake record variability on height-wise distributions of inter-storey drifts at IO, LS and CP performance level for all the reference frames. Figures 3-15 and 3-16 compare, respectively, the maximum plastic rotation ratio and global damage index of 3-, 5-, 10- and 15-storey RC frames designed based on different approaches. It should be noted that the result presented in the Figure 3-15 is the maximum plastic rotation ratio among all the columns in each of the selected frames. The errors bars in each histogram indicate the corresponding standard deviations of responses under the six artificial earthquake records.

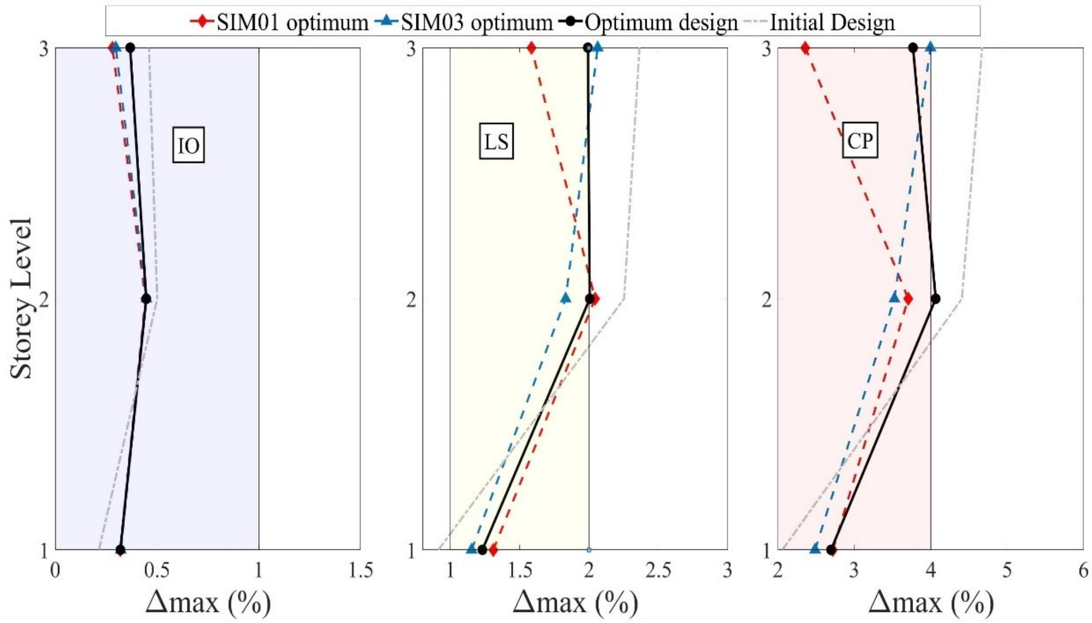

Figure 3-11: Efficiency of the selected design approach in terms of average height-wise distribution of Δ_{max} under six artificial earthquakes at IO, LS and CP levels, 3-storey frame

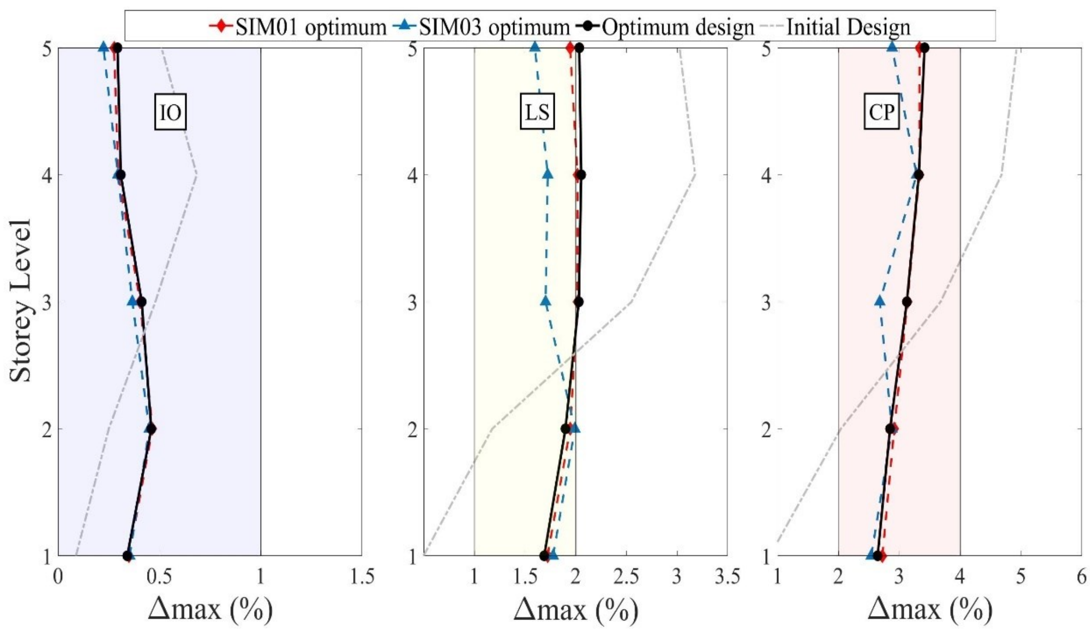

Figure 3-12: Efficiency of the selected design approach in terms of average height-wise distribution of Δ_{max} under six artificial earthquakes at IO, LS and CP levels, 5-storey frame

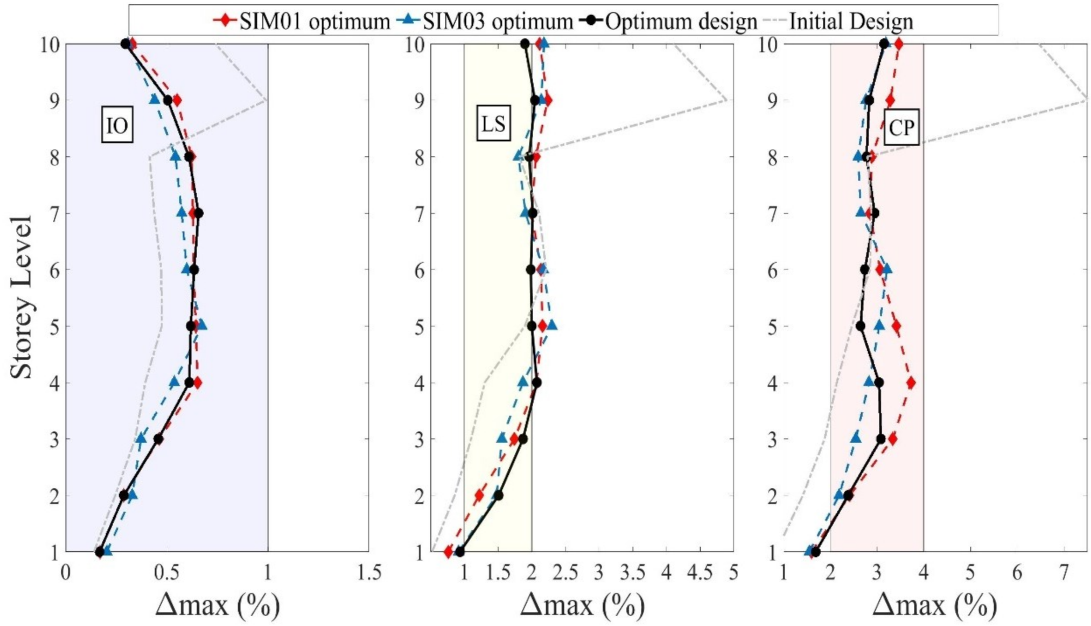

Figure 3-13: Efficiency of the selected design approach in terms of average height-wise distribution of Δ_{max} under six artificial earthquakes at IO, LS and CP levels, 10-storey frame

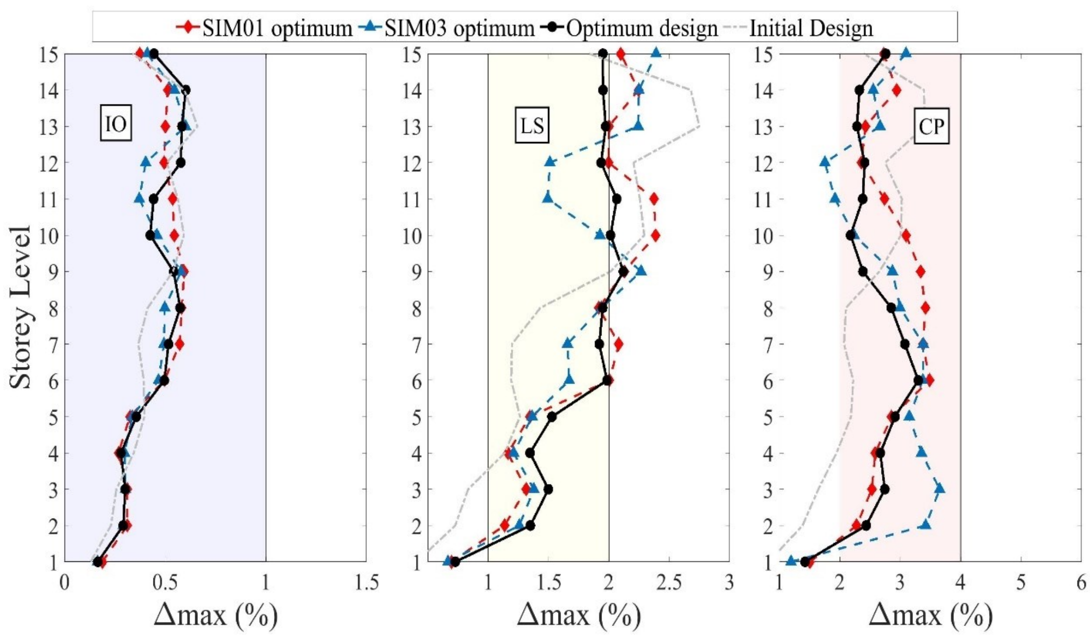

Figure 3-14: Efficiency of the selected design approach in terms of average height-wise distribution of Δ_{max} under six artificial earthquakes at IO, LS and CP levels, 15-storey frame

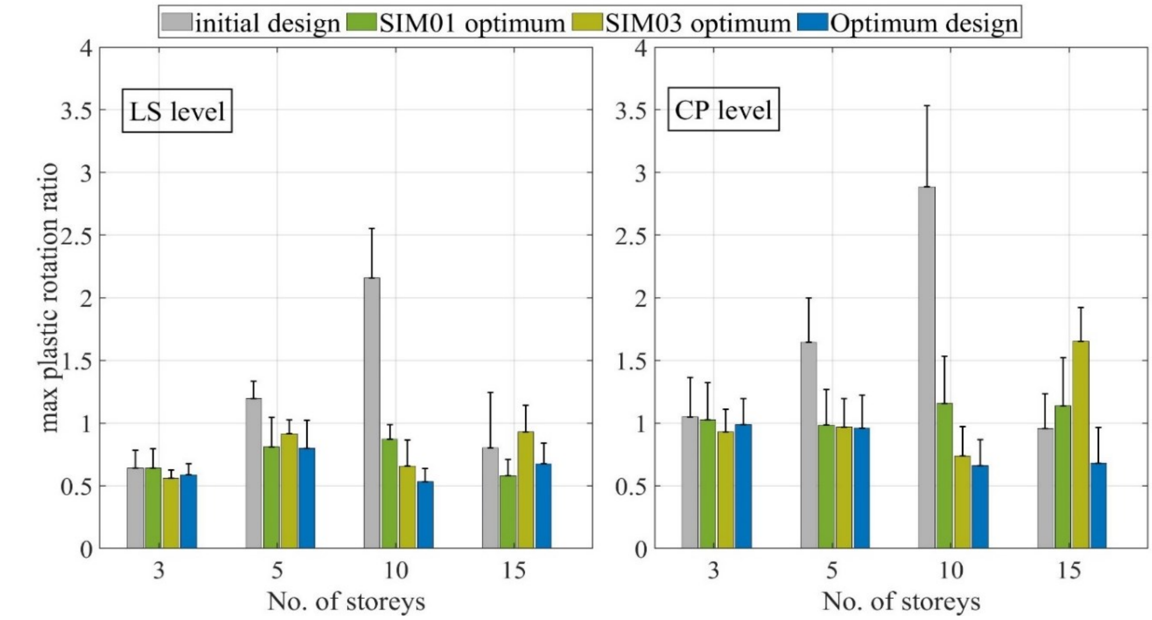

Figure 3-15: Max $\theta_{max,C}/\theta_{target,C}$ of 3-, 5-, 10- and 15-storey frame, average results (plus standard deviation) under six selected artificial records at LS and CP levels

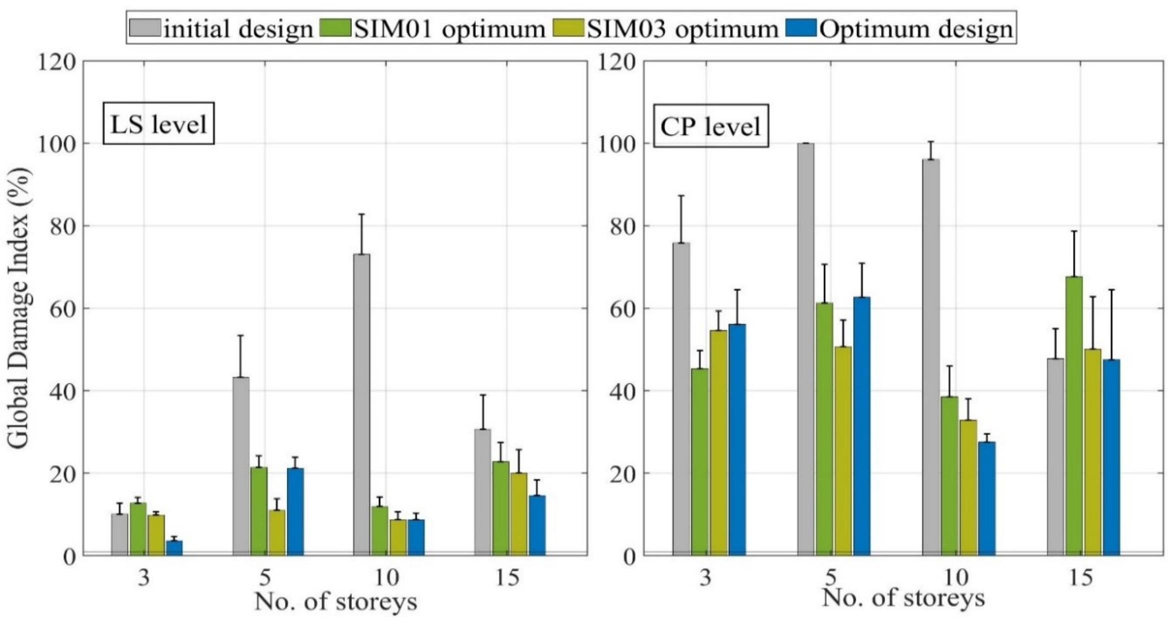

Figure 3-16: Global Damage index (%) of 3-, 5-, 10- and 15-storey frame, average results (plus standard deviation) under six selected artificial records at LS and CP levels

These results demonstrate that, for low to medium-rise buildings (i.e. 3- and 5-storey), the use of single earthquake record (SIM01 or SIM03) can lead to sufficiently accurate seismic responses. In these cases, compared to the code-based initial design, both SIM01 and SIM03 optimum frames exhibited more uniform drift distribution, and satisfied the target PBD limits at all storey levels whilst reducing the local (i.e. maximum plastic rotations) and global damage indexes significantly.

For high-rise buildings (i.e. 10- and 15-storey), the frames designed under both single earthquake records (i.e. SIM01 and SIM03) exhibited discrepancies in the values of Δ_{max} compared to the target limits specified in PBD at LS level (more than 10% difference). These frames also experienced larger plastic rotation ratios (up to 59%) and global damage indexes (up to 36%) than the corresponding optimum design solutions. These differences are especially evident in the case of the 15-storey frame at CP performance level. These results can be justified since higher mode effects in tall buildings are generally more prominent and may lead to discrepancies in structural performance when earthquakes with different characteristics are considered.

3.6.2.2. Material Usage of three design approaches

In this section, the effects of earthquake record variability are investigated in terms of the total material usage. The total concrete volume (m^3) and total reinforcement steel weights (kg) required for 3-, 5-, 10-, and 15-storey optimum designs are compared in Table 3-9, for the different optimisation approaches. It can be seen that, most of the optimum design solutions required less structural materials compared to their code-based initial design counterparts. Furthermore, for the optimisations under single earthquake, the random selection of earthquake record could clearly affect the total materials usage for all the selected RC frames. The results indicate that in the case of low to medium-rise buildings, the optimum solution under a set of earthquakes generally leads to less total material usage, compared to the case that only one input record is used. This is in agreement with the previous observations that by adopting the average response from a chosen set of earthquakes records, the effects of different characteristics of the input design earthquake (i.e. frequency content, amplitude) on the performance assessment can be reduced, and overall, a more reliable design solution is obtained (Hajirasouliha et al., 2012).

It should be noted that previous studies demonstrated that design optimisation using a single earthquake record can lead to an acceptable design for steel frames with nonlinear viscous dampers especially in the case of low-to-medium rise buildings (De Domenico and Hajirasouliha, 2021). However, this study shows that the design optimisation of RC frames based on a randomly selected single earthquake (e.g. SIM01 or SIM03) may lead to less economic solutions. This is because optimisation of RC frame systems is more complex, since, to achieve practical design solutions, it has to deal with discrete optimisation of section sizes and limits in reinforcement ratios. As a result, the solutions are sensitive to the characteristics of input earthquake records, and hence to obtain reliable design solutions for a specific design spectrum, the average performance under a set of spectrum-compatible records is necessary for the optimisation process.

Table 3-9: Total material usage for four alternative design solutions

	Total Concrete Volume (m^3)	Total Reinforcement steel weight (kg)

Design Alternative	Initial design	Optimum design	SIM01 optimum	SIM03 optimum	Initial design	Optimum design	SIM01 optimum	SIM03 optimum
3-Storey	9.66	9.72	10.11	10.11	1318.8	1373.7	1664.2	1397.3
5-Storey	18.30	19.38	19.38	19.77	2480.6	1428.7	1475.8	2464.9
10-Storey	43.95	35.14	35.37	40.01	5809.0	3697.3	3438.3	3297.0
15-Storey	72.40	58.56	58.96	61.68	10063.7	6413.4	5392.9	6264.3

While the presented results are based on the models and assumptions considered in this study, the proposed multi-level optimisation framework is general, and can be adopted for any design based on other seismic design codes and performance criteria used in common practice.

3.7. Summary and Conclusions

In this study, a multi-level performance-based optimisation framework using the concept of Unform Damage Distribution is proposed to minimise structural damage in multi-storey reinforced concrete frames under earthquake events, while minimising material usage. The key performance parameters, including plastic hinge rotations at the element level and inter-storey drift ratios at the structural level, are simultaneously considered in the optimisation procedure. A novel approach is proposed to optimise both the cross-sectional dimensions of elements and steel reinforcement ratios at elastic and plastic phases, respectively, by satisfying multiple performance-based design criteria and practical design constraints. The efficiency of the proposed method was demonstrated by optimising the design of 3-, 5-, 10- and 15-storey RC frames under a set of spectrum-compatible design earthquake records. Compared to Eurocode-based designs, the proposed optimisation framework can directly control structural seismic performances under multiple hazard levels, and in turn providing more resilient solutions with minimum material usages. Meanwhile, the application of non-linear time history analysis in the framework assists to more accurate predictions of the seismic performances. From the results presented in this study, the following conclusions can be drawn:

- The proposed multi-level optimisation framework can effectively control the key local and global structural performance parameters to satisfy multiple performance objectives (i.e. IO, LS and CP) under different earthquake intensity levels ranging from frequent to very rare earthquakes. Compared to the initial code-based design frames, the optimum solutions exhibited lower maximum inter-storey drift ratios and maximum plastic rotation ratios by 58% and 78%, respectively. The optimum structures also, in general, experienced more uniform height-wise distributions of inter-storey drift ratios and plastic rotation ratios, hence preventing local damage.

- In general, the optimum design solutions required considerably less total structural materials by utilising more efficient cross-section sizes and reinforcement ratios. This was particularly evident in the case of 10- and 15- storey frames, where both concrete volumes and total reinforcement weights were reduced by around 20% and 36%, respectively. For the 3-storey, the required structural materials were slightly increased (up to 4%) to satisfy the prescribed performance targets corresponding to multiple hazard levels.

- The magnitudes of global damage index were calculated separately when LS and CP performance levels were considered. The results showed that the 3-, 5-, 10- and 15-storey optimum frames exhibited up to 64%, 51%, 88% and 52% less global damage index compared to the code-based solutions, respectively.

- The effect of different convergence parameters (α) on the efficiency and computational speed leading to the optimum solution was investigated. It is shown that by using the convergence parameter ($\alpha = 0.2$), the optimum answer is generally achieved in less than 40 steps. This highlights the computational efficiency of the proposed method compared to other optimisation techniques such as GA.

- The effect of earthquake record variability on the optimum design was investigated in terms of both seismic performance and total material usage. It is shown that the design approach based on the proposed optimisation methodology using a single spectrum-compatible earthquake may result in less economic designs and less satisfactory seismic performances especially in the case of tall-buildings. Therefore, to obtain the most robust and economic design solution, it is recommended to use optimisation based on the average response of a set of spectrum-compatible records.

CHAPTER 4 : Life-Cycle Cost Efficiency of RC Frames Optimised Using Multi-level Performance-based Methodology

4.1. Abstract

Most conventional seismic design approaches aim to provide design solutions with sufficient strength to primarily ensure "life safety" but cannot directly limit structural damage under earthquake with different intensities, and generally lack provisions for determining whether structural configurations in these designs can be further modified to achieve greater cost efficiency. This study adopts life-cycle cost analysis as an assessment tool to investigate the economic efficiency of RC frames that are designed using a practical performance-based optimisation methodology centred on the concept of Uniform Damage Distribution (UDD). Using an iterative optimisation approach, both local (i.e. plastic hinge rotation) and global (i.e. inter-storey drift) structural responses are controlled to closely approach the predetermined target limits at multiple performance levels. As a result, material capacities for concretes and steel reinforcements in most storeys of optimum designs are fully exploited, minimising the initial construction cost of RC frames. After the optimisation, the expected damage losses are calculated relating to structural damage quantified by seismic performances in a probabilistic manner, considering six damage states and different seismic hazard levels. The total life-cycle costs of the optimum designs are evaluated as the sum of the initial cost and the expected damage losses. To demonstrate the efficiency of the proposed optimisation method, 3-, 5-, 10- and 15-storey RC frames are first optimised employing the proposed UDD optimisation method under a set of artificial earthquake records. The total life-cycle costs are then assessed for their optimum design solutions. The results highlight that, compared to the frames conventionally designed using Eurocode, optimum design solutions: (i) reduce initial construction costs by up to 14.8%, (ii) achieve up to 87.1% less damage costs and 63.6% savings in total life-cycle costs, and (iii) resulted up to 84.5% reductions in global structural damage under a wide range of earthquake intensities. Sensitivity analysis on the selection of different sets of earthquake records

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shows that a group of spectrum-compatible artificial earthquakes is better than natural earthquake records. The proposed optimisation can be used in practical seismic design of RC frames and can lead to savings not only in initial costs but, more importantly, also in expected total life-cycle costs.

Keywords: Multi-level optimisation framework; Reinforced concrete frames, Performance-based seismic design; Life-cycle cost analysis; Incremental dynamic analysis

4.2. Introduction

Conventional seismic design approaches, as adopted in most contemporary design guidelines (e.g. Eurocode 8 (CEN, 2004a), IBC 2021 (ICC., 2020), Chinese code GB 50011 (National Standard of the People's Republic of China, 2010)), mainly focus on providing design solutions with adequate strength and ductility in structural elements to satisfy life safety requirements, and ensuring the designs have sufficient stiffness to satisfy serviceability limit state requirements. However, it is evident that these methods cannot directly control structural seismic performance and damage under various earthquake intensity levels. As a consequence, this impacts on the design efficiency as it does not account for long-term risks, such as expected economic losses due to future seismic events. Meanwhile, although buildings designed according to current modern codes had successfully protected occupant's lives during major earthquakes, they also resulted in substantial direct and indirect costs due to repairable and non-repairable building damage. This has been highlighted in recent events (e.g. Northern Italy 2012, Kumamoto 2016, Osaka 2018, Petrinja 2020), for which the number of deaths was less than 50, but the resulting economic losses due to failure consequences were exceptionally high, ranging from $7 billion to $20 billion (National Geophysical Data Center 2023).

In recent decades, the need for cost-efficient design, that strikes a balance between structural safety and economic cost, has attracted increasing attention and is becoming a prominent aspect of earthquake engineering. As a result, optimisation techniques utilising performance-based seismic design (PBSD) have been developed. In such performance-based optimum designs, both structural and non-structural damage can be directly controlled to satisfy a set of performance-based design constraints that can represent building performance requirements (e.g. immediate occupancy, collapse prevention) under different earthquake intensity levels.

Numerous previous studies have employed different optimisation techniques to minimise the initial cost of RC structures while satisfying design requirements specified in the performance-based design. Studies by Zou and Chan, (2005) and Chan and Zou, (2004) demonstrate the minimisation of material costs of concrete and reinforcement steel, respectively, under minor and moderate earthquakes through the application of the Optimal Criteria (OC) algorithm, that limits inter-storey drifts at chosen performance levels. Razmara Shooli et al. (2019) proposed a performance-based design optimisation aiming to minimise initial material costs of RC frames, using a hybrid genetic algorithm (GA) and particle swarm optimisation (PSO). Mergos (2018a,

2017) used GA to minimise initial costs including costs of concrete, steel and formwork by considering PBSD target limits to control chord rotations and shear forces in structural elements. Zhang and Tian (2019) presented a simplified performance-based seismic design approach to minimise the initial construction cost of RC frames, whilst controlling plastic rotation and inter-storey drift. Overall structural stiffness and strength were considered as the only design variables. A practical performance-based optimum seismic design method for RC frames was developed by Hajirasouliha et al. (2012), using the concept of Uniform Damage Distribution (UDD) to minimise material costs. In this method the reinforcement ratio in each structural element served as the design variable, gradually being redistributed from less to more heavily damaged sections, until that the damage distribution along storey heights is more uniform and total reinforcement weight is minimised. However, this application of the UDD method mainly reduced the inter-storey drift at a certain performance level but did not necessarily lead to a better design with less structural and non-structural damage, in particular when dealing with multiple seismic hazard levels. And this optimisation considered the reinforcement ratio as only design variables, which may not result in most economically efficient designs.

In most aforementioned conventional performance-based design optimisation frameworks, the degree to which a building fulfils its pre-determined performance objective is highly contingent on the structural seismic demand and resistance capacities. These parameters are inherently uncertain in nature. Failure to adequately address such uncertainties may not necessarily lead to reliable solutions with acceptable structural damage levels and expected failure probabilities. Lin and Frangopol, (1996) showed that optimum designs satisfying deterministic performance constraints without accounting for the effect of uncertainties exhibited higher failure probability and less redundancy. Wen, (2001) also highlighted the need for reliability-based seismic designs. Fragiadakis and Papadrakakis, (2008) proposed a reliability-based optimisation framework that accounted for seismic uncertainties and concluded that, such an approach can improve both structural safety and cost-effectiveness.

Another issue is that most optimisation studies mainly aim to minimise initial construction cost without considering maintenance and repair over the effective lifetime of a structure. However, Gencturk et al. (2012) showed that initial construction cost was not satisfactory criterion in economy assessment and costs associated with damage repairs and indirect social costs can be dominant contributors to the total lifetime costs of a structure. Asadi and Hajirasouliha, (2020) indicated that design optimisation aiming to minimise initial construction costs does not necessarily lead to the best design solution when considering the total life-cycle cost. And blindly increasing the reinforcement ratio cannot guarantee a reduction in total life-cycle cost. Furthermore, Lagaros and Fragiadakis, (2011) pointed out that higher initial construction costs of RC frames did not always result in designs with diminished structural damage.

Hisahiro et al. (1998) also highlighted the importance of integrating life-cycle cost analysis. In this context, it was proposed to include life-cycle cost analysis (LCCA) for assessing both the initial costs and expected damage costs due to future earthquakes in a decision-support tool for economic efficiency. To minimise structural life-cycle cost, Wen and Kang, (2001a, 2001b) utilised a probabilistic optimisation framework that encompasses costs relating to construction, failure consequence and discounting over structural and considered both single and multiple natural hazards. Uncertainties arising from seismic hazards (i.e. random occurrence, variations on intensity levels) were effectively managed by assessing the mean occurrence rates of these hazards. The LCCA has also been accompanied by a number of different optimisation techniques to minimise initial costs and expected damage costs (or total life-cycle costs) for RC frames under earthquake loads (Razavi and Gholizadeh, 2021b; Gencturk, 2013; Zou et al., 2007). The results in this study confirm that assessing structural life-cycle costs can lead to a more efficient seismic design in terms of cost-saving and improved structural safety. The LCCA involves not only building repair costs but also costs associated with social aspects. Additionally, the expected life-cycle cost is generally calculated based on the probability of exceeding specific structural seismic performance corresponding to several limit states. This probabilistic-based analysis typically involves evaluating the vulnerability of buildings to different levels of ground motions and integrating the probabilities of levels of seismic hazards. It is thus helpful to account for uncertainties in the earthquakes events, such as their occurrence, magnitude and locations. The probabilistic approach also assesses the status where performance exceeds its target limits in a more realistic manner, whose advantages are concluded above compared to the conventional deterministic-based performance-based designs.

To reduce computational costs, the majority of previous optimisation studies incorporating with LCCA generally adopted nonlinear static (pushover) analysis to predict the seismic response. However, study by Moghaddam and Hajirasouliha, (2006a) illustrated that the commonly used pushover analysis with fixed load pattern may lead to unrealistic seismic performance predictions, especially when structure behaving nonlinearly and higher modes dominate seismic responses. This issue is particularly important in the calculation of the total life-cycle cost, as structural seismic performance and the corresponding exceedance probability of damage states are crucial elements in the cost assessment. Mitropoulou et al. (2011) studied the impacts of different seismic analysis methods, performance criteria, structure types (regular or irregular) and number of seismic records in the life-cycle assessment of optimally designed RC frames. The results concluded that the nonlinear static analysis should be avoided in life-cycle cost analysis as it produced unrealistic predictions of performance in some cases.

To address the above highlighted issues, this study adopts a practical multi-level performance-based optimisation methodology for the seismic design of multi-storey RC frames, grounded in the concept of uniform damage distribution (UDD). Both section dimensions and reinforcement ratios are iteratively optimised to minimise total material usage, while structural damage at the element (i.e. plastic hinge rotation) and structural (i.e. inter-storey drift) levels are simultaneously controlled to satisfy multiple performance objectives under various seismic hazard levels. As a consequence, materials capacities in most of stories are nearly fully exploited and more uniform damage distribution is achieved. Damage in the life-cycle cost assessment is quantified using incremental dynamic analysis, and it is represented through a global damage index and maximum floor acceleration. In this life-cycle cost calculation, fragility analysis is performed to account for the uncertainties in seismic demands and earthquake excitations and to evaluate the limit-state exceedance probability. The total life-cycle costs encompass the initial construction costs, expected damage repair costs and the social losses. These social losses consist of re-insertion costs into normal routes, costs associated with human injuries and fatality, and costs relating to loss of business and economic activities after earthquakes occurs. These loss components are caused by structural and non-structural damage, which are quantified by inter-storey drifts and maximum storey acceleration under earthquake loads, respectively. The efficiency of the methodology is investigated by assessing total life-cycle costs for 3-, 5-, 10- and 15-storey RC frames. The sensitivity of the optimum solutions to the selection of different sets of earthquakes records is examined using a set of spectrum-compatible generated artificial and a set of selected natural seismic records.

4.3. Performance-based Optimisation Methodology

4.3.1. Performance objectives and corresponding seismic hazard levels

The seismic design optimisation is formulated as a multi-objective performance-based optimisation problem. The objective is to minimise total material usage (in terms of concrete volume and reinforcement weight), whilst ensuring that local and global structural damage is simultaneously controlled to meet multiple performance objectives under different earthquake intensity levels. Three performance objectives are adopted: Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). Their relations to earthquake hazard levels (here quantified as occurrence probability of earthquakes) and to the peak ground acceleration (PGA) of subjected earthquake records are presented in Table 4-1, following recommendations in ASCE/SEI 41-13, (2014).

Table 4-1: Relation between performance objectives, annual occurrence probability of the corresponding seismic hazard level and peak ground acceleration (Dong et al., 2023)

Performance objective	Earthquake excitation	Occurrence Probability	Return Period (year)	PGA (g)
Immediate Occupancy (IO)	Frequent earthquake	50% in 50 years	72	0.1
Life Safety (LS)	Design basis earthquake (DBE)	10% in 50 years	475	0.4
Collapse Prevention (CP)	Maximum considered earthquake (MCE)	2% in 50 years	2475	0.65

Performance and design constraints

Both plastic hinge rotations in column (θ_c) and beam elements (θ_b) and inter-storey drift ratios (Δ_{max}) are considered as performance parameters at local and global level, respectively. Their values are simultaneously controlled in the design optimisation and checked against performance constraints that relate to pre-determined performance objectives in the PBSD, so that a building has sufficient "capacity" to resist seismic "demand". To clearly indicate the relationship between the structural response and the considered performance objective, this study employs performance ratios (PR_{drift}, $PR_{rotation,C}$, $PR_{rotation,B}$) that express the ratios of the maximum seismic responses (i.e. maximum beam plastic rotation, maximum column plastic rotation, maximum inter-storey drift) to the corresponding target limits at specific performance levels. More detailed information on the evaluation of the performance ratios with reference to the rotations in structural members and the drift in each storey can be found in (Dong et al., 2023). The maximum beam and column plastic rotations are computed per storey. The maximum seismic responses in terms of plastic hinge rotation and inter-storey drift are evaluated as average values for a group of selected earthquake records using non-linear dynamic (time history) analysis.

In accordance with the design requirements in ASCE/SEI 41-13, (2014), the target limiting values of plastic rotations in beams ($\theta_{target,B}$) and columns ($\theta_{target,C}$) are determined at IO, LS and CP levels, respectively. Since rotation capacities of RC elements are primarily controlled by structural flexural behaviour, the target limits ($\theta_{target,B}$, $\theta_{target,C}$) cannot be pre-determined as constant values and they are updated at each iteration of the optimisation process using section properties and loading information (i.e. axial loads, shear forces). It should be noted that ASCE/SEI 41-13, (2014) only considers plastic hinge rotations as the only structural performance parameter under seismic loads. The target limits for the inter-storey drift ratio (Δ_{target}) are decided as 1%, 2% and 4% to satisfy performance objectives IO, LS and CP, respectively, following criteria in ASCE/SEI 41-06, (2007). Apart from performance constraints, design constraints should also be considered in the optimisation formulation to achieve a practical seismic design. More information about the target limits of the plastic hinge rotations and adopted design constraints can be found in the author's previous paper (Dong et al., 2023).

4.3.2. Multi-level UDD optimisation

In UDD optimisation, materials in each storey (i.e. concrete volume and reinforcement weight) are gradually redistributed from heavily damaged to less damaged parts of a structure, until the materials in each storey are most efficiently used and seismic responses are distributed in a more uniform pattern. As a consequence, the total material usage is minimised, while limiting structural damage corresponding to various seismic hazard levels. It is worth mentioning that the optimisation methodology proposed in this study is general and can be applied to improve and modify any design solutions originally based on different seismic design codes or even non-compliant designs. This can be achieved by satisfying pre-determined design objectives, practical design constraints, and performance constraints specified in PBD guidelines.

In seismic design, it is generally assumed that structures behave nearly elastically under frequent earthquakes and behave mainly within the inelastic range when subjected to moderate to severe earthquakes. Therefore, the proposed optimisation methodology adopts an elastic and inelastic phase. The elastic phase aims to minimise total concrete volume, whilst controlling the inter-storey drift ratio (IDR) at each storey level to satisfy the IO performance objective under the design seismic hazard level (see in Table 4-1). The optimisation in the inelastic phase aims to minimise reinforcement weight, while satisfying multiple performance objectives in terms of LS and CP. The maximum plastic hinge rotation is considered to be the primary performance index; IDR is only accounted in the design optimisation when the maximum IDR exceeds its target limits at the specific performance level. Details of how the longitudinal reinforcement ratio is modified using performance indexes are found in below, based on the concept of UDD:

If the maximum IDR exceeds its limiting value and $max[PR_{rotation,C}, PR_{rotation,B}] < 1$:

$$[(\rho_C)_i]_{n+1} = [(\rho_C)_i]_n \times (PR_{drift,i}^{critical})^{\beta} \times (1 - (PR_{rotation,i,C}^{critical})^2)^{(-\beta)} \tag{4.1}$$

$$[(\rho_B)_i]_{n+1} = [(\rho_B)_i]_n \times (PR_{drift,i}^{critical})^{\beta} \times (1 - (PR_{rotation,i,B}^{critical})^2)^{(-\beta)} \tag{4.2}$$

In the other conditions:

$$[(\rho_C)_i]_{n+1} = [(\rho_C)_i]_n \times (PR_{rotation,i,C}^{critical})^{(\alpha)} \tag{4.3}$$

$$[(\rho_B)_i]_{n+1} = [(\rho_B)_i]_n \times (PR_{rotation,i,B}^{critical})^{(\alpha)} \tag{4.4}$$

where $[(\rho_B)_i]_n$ and $[(\rho_C)_i]_n$ denote reinforcement ratios in beam and column elements at i^{th} storey level in n^{th} iteration, respectively; as multiple performance objectives are required to be satisfied in this optimisation, $PR_{drift,i}^{critical}$, $PR_{rotation,i,C}^{critical}$ and $PR_{rotation,i,B}^{critical}$ are critical performance ratios representing the drift responses, the plastic rotations in columns and plastic rotations in beams in i^{th} storey, respectively, they are decided as the largest ratios relating to three pre-determined performance levels, β represents a convergence parameter for those UDD formulas where both maximum drifts and plastic rotations are accounted as performance parameters, and α is a convergence parameter controlling convergence spade when only one performance parameter is involved in the UDD formula.

The appropriate values of α and β are decided after investigating the effects of convergence factors in optimisation results. For additional details on the investigation results, they can be found in the author's previous work (Dong et al., 2023). To provide a balance between convergence speed and solution accuracy, the convergence factor (α) equals to 0.2 is utilised and kept constant in the above UDD formulas, while β is always equal to half of α.

The detailed iterative optimisation process in both elastic and plastic phases, including initial design details, the modified design variables, the controlled performance parameters and stopping criteria, are summarised and presented as flowcharts in Figure 4-1.

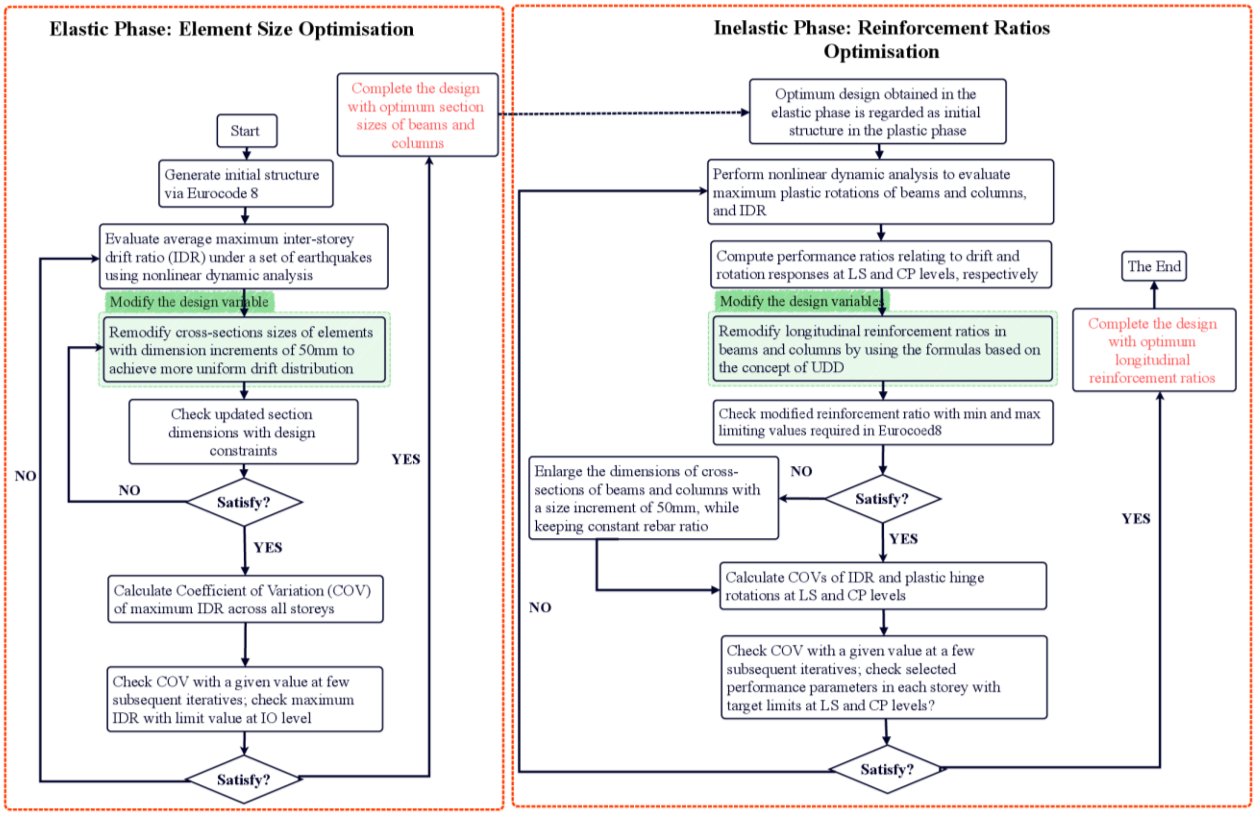


Figure 4-1: Flowchart of the utilised multi-level performance-based design optimisation of RC frames

4.4. Life-cycle cost analysis

The life-cycle cost (C_{TOT}) of a structure is defined as the total expected costs to maintain structural conditions over the design life time of a new building or the remaining effective life time of a retrofitted structure, and it can be expressed as a function of time and the design vectors as suggested by Wen and Kang, (2001a):

$$C_{TOT}(t, s) = C_{IN}(s) + C_{LS}(t, s) \tag{4.5}$$

Where: C_{IN} is the initial cost of a new or retrofitted structure; C_{LS} represents the expected life-cycle damage cost referring to all specific damage limit states; s is the design vectors corresponding to material properties, design loads and resistances that may affect structural performances; t is the lifetime of a structure.

4.4.1. Initial cost

The term "initial cost" for a new building typically refers to the initial construction costs, which mainly consist of the costs of structural materials and corresponding labour. To compare between code-based and optimum design solutions provided in this study, it is assumed that the evaluation of initial construction costs mainly involves the costs associated with column and beam elements in a frame. This is because their structural properties (i.e. cross-section dimensions and reinforcement ratios) vary as specific design variables in the proposed optimisation methodology. Other cost components, such as those associated with foundations, floor slabs erection, infill walls and non-structural elements, are not included in the calculation of the initial cost here. Design details of other structural and non-structural elements (i.e. foundation, infill wall, floor slab, stair and railings) and their corresponding construction costs can be considered in the future optimisation study, to provide more comprehensive cost calculation results.

For RC frames, the structural materials costs include concrete, steel reinforcement and formwork costs. The cost of construction mainly refers to labour costs involved in the fabrication of each structural component, which includes pouring and curing ready-mix concrete, bending and placing rebar in elements and installing or removing formwork. The unit material price and labour costs used in the calculation of the initial cost of RC frames are given in Table 4-2. Hence, the initial cost of RC frames is mathematically expressed as a function of specific design variables (i.e. width and depth of cross-section, reinforcement ratio in each element). It should be noted that, since the labour costs are decided based on 2011 Building Construction Cost Data, as suggested by Shin and Singh, (2017), the data utilised in this study should be simply converted to current value according to building cost indices and using the formula:

$$Cost\ in\ Year A = \frac{Index\ for\ Year A}{Index\ for\ Year\ B} \times Cost\ in\ Year B \tag{4.6}$$

In this study, building inflation rate in year 2022 in UK is decided as 1.3 ("Construction costs in the United Kingdom (UK)," 2022).

Moreover, a previous study by Mergos (2018) highlighted that unit prices of concrete, steel, and formwork involved in the initial cost calculation can be substituted with the material unit environmental impacts to compute the total embodied CO_2 emissions of RC frames. Hence, it can be inferred that, the optimisation study, which aims to minimise the total initial construction cost, generally also facilitating to the reduction of carbon emissions associated with RC frames.

Table 4-2: Material and labour costs of RC frames

Item	Unit	Cost (£/unit)
Material Costs		
Reinforcement Steel	Kilogram	1
Concrete, ready mix (30 MPa)	m3	100
Formwork	m2	25
Labour Costs		
Placing steel in elements	Kilogram	0.6
Pouring concrete	m3	50
Installing forming	m2	90

4.4.2. Expected damage cost

4.4.2.1. Elements in expected damage cost

In formulating the total life-cycle cost (TLCC), the expected damage cost refers to potential direct and indirect economic losses caused by structural and non-structural damage under future earthquakes over the effective lifetime of a structure. To calculate these losses, suitable damage indices relating to structural seismic performances are first selected to accurately quantify structural and non-structural damage under different earthquake intensities. More details about selecting the damage indices and evaluating the corresponding seismic performances will be discussed latter. The total life-cycle cost is calculated based on three categories: initial construction cost, expected damage repair cost (direct loss) and social losses associated with the occurrence of earthquake (indirect loss), as suggested in previous works by Möller et al. (2015) and Zou et al. (2007). In general, the building repair is a costly operation, as it consists of not only costs of retrofitting damaged parts, but also costs of removing fully damaged elements. The social cost is economic loss that society has to face after earthquakes, which includes costs of re-insertion into normal routine, loss of human injuries and lives, costs due to loss of business and economic activity. It should be noted that quantifying the life loss is the most challenging and debatable part in the economic assessment, as it needs to consider not only pure economic reasons but also the fact that human lives are irreplaceable. In this study, the expected damage cost for the i^{th} damage state is calculated following equation:

$$C_{LS}^{i} = C_{str-dam}^{i} + C_{nonstr-dam}^{i} + C_{ren}^{i} + C_{com}^{i} + C_{min-inj}^{i} + C_{maj-inj}^{i} + C_{fat}^{i} \tag{4.7}$$

where: $C_{str-dam}^{i}$ is repair cost for structural damage; $C_{nonstr-dam}^{i}$ represents damage repair cost of non-structural elements; C_{ren}^{i} is loss of rental cost; C_{com}^{i} is commercial loss cost which is related to downtime of working and potential loss to the company band; $C_{min-inj}^{i}$ and $C_{maj-inj}^{i}$ are costs due to minor and major injuries, respectively, and the cost of major injuries is estimated based on data in accidental permanent injury insurance in UK; C_{fat}^{i} is cost of human fatalities.

The detailed evaluation of the expected damage costs including unit prices and formulas used are provided in Table 4-3. These are based on previous studies (Mitropoulou et al., 2011; Wen and Kang, 2001b) as well as relevant insurance data ("Accidental Permanent Injury Insurance," 2018). This study assumes that downtime for a damaged building is 6 months, and occupancy rate for a residential building is two persons per 100 m^2. Moreover, since the data provided in the Table 4-3 are based on a study in year 2011, a rate equal to 1.3 is utilised to convert the costs considering building inflation till nowadays. Information on the parameters used in the expected damage cost equation (e.g. mean damage index, loss of function, expected injury and fatality rates), their corresponding values as well as their relation to damage states can be found in Table 4-4, in accordance with design codes FEMA 227, (1992) and ATC 13, (1985).

Table 4-3: Basic costs and limit-state cost calculations (Mitropoulou et al., 2011; Wen and Kang, 2001b)

Variable	Related to	Equation	Basic Cost
$C^i_{str-dam}$	Global damage index	Replacement cost x floor area x mean damage index	£1500/m2
$C^i_{nonstr-dam}$	Max floor acc.	Replacement cost x floor area x mean damage index	£500/m2
C^i_{ren}	Global damage index	Rental rate x gross leasable area x loss of function	£10/month/m2
C^i_{com}	Global damage index	Rental rate x gross leasable area x downtime	£2000/month/m2
$C^i_{min-inj}$	Global damage index	Minor injury cost per person x floor area x occupancy rate x expected minor injury rate	£2000/person
$C^i_{maj-inj}$	Global damage index	Major injury cost per person x floor area x occupancy rate x expected major injury rate	£100000/person
C^i_{fat}	Global damage index	Fatality cost per person x floor area x occupancy rate x expected fatality rate	£2800000/person

Table 4-4: Classification of levels of damage states, limiting values of the damage indices and corresponding limit state parameters for life-cycle cost assessment

Damage state	Floor acceleration (g)	FEMA 227, (1992)				ATC 13, (1985)	
		Mean damage Index of elements (%)	Minor injuries	Major injuries	Fatalities	Downtime (%)	Loss of function (%)
None.	$0.15 > a_{floor}$	0	0	0	0	0	0
Slight	$0.15 < a_{floor} < 0.30$	0.50	0.00003	0.000004	0.000001	0.9	0.9
Light	$0.30 < a_{floor} < 0.60$	5	0.0003	0.0004	0.00001	3.5	3.5
Moderate	$0.60 < a_{floor} < 1.20$	20	0.003	0.004	0.0001	13	13
Heavy	$1.20 < a_{floor} < 1.80$	45	0.03	0.04	0.001	35	35
Major	$1.80 < a_{floor} < 2.40$	80	0.3	0.4	0.01	65	65
Destroyed	$2.40 < a_{floor}$	100	0.4	0.4	0.2	100	100

This study assumes that the structure will be restored to its original condition after each seismic event. A Poisson process model is utilised to quantify the earthquake occurrence rate as *v/year*. The expected damage cost (C_{LS}) is further calculated considering *N* damage states by using the formula (Wen and Kang, 2001b):

$$C_{LS}(t,s) = \frac{(1-e^{-\lambda t})}{\lambda}\sum_{i=1}^{N} C^i_{LS}P^i \tag{4.8}$$

$$P^i = P\left(DI_{max} > DI_{max}^i\right) - P(DI_{max} > DI_{max}^{i+1}) \tag{4.9}$$

where: *N* donates the total number of expected damage states; P^i is the occurrence probability of the i^{th} damage state; λ is constant discount rate per year (λ = 6%), so that costs caused by future hazards can be converted into present values; t is design life period, which is considered to be 50 years for a new building in this study. More details on the evaluation of the occurrence probability (P^i) will be explained in the following sections.

4.4.2.2. Selected damage indices

The global damage index (DI_{global}) is selected as the damage index (DI) in the life-cycle cost assessment, to quantify the structural damage at global level under different seismic hazard levels. To evaluate the global damage index, structural damage index at each storey level (D_i) is first calculated based on inter-storey drift responses (Powell and Allahabadi, 1988):

$$D_i = \left(\frac{\Delta_{max,i} - \Delta_{yield,i}}{\Delta_{ultimate,i} - \Delta_{yield,i}}\right)^b \tag{4.10}$$

where $\Delta_{max,i}$, $\Delta_{yield,i}$ and $\Delta_{ultimate,i}$ are maximum inter-storey drift, yielding drift and ultimate drift in *i*th storey, respectively. The $\Delta_{max,i}$ is obtained by performing nonlinear dynamic analysis, and the $\Delta_{yield,i}$ is evaluated by analysing a bilinear elastic-plastic structure which has the same ultimate displacement and the same energy absorption capacity as the case study. The $\Delta_{ultimate,i}$ is assumed to be the drift-based limiting value corresponding to the CP performance objective when structural failure is assumed to occur, which is 4% as per ASCE/SEI 41-06, (2007). Constant parameter b is suggested to be 1.5 for RC frames (Cosenza and Manfredi, 2000). More details of the evaluation of the damage index and the yielding drift can be found in (Dong et al., 2023).

The overall structural damage (DI_{global}) is quantified to calculate a weighted average of local damage indices (D_i) among all stories through weight (w_i) assigned to i^{th} storey:

$$DI_{global} = \frac{\sum_{i=1}^{N} D_i w_i}{\sum_{i=1}^{N} w_i} \tag{4.11}$$

where N denotes the number of storey levels, the weighting factor w_i reflects the varying degrees of importance of damage states of i^{th} storey in maintaining the structural integrity. In this study, w_i depends on the magnitude of the damage index for the i^{th} storey (D_i), ensuring that storeys experiencing severe damage are weighted more heavily and assigned greater importance in quantifying the overall structural damage.

Such a combination of damage indices to calculate an overall value through a weighted average form is also widely used in previous studies (De Domenico and Hajirasouliha, 2021; Nabid et al., 2018). The global damage index D_g is a ratio in a range from 0 (undamaged) to 1 (completely damaged). This approach is widely used due to its balance between simplicity and accuracy (Ghobarah et al., 1999).

In the cost assessment, non-structural damage is considered, with maximum floor acceleration (a_{floor}) adopted as the damage index to quantify the damage of non-structural components (e.g. damage of architecture elements, furniture and equipment), as suggested in a previous study (Asadi and Hajirasouliha, 2020).

Six damage statuses classified into six states, ranging from slight damage to destroyed, are considered to quantify structural and non-structural damage in the calculation of the total life-cycle cost. The relationship between the target limiting values of both specific damage indices (global damage index (DI_{global}) and maximum floor acceleration (a_{floor})) corresponding to the pre-determined damage states is presented in Table 4-4. The selected seismic performances are assessed under different earthquake intensities through incremental dynamic analysis (IDA), the details of which are provided in the section below.

4.4.2.3. Fragility analysis

Fragility analysis expresses structural seismic demand exceeding its limit value at a particular damage state in a probabilistic manner. Fragility curves are commonly generated using a range of different seismic hazard levels and in accordance with various predetermined damage states.

As shown in the Equations (4.8) and (4.9), to calculate the expected life-cycle damage cost, it is required to first estimate the annual probability of exceedance (($P\left(DI_{max} > DI_{max}^{i}\right)$) when the maximum value of damage index (DI_{max}) exceeds a given limiting value (DI_{max}^{i}) of the i^{th} damage state by using the follow equation:

$$\mathrm{P}(DI_{max} > DI_{max}^{i}) = \int_{0}^{\infty} \mathrm{P}\left(DI_{max} > DI_{max}^{i} \middle| \mathrm{IM} = \mathrm{PGA}_{j}\right) \left|\frac{\mathrm{d}\nu(\mathrm{IM})}{\mathrm{dIM}}\right| \mathrm{dIM} \qquad (4.12)$$

where: $\mathrm{P}(DI_{max} > DI_{max}^{i}|\mathrm{IM} = \mathrm{PGA}j)$ denotes the probability exceeding the i^{th} damage state under a given intensity level ($\mathrm{IM} = \mathrm{PGA}j$), also namely as the fragility function; $\frac{\mathrm{d}\nu(\mathrm{IM})}{\mathrm{dIM}}$ represents the mean annual rate of earthquake intensity, IM, which can be obtained as the slope of hazard curve. In this study, both global damage index (DI_{global}) and maximum floor acceleration will be considered as the parameter DI_{max} in the function of exceedance probability, respectively. The limiting values at the i^{th} damage state can be found in the Table 4-4.

In the Equation (4.12), it requires numerically integrating the exceedance probability of the specific damage state with result from site seismic hazard analysis (i.e. the mean annual rate of earthquake intensity). Incremental dynamic analysis under a wide range of earthquakes with multiple intensity levels is used in the evaluation. When the global damage index (DI_{global}) is utilized as the damage index, the conditional limit-state probability at each practical intensity level ($\mathrm{P}(DI_{max} > DI^{i}_{max}|\mathrm{IM} = \mathrm{PGA}j)$) is calculated using the following fragility function:

$$\mathrm{P}\left(DI_{global} > DI^{i}_{global,target}\middle|\mathrm{IM} = \mathrm{PGA}_j\right) = 1 - \Phi\left[\frac{DI^{i}_{global,target} - \overline{\mathrm{DI}_{global}|\mathrm{IM=PGA}_j}}{\beta_{\mathrm{DI}_{global}}}\right] \tag{4.13}$$

where: $\Phi[.]$ is the standard normal cumulative distribution function; $\mathrm{DI}_{global}|\mathrm{IM} = \mathrm{PGA}j$ and $\beta_{\mathrm{DI}_{global}}$ are the average and standard deviation results of the DI_{global} under a group of earthquakes with a intensity level $\mathrm{PGA}j$, $DI^{i}_{global,target}$ is the given target limit of the DI_{global} at the i^{th} damage state.

Furthermore, the maximum floor acceleration (a_{floor}) is used as another damage index, in the fragility analysis, at a specific intensity level ($\mathrm{IM} = \mathrm{PGA}j$), it is assumed that its distribution is lognormal. The equation of limit-state probability is thus further developed as:

$$\mathrm{P}\left(a_{floor} > a^{i}_{floor,target}\middle|\mathrm{IM} = \mathrm{PGA}_j\right) = 1 - \Phi\left[\frac{\ln\left(a^{i}_{floor,target}\right) - \overline{\ln\left(a_{floor}\right)}}{\beta_{a_{floor}}}\right] \tag{4.14}$$

where: $\ln\left(a^{i}_{floor,target}\right)$ is threshold value of the damage index (i.e. maximum floor acceleration) in a logarithmic form for the i^{th} damage state, $\overline{\ln(a_{floor})}$ and $\beta_{a_{floor}}$ are the average and standard deviation results of the natural logarithm of a_{floor} under a set of earthquake with a specific intensity level ($\mathrm{IM} = \mathrm{PGA}j$).

This study uses global damage index (DI_{global}) and maximum floor acceleration (a_{floor}) as the two damage indices to quantify structural and non-structural damage, respectively, and their results were evaluated using incremental dynamic analysis (IDA). Fragility analysis is performed using the performance results from the IDA to generate fragility curves for the damage parameters, considering multiple damage states. The limit-state probability of the damage indices is first calculated using Equations (4.13) and (4. 14), at a particular damage state. After calculating the limit-state probability, the occurrence probability is estimated by using Equation (4.9) under multiple seismic hazard levels for a specific damage state. Consequently, expected damage cost (C_{LS}) of a structural over its lifetime was estimated by using Equations (4. 7) and (4. 8), after the fragility results were produced considering N damage states; the cost was calculated as sum of damage costs of different elements (i.e. structural damage repair cost, human minor and major injury costs). The total life-cycle cost was calculated after evaluating the expected damage cost by using Equation (4. 5).

4.5. Implementation of Optimisation Methodology in Multi-storey Design

4.5.1. Modelling of RC frames and assumptions

The details of the 3-, 5-, 10- and 15-storey RC frames used in the implementation of the methodology are shown in Figure 4-2. Details of the initial and optimum designs of the RC frames, including section sizes and longitudinal reinforcement ratios are summarized in tables in Appendix B and C. The dead and live loads for interior storeys were assumed to be 4.6 kN/m^2 and 2 kN/m^2, respectively, while these loads were reduced to 4 kN/m^2 and 0.7 kN/m^2 for roof. To model residential buildings in high-seismic regions, initial frames are designed using seismic loads based on the Eurocode 8-based design spectrum with a peak ground acceleration (PGA) of 0.4g, and site soil type C. It should be emphasised that any design PGA and soil parameters, aligned with survey results of a certain seismic region, can be employed for this initial frame design. The values of these parameters are independent of the obtained optimum results in this study. The compressive strength of concrete and yield strength of steel reinforcement were taken to be 30MPa and 500MPa, respectively. The initial buildings were designed to primarily satisfy serviceability, safety and durability design requirements specified in Eurocode 2 and 8 (CEN, 2004b, 2004a).

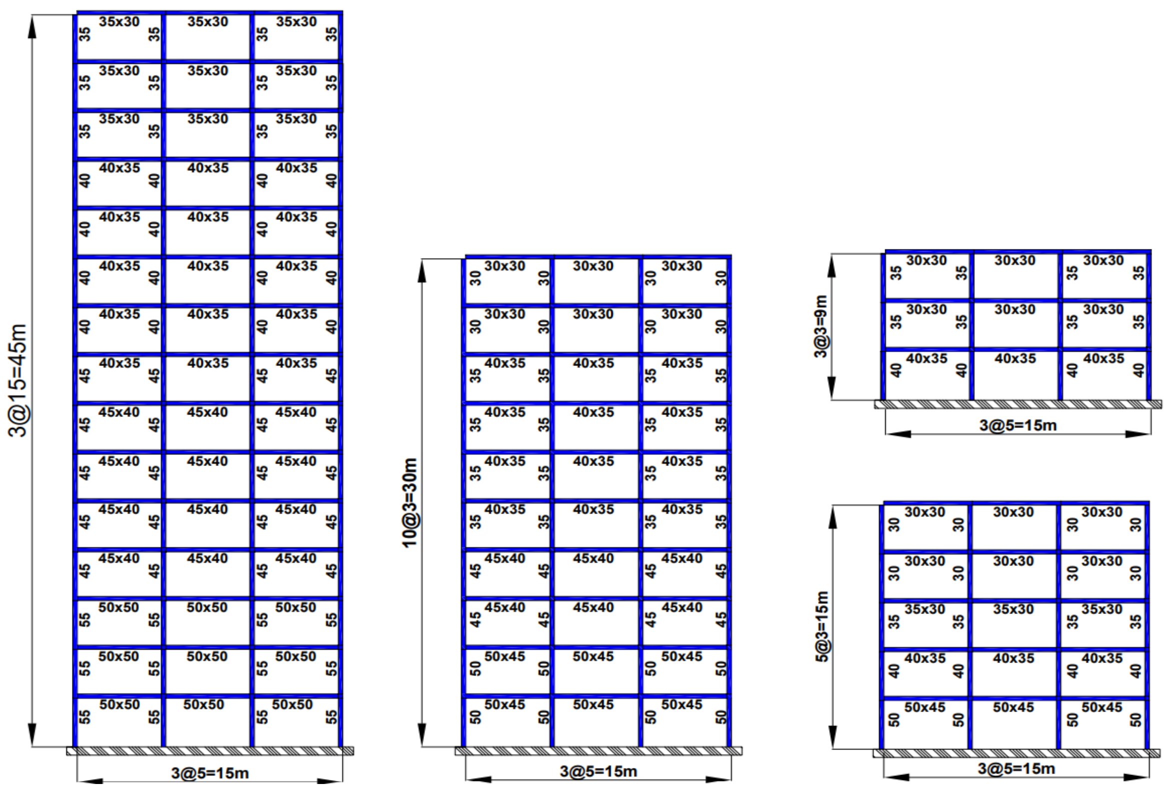


Figure 4-2: Geometry of 3-, 5-, 10- and 15-storey RC frames and section details of the code-based designs (Columns: dimension of square section, Beams: height x width)

The frames are modelled and analysed using the finite element software OpenSees (McKenna et al., 2006). "Concrete02" and "Steel02" (or Giuffre-Menegotto-Pinto) models are utilised to represent the material properties of concrete and reinforcement steel, respectively. Beam and column elements were modelled employing force-based nonlinear elements ("*forceBeamColumn*"), where six Modified Gauss-Radau integration points are considered for each element (Scott and Fenves, 2006). The non-linear time history analysis is performed using the distributed-plasticity approach in the OpenSees that assumes that yielding can develop within specific regions of beam and column elements (plastic hinge regions) where structural nonlinearity is most likely to occur (Neuenhofer and Filippou, 1997). The length of the plastic hinge region is determined in accordance with material and structural properties of RC frames at each iteration, using formulars in Eurocode 8, part 3 (CEN, 2005). More details on the application of the finite element models are given in (Dong et al., 2023). In accordance with the "Modified Gauss-Radau" integration method, the plastic rotation can be calculated as the product of the average curvatures in the plastic hinge region times the length of the plastic hinge region (Scott and Fenves, 2006). P-Delta effects were also considered for analyses of the structures, Rayleigh damping model with a constant ratio 5% was assigned to the first mode and to any modes at which cumulative mass participation exceeds 95% (De Domenico and Hajirasouliha, 2021; Asadi and Hajirasouliha, 2020; Moghaddam and Hajirasouliha, 2008). The effect of concrete cracking on overall element stiffness is considered by utilising effective flexural and shear stiffness for each element.

4.5.2. Selected earthquake records and code-based design spectrum

During the design optimisation process, a set of fifteen natural earthquake records is used and selected from both Pacific Earthquake Engineering Research Centre (PEER) online database and SIMBAD database (Smerzini et al., 2014). Figure 4-3 shows the elastic acceleration response spectra of the selected un-scaled seismic ground motions as well as Eurocode 8-based design spectrum. The designations and characteristics of the fifteen natural earthquakes are also summarized in Table 4-5. It can be observed that the average spectrum of the natural records is close to the target design spectrum within the range of periods that contain the fundamental periods of the four selected RC frames.

Table 4-5: Characteristics of selected natural earthquake records (Smerzini et al., 2014)

No.	Earthquake	Mw	Station ID/component	PGA(g)	PGV (cm/s)	PGD (cm)
1	1979 Imperial Valley	6.5	IMPVALL/HE04140	0.485	37.4	20.23
2	1987 Supersition Hills	6.7	SUPERST/BICC000	0.358	46.4	17.50
3	1989 Loma Prieta	6.9	LOMAP/G03000	0.555	35.7	8.21
4	1989 Loma Prieta	6.9	LGPC/x	0.531	51.5	55.21
5	1990 Manji Abbar	7.4	MANJIL/ABBAR-T	0.496	52.1	20.77
6	1992 Cape Mendocino	6.9	CAPEMEND/PET000	0.590	48.4	21.74
7	1994 Northridge	6.7	ST_24279/x	0.583	74.9	17.70
8	1994 Nothridge	6.7	NORTHR/NWH360	0.590	97.2	38.05
9	1995 Kebe Hyogo	6.9	JMA/y	0.832	91.1	20.36
10	1999 Kocaeli	7.5	KOCAELI/DZC270	0.356	46.3	17.66

11	1999 Duzce	7.2	DUZCE/DZC270	0.535	83.5	51.59
12	2000 South Iceland	6.4	ST_109/y	0.706	105.1	26.36
13	2000 Tottori Prefecture	6.6	TTR009/y	0.611	36.3	13.00
14	2005 NW Off Kyushu	6.6	FKO006/y	0.279	57.7	16.75
15	2007 Niigata prefecture	6.6	NIG018/x	0.506	83.8	34.26

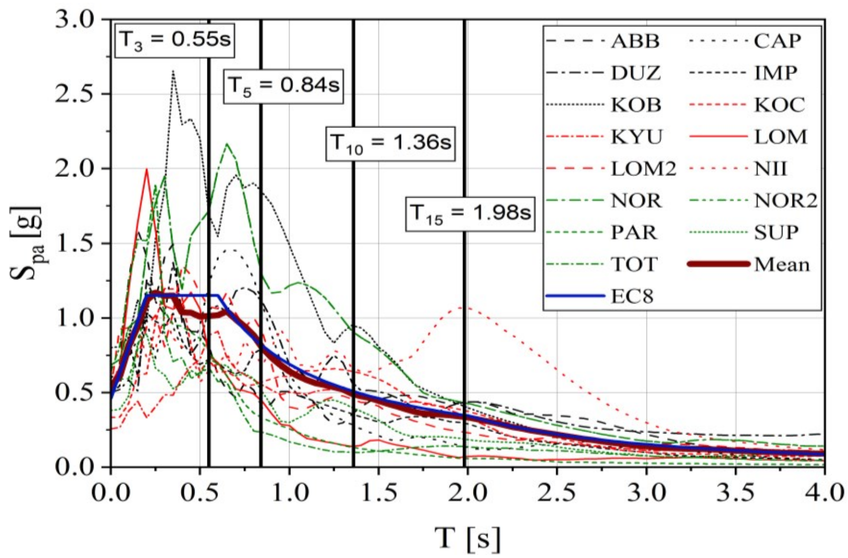


Figure 4-3: Eurocode 8-based target design response spectrum and elastic response spectrum of fifteen natural records

In additional to the above, six artificial seismic ground motions records are used that are fully compatible with the target design spectrum, generated using target acceleration spectra compatible time histories (TARSCTHS) (Papageorgiou et al., 2002). The spectrum compatibility conditions of the generated artificial earthquakes are also verified, in a range covering fundamental periods of four selected RC frames, so that their mean response spectrum is within a ±10% tolerance compared to code-based design spectrum, as confirmed in Figure 4-4. As the target design spectrum is utilised to represent the different pre-determined seismic hazard levels, both selected natural and generated artificial records are scaled to different target PGA levels.

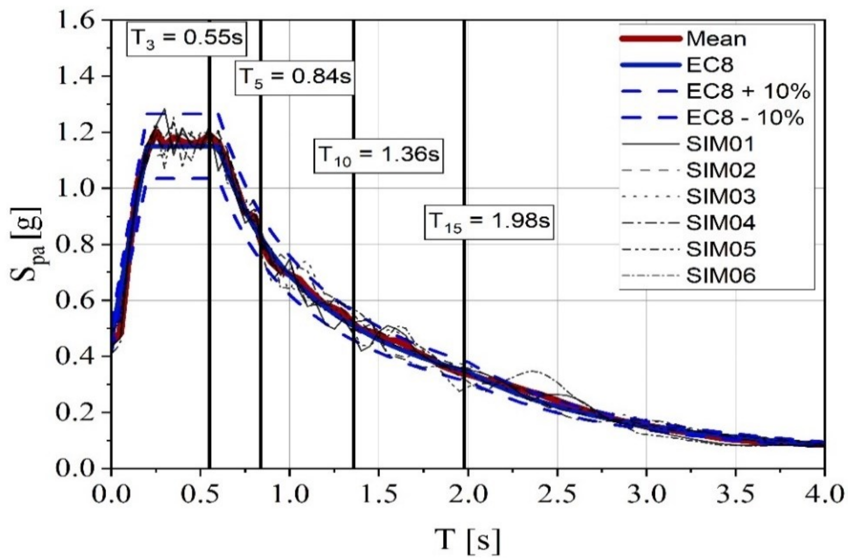


Figure 4-4: Eurocode 8-based target design response spectrum and mean spectrum of artificial earthquake records

4.6. Life-cycle Cost Assessment of Optimum Design

4.6.1. Incremental dynamic analysis results

Uncertainties arising from seismic inputs including randomness in earthquake intensity can be properly managed by employing the incremental dynamic analysis (IDA) in the seismic design, it aims to provide a complete range of the structural seismic response under different earthquake intensities. The IDA relies on: (i) an intensity measure (IM) expressing earthquake intensity level, and serving as scaling factor of the selected earthquake records, and (ii) an engineering demand parameter (EDP) describing seismic response of a structure (i.e. maximum inter-storey drift, maximum floor acceleration, global damage index).

A suitable selection of an IM in the IDA, serving as the first step of fragility curve, is hence crucial in this study. Previous study has highlighted that Peak Ground Acceleration (*PGA*), and first-mode Spectral Acceleration ($Sa(T_1)$) are the most commonly used IM parameters (Zentner et al. 2017). Most previous studies have illustrated that the parameter $Sa(T_1)$ is more efficient than *PGA* in reducing the dispersion of IM values (Zhou and Li 2015; Shahi et al. 2014; Vamvatsikos and Cornell 2002). However, since $Sa(T_1)$ is computed based on the natural period of vibration of each structure, it generally differs for initial and optimum structures in this study, considering the iterative modification of section sizes of structural elements in the optimisation process. PGA, which is only related to the seismic ground motion characteristics, is therefore more recommended as the IM parameter in the analysis. It can remain constant for code-based and optimum design solutions under each specific earthquake intensity level, facilitating performance comparison purposes in this study. Another reason is that, as the structural storey increases, the effects of higher modes may play a dominant role in determining $Sa(T_1)$, which is based on structural first mode. Selecting PGA as the IM eliminates the dependency of the IDA results on the model specifications and geometry of the structures (Mohsenian et al. 2023).

The IM is selected as peak ground acceleration (PGA) for the code-based 5%-damped elastic response spectrum, ranging from 0.05g to 1.0g. For each IM value, a group of artificial earthquake records are applied and simply scaled to the target PGA level. The global damage index, serving as the EDP in this analysis, is computed based on maximum inter-storey drifts under each pre-determined IM level using Equations (4.10) and (4.11) as shown above. For each referenced frame, average values of the global damage indices from six spectrum-compatible artificial earthquakes are computed and compared between the optimum design and the initial design codified by Eurocode 8 under different earthquake intensity level.

Results from the IDA that present the relationship between the selected seismic hazard level and the maximum seismic performance of the referenced frames are given in Figures 4-5 and 4-6. The design region is determined according to the specific PGA value for the maximum considered earthquake (MCE) level (here decide as PGA = 0.65g), which represents the maximum level of seismic ground motion considered in seismic design to maintain structural integrity and the highest level expected to occur at a site over the structural life period. It should be noted the determination of MCE level varies with country, historical seismic data and geological study results, which in turn affects the decision of this design region.

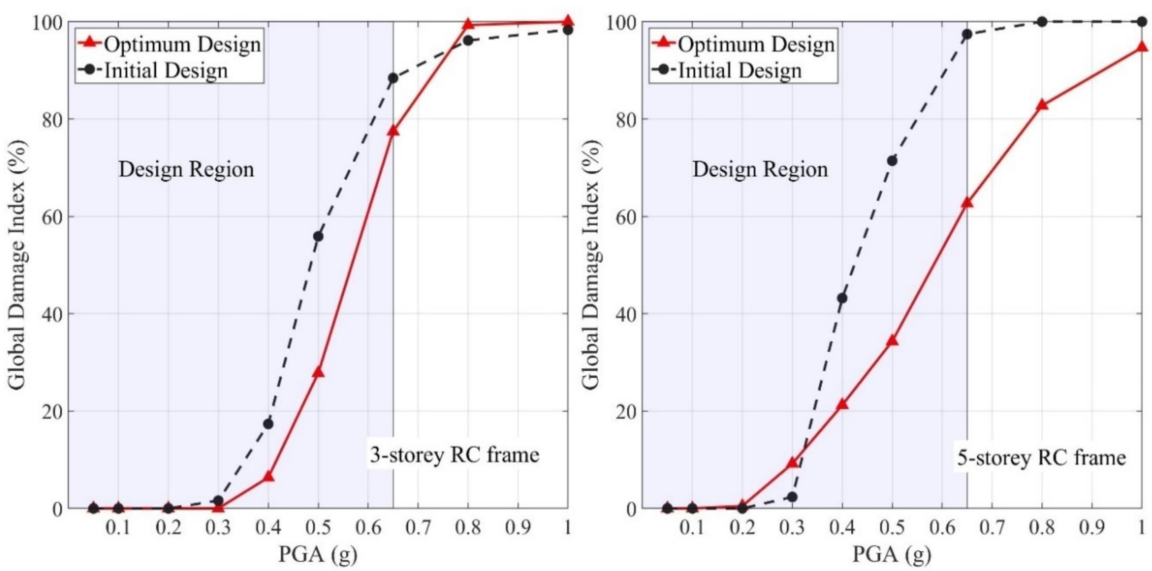


Figure 4-5: Incremental dynamic analysis (IDA), average global damage index results for initial and optimum design solutions for 3- and 5-storey RC frames, under six artificial records

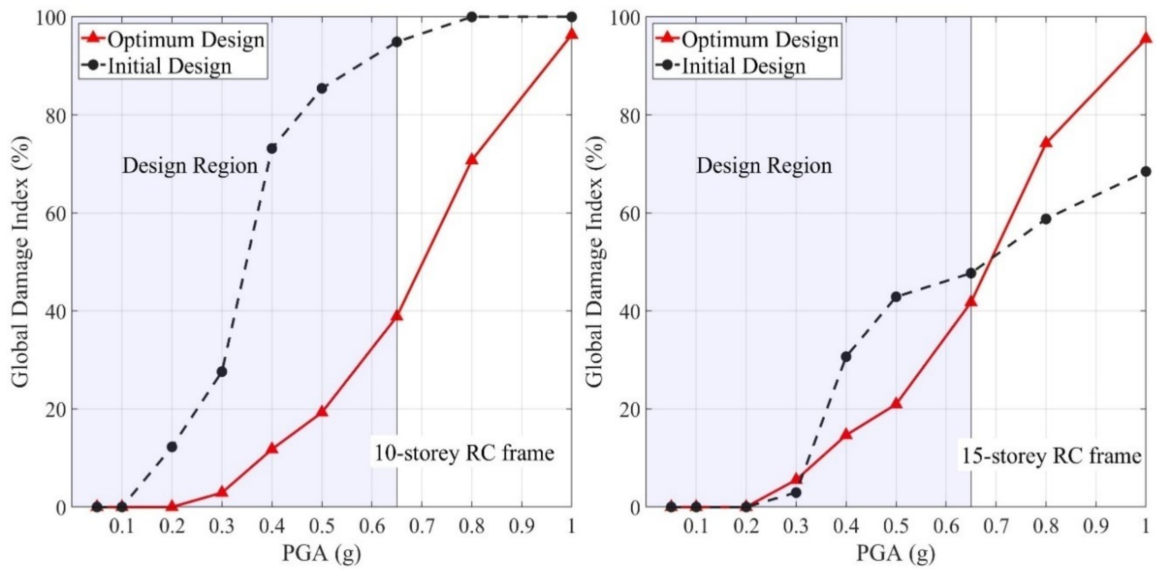


Figure 4-6: Incremental dynamic analysis (IDA), average global damage index results for initial and optimum design solutions for 10- and 15-storey RC frames, under six artificial records

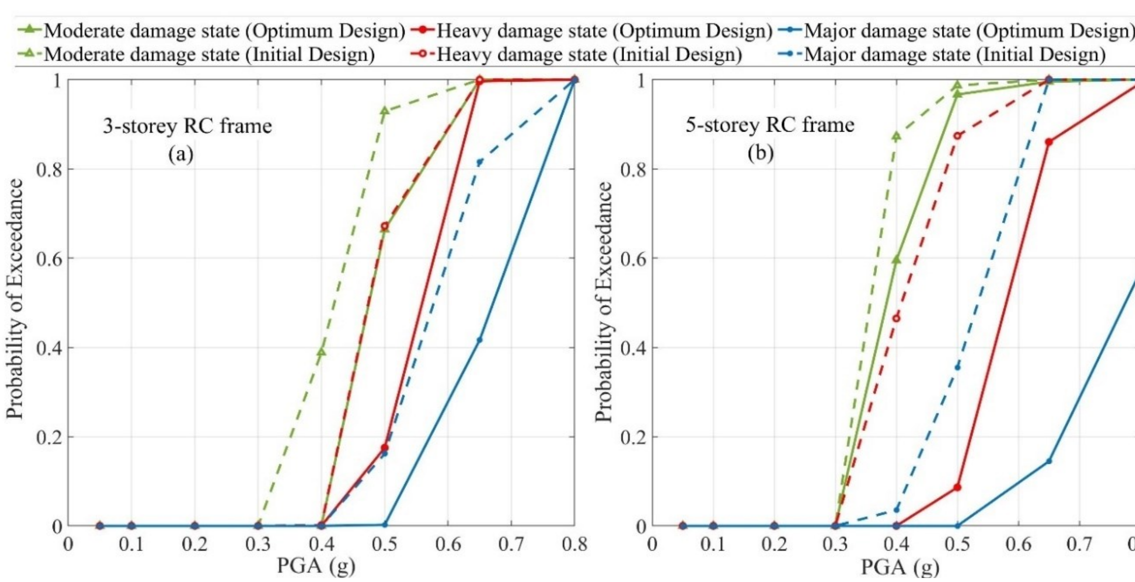
It can be seen that the optimum designs experience less overall structural damage at most of intensity levels compared to the initial code-based designs, especially when the PGA level is higher than 0.4g. The optimally designed frames achieve up to 84.5% reduction in the global damage index. For 3-, 5- and 10-storey RC frames, the initial code-based buildings suffer from severely to total damage under severe earthquakes with PGA values larger than 0.65g. In accordance with the details of structural optimisation presented in the previous chapter, the main aims of the proposed optimisation framework are to minimise total material usage while minimising structural damage through a more efficient use and exploration of material capacity in each storey. As a result, this optimisation process leads to designs exhibiting more uniform distributions of specific performance parameters (both inter-storey drift ratios and plastic hinge rotations) along storey levels. It also changes the order of occurrence of plastic hinges to avoid a sudden damage in a certain storey/element, successfully preventing “soft storey” failure. By distributing damage more uniformly, it exhibits relatively less maximum concentrated damage. Similar results can also be found in result sections in Chapter 3. It concludes that the proposed structural optimisation changes the failure mechanism of the structures.

It should be noted that, the optimum 15-storey frame experiences larger global damage index when the PGA level is higher than 0.8g, but overall it saves initial construction cost by 14.7%. More information about the initial costs, as well as total material usages for both initial and optimum designs will be discussed in the next section. Indeed, considering the design purpose that strikes a balance between multiple design objectives: “cost saving” and “minimising structural damage”, the proposed UDD optimisation method can lead to a safe and acceptable design solutions in most situations as the possibility of earthquakes exceeding PGA level equal to 0.8g is rare in reality.

4.6.2. Fragility curves for initial and optimum designs

The maximum values of the EDPs (i.e. inter-storey drift, global damage index) obtained using the IDA under six generated artificial earthquakes are utilised in the evaluation of the probability of exceedance of the specific damage states. Such the probability of exceedance is used to develop the fragility curves. Figure 4-7 shows the fragility curves for the 3-, 5-, 10- and 15-storey RC frames for three damage states (i.e. moderate, heavy, major).

The reliability and accuracy of these fragility curves are first validated by comparing them with results from a reference study by Martins and Silva (2021). In the comparison results shown in the Figure 4-8, the x-axis represents the Intensity Measure (IM), considered as Spectral Acceleration (Sa) at 0.6 seconds; the y-axis represents the Probability of Exceedance based on three damage states (i.e. slight, moderate, major). The referenced curves in the comparison are computed by considering a mid-rise RC building with a moment resisting bare frame, which is designed following modern design guidelines. The validation results illustrate that the fragility curves in this study, computed by utilising an optimum 5-storey RC frame, exhibit similar damage trends to the referenced curves along Sa for all selected damage states. However, since the buildings used to estimate both curves differ in geometry and structural properties, and the earthquake records they were subjected to in the analysis are different, the results do not completely match each other. The fragility curves in this study generally exhibit a lower probability of exceedance within the region of designed earthquake intensities (Sa (T=0.6s) < 1.88(g)), particularly considering the major damage state. This aligns with the previous conclusion that the building optimised using the proposed methodology exhibits less structural damage under multiple seismic hazard levels. Further analytical model validations can be confirmed by comparing results from eigenvalue analysis for selected RC frames modelled in SAP2000 with those in OpenSees. The results are not presented here for brevity, but they highlight the validity of the utilised structural modelling in this study.



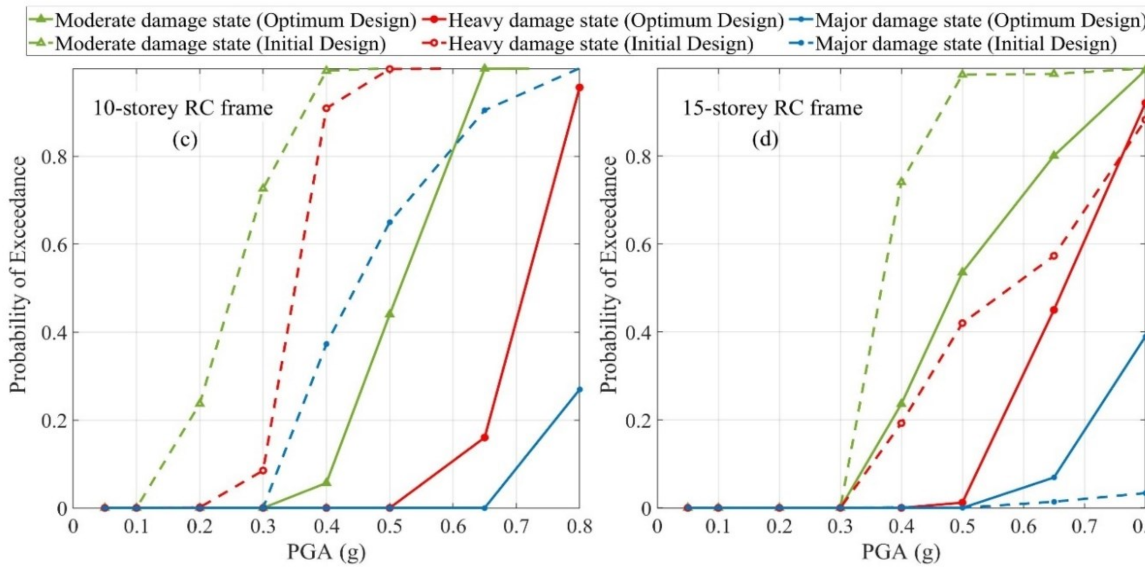


Figure 4-7: Fragility curves for initially and optimally designed (a) 3-storey, (b) 5-storey, (c) 10-storey and (d) 15-storey RC frames, average results under six artificial records, corresponding to moderate, heavy and major damage states

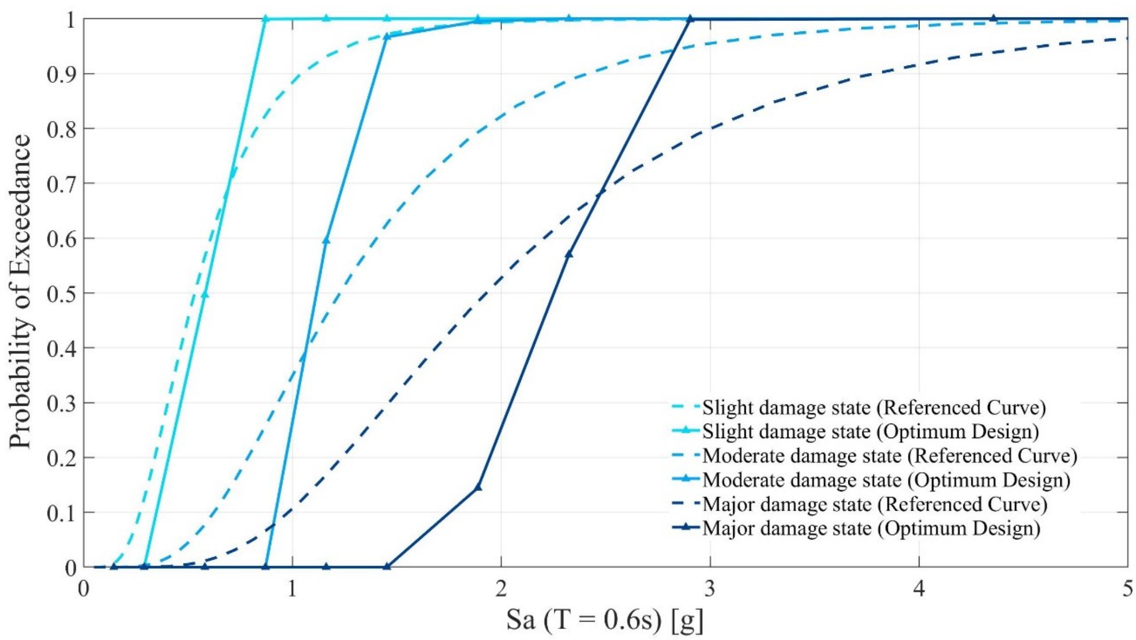


Figure 4-8: Validation of Fragility curves: comparison between a optimally designed 5-storey RC frame in this study and a mid-rise RC building in a reference study, considering three damage states (slight, moderate, major)

The fragility curves results confirm that, compared to the code-based initial designs, the optimum structures always exhibit significant reductions in the exceedance probabilities corresponding to three selected damage states, under a wider range of PGA levels (from 0.05g to 0.8g) that in reality cover most possible earthquake intensity levels.

4.6.3. Initial construction cost

The initial construction cost of the RC frames, shown in Table 4-6, is calculated based on total material usage and unit prices of materials and fabrications costs given in Table 4-2. Compared to the initial designs, the frames optimised using the proposed UDD-based optimisation method save the initial construction costs by up to 14.8%. This is because the optimum design solutions utilise more efficient materials for beam and column elements in most storeys. It should be noted that the initial design of the 3-storey frame in accordance with Eurocode 8 did not satisfy some of the performance-based constraints. Therefore, the required total materials for optimum design in this case are slightly increased (leading to more initial construction cost) to satisfy the multiple pre-determined performance objectives corresponding to different earthquake intensity levels ranging from minor to severe.

Table 4-6: Initial construction cost for optimum and initial designs

RC Frames	Initial Construction Cost (£)		Gain (%)
	Initial design	Optimum design	
3-Storey	18040	18149	-0.6%
5-Storey	32105	31434	2.1%
10-Storey	72126	61797	14.3%
15-Storey	115777	98702	14.8%

4.6.4. Total life-cycle cost

The total life-cycle cost (C_{TOT}) is composed of: (i) the initial construction cost (C_{IN}), and (ii) the expected life-cycle damage cost caused by future possible earthquakes (C_{LS}). The expected damage costs (C_{LS}) include direct losses due to structural ($C_{str-dam}$) and non-structural damage ($C_{nonstr-dam}$), indirect losses on the social cost aspects, which relate to loss of rental (C_{ren}) and commercial (C_{com}), as well as losses due to minor ($C_{min-inj}$) and major human injuries ($C_{maj-inj}$), and human fatalities (C_{fat}) after the occurrence of earthquakes. Six damage states ranging from slight to destroyed are considered in this damage cost assessment. Tables 4-7 - 4-10 summarise the calculated results of each aforementioned cost component, the expected damage cost (C_{LS}) and total life-cycle cost (C_{TOT}) for initial and optimum designs, respectively. The results show that, compared to the initial designs, the optimum designs generally save both damage repair costs and social costs due to interrupting economic activities (e.g. rental and commercial), injuries and fatalities. In conclusion, the optimum designs require less expected damage cost up to 87.1% and less total life-cycle cost up to 63.6%. The results can be justified since the proposed UDD-based optimisation framework limits structural damage by satisfying multiple performance objectives at both local and global levels, while minimising initial material costs of RC frames. The results of the fragility functions are in turn reduced for most prescribed damage states in the optimum designs.

Table 4-7: Cost components (in 100£), expected damage costs (in 100£) and total life-cycle costs (in 100£) for initial and optimum designs for 3-storey RC frames

Design Alternative	$C_{str-dam}$	$C_{nonstr-dam}$	C_{ren}	C_{com}	$C_{min-inj}$	$C_{maj-inj}$	C_{fat}	C_{LS}	C_{TOT}
Initial design	39.67	45.10	1.27	25.37	0.22	3.31	37.82	152.76	333.16
Optimum design	23.20	39.89	0.73	14.57	0.11	1.82	21.36	101.69	283.18
Reduction								**33.4%**	**15.0%**

Table 4-8: Cost components (in 100£), expected damage costs (in 100£) and total life-cycle costs (in 100£) for initial and optimum designs for 5-storey RC frames

Design Alternative	$C_{str-dam}$	$C_{nonstr-dam}$	C_{ren}	C_{com}	$C_{min-inj}$	$C_{maj-inj}$	C_{fat}	C_{LS}	C_{TOT}
Initial design	101.57	63.24	3.38	67.64	0.62	14.68	188.26	439.39	760.44
Optimum design	57.80	37.38	1.70	33.99	0.11	1.44	15.91	148.34	462.68
Reduction								**66.2%**	**39.2%**

Table 4-9: Cost components (in 100£), expected damage costs (in 100£) and total life-cycle costs (in 100£) for initial and optimum designs for 10-storey RC frames

Design Alternative	$C_{str-dam}$	$C_{nonstr-dam}$	C_{ren}	C_{com}	$C_{min-inj}$	$C_{maj-inj}$	C_{fat}	C_{LS}	C_{TOT}
Initial design	413.85	133.81	13.15	263.08	2.23	49.96	633.22	1509.3	2230.6
Optimum design	54.49	99.20	1.59	31.84	0.07	0.70	6.75	194.65	812.62
Reduction								**87.1%**	**63.6%**

Table 4-10: Cost components (in 100£), expected damage costs (in 100£) and total life-cycle costs (in 100£) for initial and optimum designs for 15-storey RC frames

Design Alternative	$C_{str-dam}$	$C_{nonstr-dam}$	C_{ren}	C_{com}	$C_{min-inj}$	$C_{maj-inj}$	C_{fat}	C_{LS}	C_{TOT}
Initial design	170.20	202.46	5.05	100.93	0.21	1.44	10.15	490.44	1648.2
Optimum design	111.47	116.84	3.25	64.99	0.20	3.11	36.24	336.09	1323.1
Reduction								**31.5%**	**19.7%**

It should be noted that previous optimisation study, which mainly considered minimum initial cost as an objective, may not necessarily lead to a more cost-effective design solution regarding total cost over the structure's lifetime, as illustrated in (Asadi and Hajirasouliha, 2020). However, the results of this study indicate that, the optimally designed structures using UDD concept can not only save the initial construction cost and experience less structural damage, but also efficiently reduce the total life-cycle, compared to the initial frames codified by Eurocode 8. In the case of 3-storey RC frame, although the initial construction cost is slightly increased by 0.6% in the optimum design, the total life-cycle cost is reduced by around 15% when multiple performance objectives are satisfied. For the 10-storey frame, initial designs exhibit typical "soft storey" failure (large concentrated maximum inter-storey drift) and large concentrated localised damage in higher stories, as shown in the Figure 3-3 and Figure 3-6, respectively. Additionally, there was significant overall structural damage, quantified by global damage index, as illustrated in the Figure 4-7. Consequently, this results in a significantly higher probability of occurrence of damage states, particularly considering severe damage (i.e. heavy, major and destroyed states), and incurred more expensive damage cost (C_{LS}).

4.7. Sensitivity of Optimum Designs to Selected Artificial and Natural Earthquakes

4.7.1. Material usage and initial construction cost

Considering unpredictable and uncertain characteristics (e.g. frequency contents and amplitudes) existed in earthquakes this section investigates the effect of the selection of different sets of earthquake records on the optimum seismic design solutions. Apart from the spectrum-compatible artificial earthquakes, a set of fifteen natural earthquake records (listed in Table 4-5) are also used in the same optimisation framework. Three alternative seismic designs are examined: (i) conventional seismic design using Eurocode 8 ("Initial design"), (ii) optimum design obtained using the average response from the six artificial earthquakes ("Mean artificial optimum"), and (iii) optimum solution obtained using the average response from fifteen natural earthquake records ("Mean natural optimum").

The total steel and concrete quantities used in both optimum solutions, and their reductions compared to Eurocode-based initial designs are summarised in Table 4-11. More details of the section properties of the Mean natural optimum designs, in terms of section dimensions and longitudinal reinforcement ratios are presented in table in Appendix D.

Table 4-11: Total concrete volumes and reinforcement steel weights for optimum and initial designs

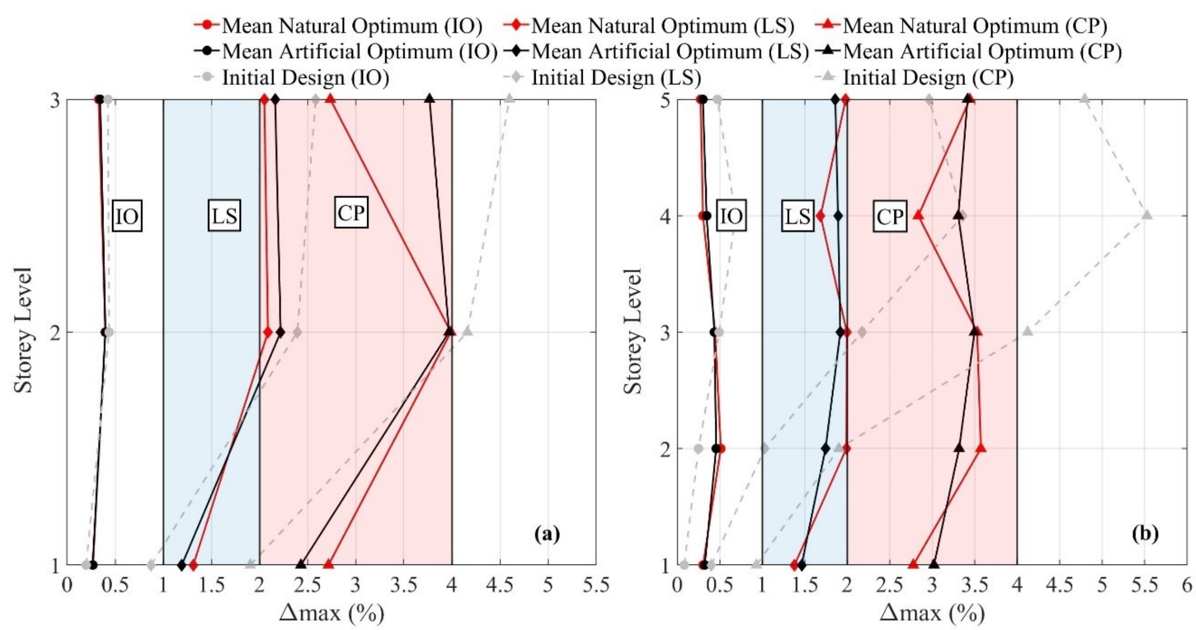
	Total Concrete Volume (m^3)			Total Steel Weight (kg)		
	Initial design	Mean artificial optimum (% reduction)	Mean natural optimum (% reduction)	Initial design	Mean artificial optimum (% reduction)	Mean natural optimum (% reduction)
3-Storey	9.66	9.72 (-0.6%)	9.72 (-0.6%)	1318.8	1373.8 (-4.2%)	1413.0 (-7.1%)
5-Storey	18.30	19.38 (-5.9%)	18.27 (0.2%)	2480.6	1428.7 (42.4%)	1452.2 (41.5%)

10-Storey	43.95	35.37 (19.5%)	35.82 (18.5%)	5809.0	3720.9 (35.9%)	4648.9 (20%)
15-Storey	72.40	58.56 (19.1%)	58.87 (18.7%)	10063.7	6413.4 (36.3%)	8540.8 (15.1%)

Compared to the code-based designs, the proposed optimisation method can always lead to design solutions with similar or less total material usages under both examined earthquake types. However, the optimum solutions obtained under the natural records can require slightly more materials, especially more steel reinforcement, than the mean artificial optimum. This can be justified as natural records generally have a wider range of variability in dynamic characteristics (i.e. amplitudes and frequency contents)

4.7.2. Seismic performance assessment

Figure 4-9 compares the response results from the fifteen independent natural earthquakes, in terms of height-wise distributions of maximum inter-storey drift ratios (Δ_{max}). As expected, both optimised designs exhibit more uniform height-wise drift distribution whilst satisfying multiple performance objectives and can lead to maximum inter-storey drift reductions of almost 50%. The design solutions obtained using the artificial ground motions occasionally violate drift-based target limits under the natural records, but the tolerance is less than 10%. This is reasonable because the responses are analysed under independent natural earthquakes that are different from the earthquakes subjected in the optimisation process for mean artificial optimum designs. It should be mentioned that, for the mean artificial optimum designs, the drift responses assessed under the utilised artificial earthquakes (here not presented for brevity) exhibit similar but more uniform distributions; and the reductions in maximum inter-storey drifts are up to 58% for selected RC frames.



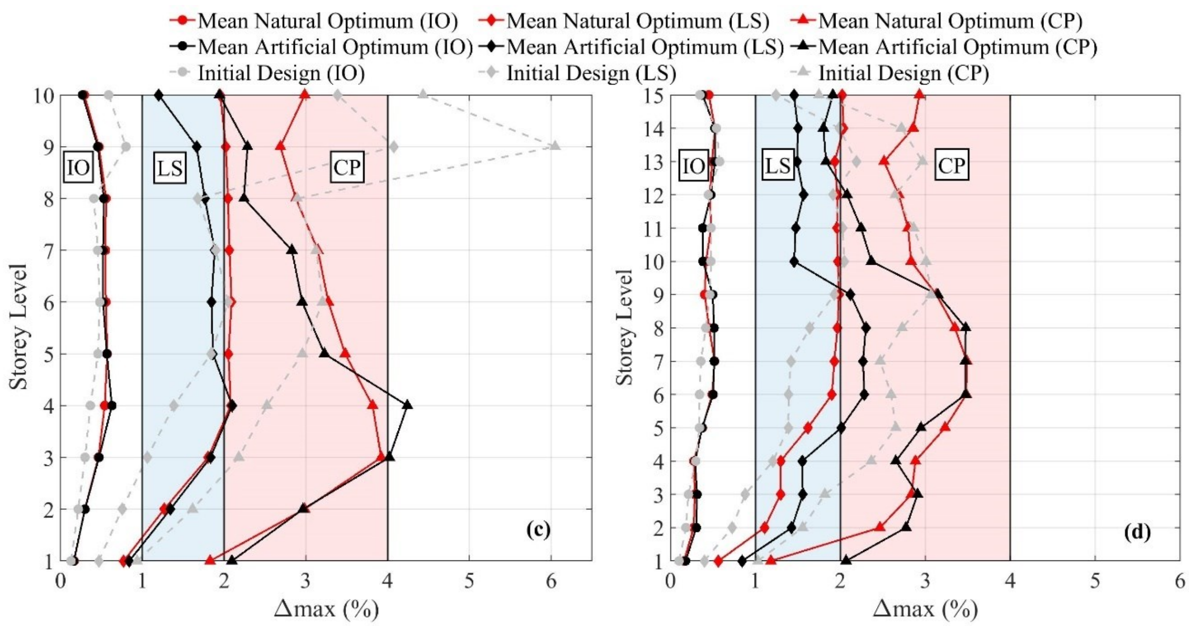


Figure 4-9: Height-wise distribution of Δ_{max} for three alterative design approaches for (a) 3-storey, (b) 5- storey, (c) 10-storey and (d) 15-storey frames, average results under fifteen natural records corresponding to IO, LS and CP performance objectives

Figure 4-10Figure 4-11Figure 4-12Figure 4-13 show the height-wise distribution of maximum plastic hinge rotation ratios (ratio of $\theta_{max,C}$ to $\theta_{target,C}$) in column elements in each storey, under the natural records corresponding to DBE and MCE levels, respectively. The results indicate plastic hinge rotation reductions of up to 60% for the optimum designs. Differences in the plastic rotation ratios between the two alternative optimum designs are more obvious on the top floors in tall buildings. This is mainly due to the higher-mode effects in the tall buildings that can be amplified by the random characteristics of the earthquake records.

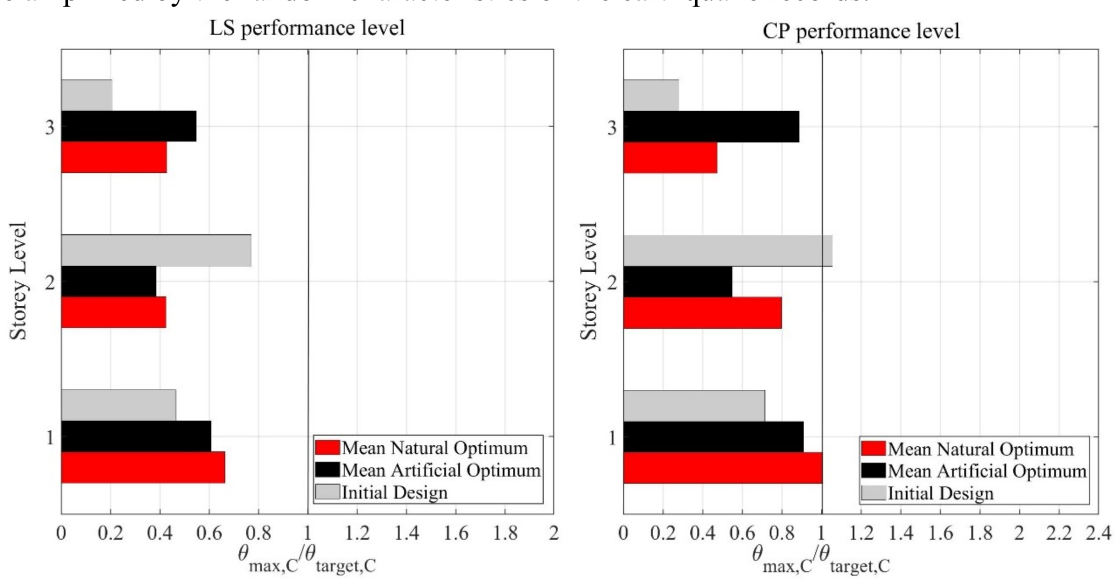


Figure 4-10: Height-wise distribution of plastic rotation ratios ($\theta_{max,C}/\theta_{target,C}$) for optimally and initially designed 3-storey frame, average results under fifteen natural earthquakes at LS and CP levels

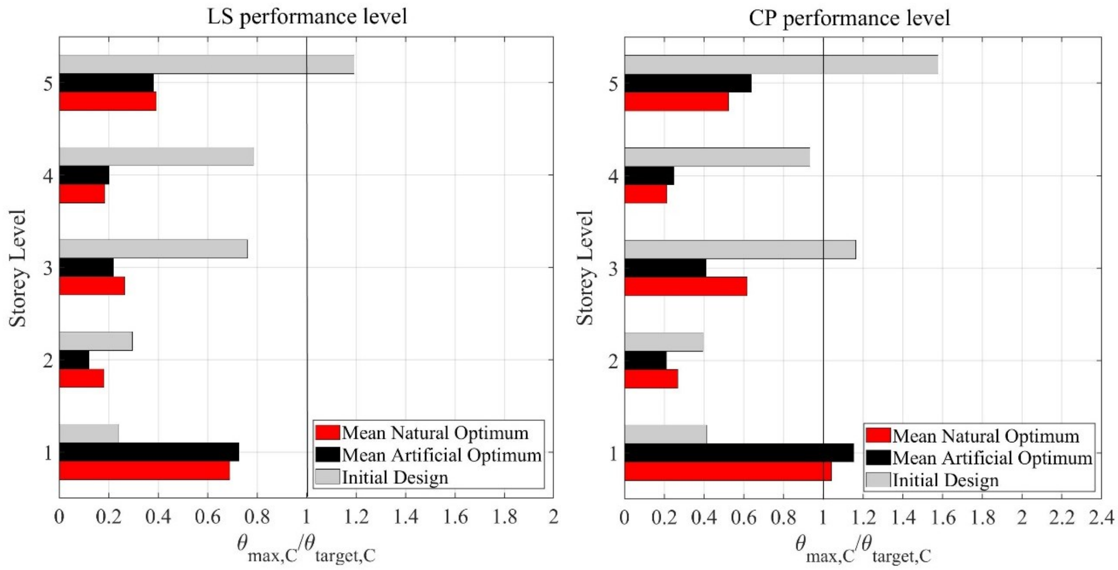


Figure 4-11: Height-wise distribution of plastic rotation ratios ($\theta_{max,C}/\theta_{target,C}$) for optimally and initially designed 5-storey frame, average results under fifteen natural earthquakes at LS and CP levels

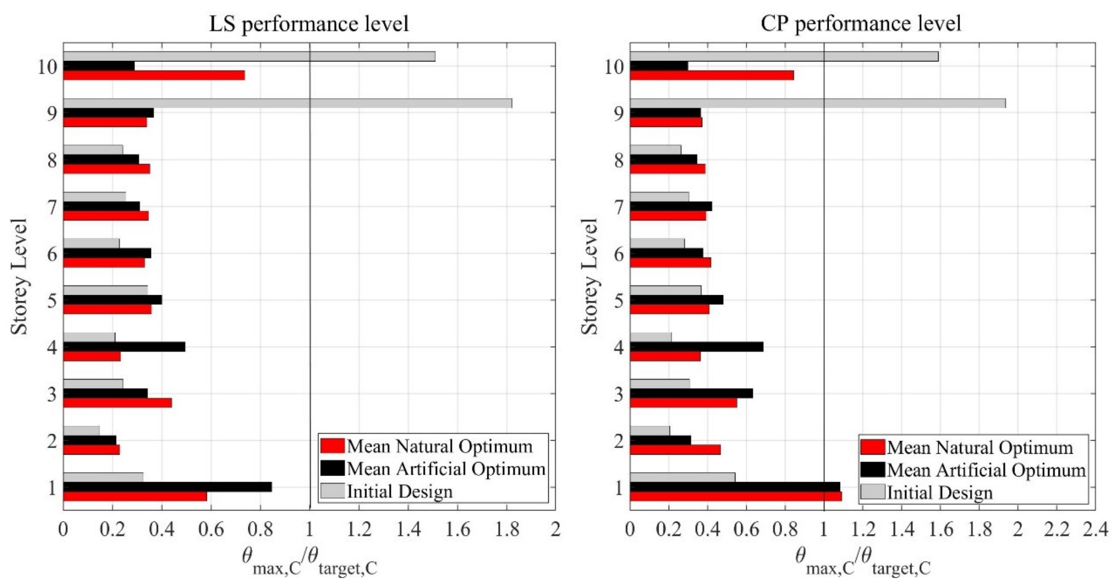


Figure 4-12: Height-wise distribution of plastic rotation ratios ($\theta_{max,C}/\theta_{target,C}$) for optimally and initially designed 10-storey frame, average results under fifteen natural earthquakes at LS and CP levels

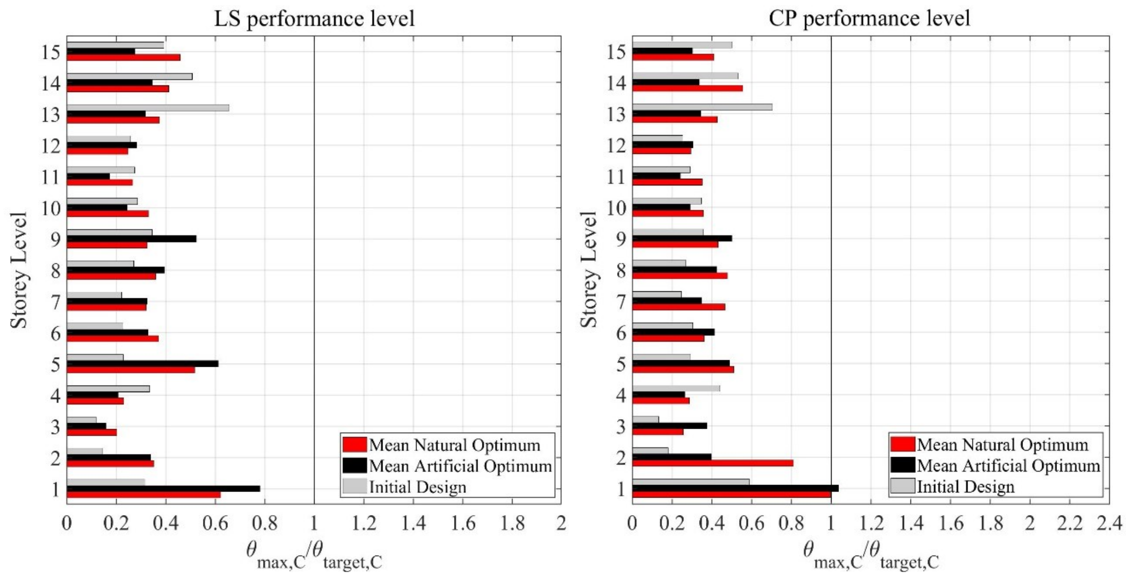


Figure 4-13: Height-wise distribution of plastic rotation ratios ($\theta_{max,C}/\theta_{target,C}$) for optimally and initially designed 15-storey frame, average results under fifteen natural earthquakes at LS and CP levels

Table 4-12 compares the average maximum plastic rotations ratios of beam elements ($\theta_{max,B}/\theta_{target,B}$) at LS and CP performance levels, respectively. The results demonstrate that the proposed UDD optimisation method is also efficient in reducing maximum plastic rotation ratios of beam elements around 40%.

Table 4-12: Maximum plastic potations ratios in beam elements for 3-, 5-, 10- and 15-storey RC frames, average results under fifteen natural earthquakes

Performance Level	Life Safety			Collapse Prevention		
RC frames	Initial design	Mean artificial optimum	Mean natural optimum	Initial design	Mean artificial optimum	Mean natural optimum
3-storey	0.92	0.78	0.73	0.91	0.79	0.58
5-storey	1.03	0.59	0.65	0.89	0.57	0.57
10-storey	0.89	0.58	0.51	0.72	0.54	0.49
15-storey	0.52	0.61	0.48	0.53	0.46	0.46

4.7.3. Local and global damage index

The previous sections have assessed structural performance (i.e. inter-storey drift ratios, plastic rotation ratios) for both initial and optimum design solutions. However, the maximum seismic response parameters itself may not accurately predict structural damage in seismic design, considering effects of cyclic loadings and structural energy absorbing capacity in the damage evaluation (Kappos, 1997; Park and Ang, 1985). To investigate the efficiency of the proposed optimisation methodology on reducing overall structural damage, Figure 4-13 presents the average values of global damage indices with the error bars representing the standard deviations of the damage index results for three alternative designs, under the fifteen natural earthquakes.

In general, the proposed UDD optimisation method is able to lead to design solutions with less global structural damage, quantified by the parameter global damage index, at both LS and CP performance levels. It can be found from the Figure 4-14 that, for the 3-, 5- and 10-storey design cases, compared to the code-based initial frames, the mean artificial optimum design solutions reduce global damage indices (DI_{global}) up to 22.3%, 31.4% and 52.6%, respectively; and the same frames optimally designed under the selected natural earthquakes experience less damage index up to 27.1%, 19.9% and 46.7%.

It should be noted that the 15-storey optimum frames in both cases slightly increase the index compared to the initial design but require less total material usages. Overall, the UDD optimisation is helpful to prevent structures from completely damaged under multiple seismic hazard levels. It should be mentioned that the global damage index histograms obtained under six artificial earthquakes (here not shown for brevity) illustrate similar answers (i.e. damage indices of the optimum designs are significantly reduced), but relatively smaller stand deviations. This is in agreement with a fact that the selected natural earthquake records have a relatively wider range of amplitudes, magnitude, and frequency contents.

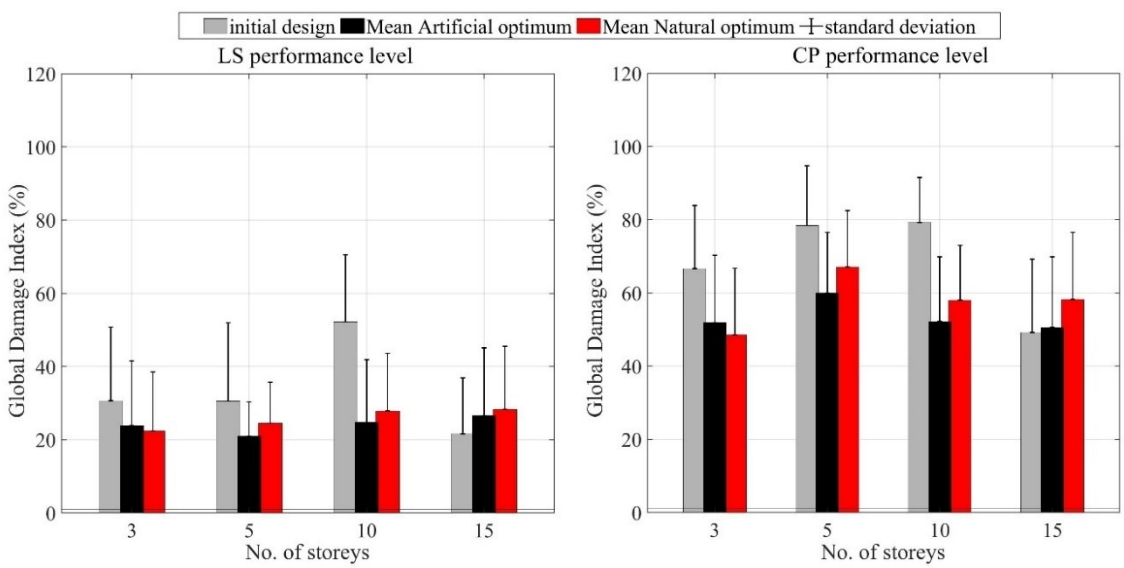


Figure 4-14: Global damage index for 3, 5, 10 and 15-storey frames at LS and CP performance levels, average (plus standard deviation) results under fifteen natural earthquakes

By using the UDD method, the optimum design solutions can be successfully obtained under either set of generated spectrum-compatible artificial earthquakes or a group of selected unscaled natural records. Meanwhile, it has observed that both optimum design solutions exhibit similar trends of damage and comparable seismic responses at local and global levels. The optimum design solutions of RC frames and their performance assessments are not significantly affected by the selection of different earthquake records once their spectrum compatibility to a target spectrum is ensured.

It is also observed that, for all the reference frames, the mean artificial optimum designs still perform satisfactory and result in better local and global seismic performances than the code-based initial structures under a group of independent natural earthquakes. Meanwhile, more economic design solutions are provided by using the artificial records, while they experience similar or even less global structural damage compared to the optimum structures obtained under the selected natural earthquakes. Therefore, the generated spectrum-compatible artificial earthquakes are recommended to be utilised as seismic inputs in the UDD optimisation process. This is especially beneficial when there is not enough data on real earthquake records for the selected site, and hence helps the practical application of the proposed optimisation framework once the target design spectrum is provided by seismic design guidelines.

4.8. Summary and Conclusions

Life-cycle cost assessment is used to investigate the economic efficiency of optimally designed RC frames that are developed using a practical multi-level performance-based optimisation method based on the concept of Uniform Damage Distribution (UDD). In this proposed UDD optimisation framework, both cross-section sizes and longitudinal reinforcement ratios are gradually optimised as design variables in elastic and inelastic phases, respectively, with a goal of fully exploiting the material capacity in each storey, which is helpful to minimise initial material costs of RC frames. Structural seismic responses at both element (i.e. plastic hinge rotations) and more structural (i.e. inter-storey drift) levels are simultaneously controlled to satisfy pre-determined performance target limits at multiple performance levels (i.e. IO, LS and CP).

The total life-cycle cost is calculated as the sum of: (i) initial construction cost including costs of material and labour, (ii) expected repair costs due to structural and non-structural damage, quantified by global damage index and maximum floor acceleration, respectively, and (iii) the corresponding social losses, including costs associated with human injuries and fatality, as well as costs associated with losses of rental/commercial incomes. In the life-cycle cost assessment, seismic performances of both initially and optimally designed 3-, 5-, 10- and 15-storey RC frames are first investigated through incremental dynamic analysis under a range of artificial earthquakes with different intensity levels. Subsequently, fragility analysis is performed to evaluate exceedance and occurrence probability of specific damage states for both initial and optimum design solutions. The efficiency of the proposed optimisation method is also demonstrated by processing the proposed design optimisation under a set of generated spectrum-compatible artificial earthquakes and selected unscaled natural records, respectively. The following can be concluded:

- Optimum designs produced up to 84.5% reductions in global structural damage compared to code-based initial solutions for 3-, 5-, 10- and 15-storey RC frames, under a wide range of earthquakes with eight different PGA values.

- Optimum solutions were also efficient in reducing limit-state probabilities (fragilities) corresponding to most of pre-determined damage states for each of selected RC frames, under a wide range of different earthquake intensity levels.

- The optimum solutions save the initial construction costs by up to 15% by utilising more efficient cross-section dimensions and reinforcement amounts than the code-based initial designs, whilst the total life-cycle costs were up to 64% lower for the optimum designs.

- It is found that, for the 3-storey design case, compared to the Eurocode-based solution, the optimum solution required slightly higher initial costs (up to 0.6%) to satisfy multiple performance targets, but it significantly reduces the total life-cycle cost by around 15%. It highlights that a design optimisation primarily aiming to minimise structural initial cost does not necessarily lead to an economical design solution when considering expected economic losses caused by structural damage under future earthquakes. However, the results indicates that effectively controlling structural performances under multiple seismic hazard levels in an optimisation framework generally results in a reduction in the total life-cycle cost.

- The UDD optimisation algorithm utilised in this study achieves a more efficient usage of material in each storey whist satisfying multiple pre-determined performance objectives, which helps eventually achieve a great balance between “initial cost” and “damage loss”. Hence, the method can be easily and practically used in future seismic design of RC frames.

- Compared to the artificial earthquakes, using natural earthquakes in the design optimisation process results in optimum designs with slightly higher total material usage. But they still require less total concrete volumes (up to 19%) and less total reinforcement amounts (up to 42%) than the code-based designs.

- A sensitivity analysis on the seismic records used show that, compared to the initial code-based designs, using the natural earthquakes results in (i) less maximum inter-storey drift ratios for mean artificial optimum (up to 48%) and mean natural optimum design (up to 49%); and (ii) less maximum plastic rotation ratios in column elements for mean artificial optimum (up to 58%) and mean natural optimum design (up to 60%). It confirms that the proposed optimisation methodology leads to similar design results under both artificial and natural earthquake sets, once the spectrum compatibility of the selected earthquake sets to a target design spectrum is ensured.

- Compared to the mean artificial optimum designs, the mean natural optimum designs generally exhibit similar or slightly larger global structural damage (up to 14%) under a set of natural earthquakes. Compared to the artificial records, using the natural earthquakes also leads to optimum designs with higher standard deviations of the damage index results, this is due to a wider range of variability in natural records, characterised by different frequency contents and amplitudes.

- Overall, it recommends using a set of spectrum-compatible artificial records in the UDD-based optimisation framework. It is especially beneficial when there are not enough real seismic ground motions records for a selected site, and it in turn helps the practical application of the proposed optimisation method.

CHAPTER 5 : Effect of Uncertainties on the Efficiency of Multi-level Performance-based Seismic Optimisation of RC Frames

5.1. Abstract

Seismic design optimisation of reinforced concrete (RC) frames is generally approached deterministically, making it difficult to assess how various sources of uncertainties impact the optimum structure. To provide optimum design solutions with acceptable seismic reliability, this study uses uncertainty analysis to investigate how uncertainties in geometry (i.e. area of longitudinal rebar in structural elements) and material properties (i.e. concrete and steel strengths) affect seismic performances of the optimised structures. And the impact of earthquake randomness on the effectiveness of the proposed optimisation framework which is constructed based on the concept of Uniform Damage Distribution is also assessed in this study. Variations in material properties and sectional details are derived using the Monte Carlo simulation method. The global damage index is chosen as the performance parameter, estimated under multiple seismic hazard levels. The effect of seismic input uncertainty is assessed by analysing the code-based and optimum structures under a group of independent natural earthquake records. The results show that: (i) the most influential variable in the sensitivity analysis is concrete compressive strength, (ii) optimum seismic designs always exhibit lower damage indexes (up to 82%) compared to conventionally designed frames, and (iii) the optimum frames exhibit less sensitivity to increased variability for uncertain parameters. For seismic input uncertainty, the optimum designs experience lower maximum inter-storey drifts (up to 49%), reduced maximum plastic hinge rotations in columns (up to 58%) and lower global damage indexes (up to 53%) than the code-based designs. These findings provide valuable insights into how different sources of uncertainties impact code-based designs and underline the effectiveness of the proposed optimisation method. This understanding contributes to enhancing the reliability of the optimisation framework for practical applications in various environments.

Prepared for submission in Journal of Engineering Structure

5.2. Introduction

Reinforced concrete (RC) frames are amongst one of the most common lateral load resistance structural systems, widely employed in the construction of multi-storey residential, commercial and public buildings, including schools and offices. The ability to provide accurate and reliable seismic design for these RC frames is closely related to the reasonable estimation of their structural seismic performance during the design phase. Previous studies have indicated that structural dynamic behaviours of both RC members (i.e. column and beam) are affected by the main structural design parameters. For example, the mechanical behaviours of the RC columns are closely related to the material strength, longitudinal reinforcement ratio, transverse stirrup ratio and axial force ratio, etc, under dynamic loading rates (Li et al., 2023). Observing the strength and deformation capacity of RC beam elements in dynamic tests, it has found that they are affected by material strength, shear span ratio and transverse stirrup ratio (Adhikary et al., 2014; Fujikake et al., 2009). However, in practice, sectional properties vary due to tolerances, and material properties also vary and change over time period due to maturity and deterioration. As a result, predicting the seismic performance of structures is highly uncertain, not only due to the randomness of the imposed seismic loads, but also due to the uncertainty in determining their seismic capacity.

Numerous previous studies examined the sensitivity of the performances of RC frames to several major uncertain variables under seismic loads. Kwon and Elnashai, (2006) found that material uncertainty (i.e. variability in concrete and steel strength) affects the seismic vulnerability curves of RC structures especially at high ground motion levels but is less significant than the uncertainty in seismic input. In uncertainty study by Porter et al. (2002), where the author indicated that overall economic performance (represented by building's damage repair cost) of a high-rise RC frames is primarily sensitive to uncertainties in assembly capacity to resist seismic damage, seismic ground motion intensity.

The significant contribution of the seismic input variability on uncertainty in performance of RC structures has been confirmed in the aforementioned literatures and in study by Wen, (2001). Many previous seismic designs use a group of earthquake records to assess seismic performance, effectively capturing and addressing variability in the earthquake records. (De Domenico and Hajirasouliha, 2021; Hajirasouliha et al., 2012). However, the structural model utilised in these design cases is typically deterministic, where mechanical properties of reinforcement steel and concrete are normally dealt as deterministic parameters, their median values are normally utilised and kept constant during the design process. Such a model cannot accurately account for the uncertainty in material strength and geometrical parameters. Through sensitivity analysis, previous studies have demonstrated that uncertainties in material properties, sectional properties and modelling parameters, as separate uncertain variables, have substantial impacts on the seismic performance assessments of RC frames (Mohamed et al., 2023; Celarec and Dolšek, 2013; Asprone et al., 2012). Segura et al. (2022) also confirmed the significant effects of vibrations in mechanical properties of concrete and reinforcing steel on the evaluation of nonlinear seismic responses of RC bridge column structures. Therefore, for reinforced concrete structures, ignoring inherent variability in the mechanical properties of steel and concrete in the analytical model may result in inaccurate seismic performance evaluation and also unreliable seismic design solution.

Furthermore, the effect of variability in construction quality was studied by Rajeev and Tesfamariam, (2011), where construction quality was categorised into three levels: poor, average and good. The conclusions show that material and structural detailing uncertainties influence significantly structural fragility curves of RC frames under multiple earthquake hazard levels. Additionally, Kim et al. (2020) examined the effects of uncertainties in concrete strength, steel yield strength, longitudinal reinforcement ratio, and volumetric ratio of transverse reinforcement on the seismic vulnerability of RC frames. The results revealed that uncertainty in concrete strength played the most crucial role in influencing structural seismic vulnerability at a global level, but local vulnerability (shear) was more sensitive to the volumetric ratio of stirrups. The findings in this study also indicated that (i) consideration of uncertainties in material strengths (i.e. concrete compressive strength, yield strength of reinforcing steel) is essential for accurate assessment of structural performance under seismic loads, (ii) the inclusion of design detailing uncertainties, including the spacing of confinement reinforcement and longitudinal reinforcement ratio, significantly impacts the shear and flexural strength of RC members.

Past seismic events have provided valuable insights, highlighting that the construction quality can affect structural performance. For instance, the 1999 earthquake in Kocaeli, Turkey (Mw=7.4) led to the collapse of many residential and commercial RC buildings; these collapses were primarily attributed to inadequate reinforcement detailing in RC columns and beam-column joints, poor construction practices and the continued use of nonductile seismic detailing (Sezen et al., 2003). In a recent seismic event in Luding, China 2022 (Mw=6.8), most RC buildings, constructed in compliance with modern seismic design codes, successfully avoided collapse. Nonetheless, many exhibited unrepairable structural damage. It was observed that this damage was primarily caused by poor concrete quality, insufficient reinforcement detailing, and improper design to meet “repairable” design requirements under the design basis earthquake (DBE) level (Qu et al., 2023). These past experiences confirmed that ignoring uncertainties stemming from the construction quality may lead to unsafe designs, especially under major to severe earthquakes.

Previous studies on seismic design optimisation of RC frames generally address uncertainty effects by incorporating reliability constraints that express structural performance in probabilistic forms in the optimisation formula (Seify Asghshahr, 2021; Gholizadeh and Aligholizadeh, 2019; Zou et al., 2018; Yazdani et al., 2017; Khatibinia et al., 2013; Mitropoulou et al., 2011). In (Gholizadeh and Aligholizadeh, 2019), the Monte-Carlo simulation method was employed to construct the reliability constraints, considering uncertainties in both material properties and seismic loading. The study reported that, compared to a reliability-based optimum design, the deterministic optimisation approach resulted in highly vulnerable designs with higher non-performance probabilities at multiple performance levels. Khatibinia et al. (2013) proposed a reliability-based design optimisation framework to quantify effects of inherent uncertainties in material properties of concrete, steel and soil, as well as in earthquake characteristics, on the seismic behaviour of RC frames. The results show that the use of reliability constraints is helpful to avoid design solutions with unexpected failure probabilities. A study by Hajirasouliha et al. (2016) quantified and managed the effect of uncertainties in storey shear-strength, damping ratio, and seismic ground motion on the seismic performance of both optimum and conventionally designed braced steel frames. The results confirm that sources of uncertainties generally exerted significant impact on both code-based and optimum design solutions.

The primary objective of this study is to investigate the impact of variability in construction quality and randomness in earthquake records on the seismic performance of conventionally and optimally designed RC frames. Optimum designs for a set of 3-, 5-, 10- and 15-storey RC frames are determined using an optimisation method based on the concept of Uniform Damage Distribution (UDD). In accordance with past studies, the key parameters for construction quality include: concrete compressive strength, yield strength of reinforcing steel, and cross-section area of rebar. This chapter begins by investigating the sensitivity of seismic performance due to variations in these uncertain parameters and their combined effect on the efficiency of optimum designs, using the Monte Carlo simulation method with an appropriate sample size. The global damage index is used as the seismic performance parameter, calculated based on deformation in each storey through non-linear time history analysis. Furthermore, the impact of uncertainty in seismic ground motions is assessed by estimating both local and global seismic responses for code-based and optimum frames under a set of independent earthquake records, which differ from the earthquakes used in the optimisation process.

5.3. Implementation of the Uncertainty Analysis

The Monte Carlo Simulation (MCS) method adopted here is a widely accepted and reliable technique for conducting uncertainty analysis in optimisation processes (Gholizadeh and Aligholizadeh, 2019; Möller et al., 2015; Khatibinia et al., 2013). The MCS method generates N Monte Carlo samples representing N possible instances of the structures, where a set of N random values is created from the specific probability distribution type of the selected uncertainty variables. These generated uncertainty variables are considered as inputs to perform N nonlinear time history analyses and evaluate the performance results as outputs. To assess the effect of uncertainties on the structural seismic performance (i.e. structural global damage index), the average performance results over all MC samples are utilised in the analysis. The sample size N is determined by investigating the effect of sample size on performance sensitivity, as detailed in the following sections. MCS generally requires a large number of random samples (MC samples). It is essential to determine an appropriate sample size that strikes a balance between result accuracy and computational efficiency before processing the uncertainty analysis.

In this section, the different uncertainty variables used are first summarised, including their statistical parameters. The details on generating MC samples based on the random values are then explained. And the specific performance parameter is selected, its evaluations including detailed formulas are introduced in the following section.

Step 1: Classification of the uncertain variables

This study first investigates the effects of uncertainties associated with material and sectional properties, which are linked to construction quality uncertainty. Based on past experiences with buildings that suffered damage during seismic events, it has been observed that most of the affected constructions exhibited inadequate seismic resistance, mainly due to the use of low-quality materials and insufficient seismic design detailing of steel reinforcement (Feliciano et al., 2023; Qu et al., 2023; Sezen et al., 2003). Moreover, several previous studies have shown that the construction quality variability can be reasonably quantified by altering material strength and adjusting structural detailing, in terms of details of transverse and longitudinal reinforcement (Kim et al., 2020; Tesfamariam et al., 2013; Rajeev and Tesfamariam, 2011). Therefore, in this study, material uncertainty is represented by the compressive strength of concrete (f_c') and yield strength of reinforcement steel (f_y). Besides, the sectional property uncertainty is represented by the area of longitudinal reinforcement in both beam and column elements (A_s). Since the proposed optimisation framework mainly focuses on flexure failure modes, it assumes that both initial and optimum designs have adequate amount of shear reinforcement. Thus, uncertainty in transverse reinforcement, such as tie spacing variability, is not considered in this uncertainty analysis.

The statistical parameters describing the variability of the selected variables are summarised in Table 5-1, which characterises their probabilistic distribution type and the corresponding coefficient of variation (COV). These parameters are determined in accordance with results from previous experimental and numerical tests (Badalassi et al., 2017; De Stefano et al., 2001; Lu et al., 1994). Similar COV values for the selected uncertainty variables are also adopted in most recent study (Hariri-Ardebili et al. 2024). To account different uncertainty levels that represent levels of variability of uncertainty variable and cover most possible uncertainties during the construction practice, the different $COVf_c'$ can vary from 0.075 to 0.3, the $COVf_y$ can vary from 0.015 to 0.06, and the $COVA_s$ can vary from 0.02 to 0.08 in this study. This study assumes that either the uncertain material properties or uncertain design detailing parameters (i.e. area of longitudinal reinforcement) in all structural elements of an RC frame are described by the same probability distribution in the Monte Carlo estimation. This has been effectively employed in previous studies (Celarec and Dolšek, 2013; Celik and Ellingwood, 2010) to reduce computational costs with satisfactory accuracy.

The design compressive strength of concrete (f_c') used in the original design optimisations and in the code-based designs is 30 MPa; the design yielding strength of reinforcement steel (f_y) used in the original design optimisations and in the code-based designs is 500 MPa. Detailed design specification for the longitudinal reinforcement steels in both code-based and optimum solutions are provided in the tables in Appendix B and C.

Table 5-1: The parameters of input uncertain variables of the structural models, the corresponding probability distribution and coefficient of variation

Uncertainty sources	Variable	Distribution	COV	Reference
Material strength	Concrete strength (f_c')	Normal	0.15	(Badalassi et al., 2017)
	Steel strength (f_y)	Normal	0.03	(De Stefano et al., 2001)
Sectional property	Area of longitudinal rebar (A_s)	Normal	0.04	(Lu et al., 1994)

Step 2: Generation of Monte Carlo samples

It is assumed that the selected uncertainty variables, as shown above, vary independently from each other but uniformly throughout the structure with the same uncertainty level, characterised using COV values. In other words, when considering the impact of the uncertainty on the efficiency of the UDD-based optimisation methodology, in terms of the random combination of the three predetermined uncertainty variables, 3N random values are generated for these variables, while N non-linear time history analysis are processed for N samples for each specific seismic hazard level.

Random uncertainty variable in each MC sample is generated based on the random value, using the following equations:

$$f_c' = f_{c,0}' \times \left(1 + N_{R,f_c'} \times COVf_c'\right) \tag{5.1}$$

$$f_y = f_{y,0} \times \left(1 + N_{R,f_y} \times COVf_y\right) \tag{5.2}$$

$$A_s = A_{s,0} \times \left(1 + N_{R,A_s} \times COVA_s\right) \tag{5.3}$$

where f_c', f_y and A_s are the randomised values of concrete strength, steel yield strength and cross-sectional areas of longitudinal steels, respectively. $f_{c,0}'$, $f_{y,0}$ and $A_{s,0}$ are the original design details on the concrete strength, steel yield strength and cross-sectional areas of longitudinal steels, respectively. $N_{R,f_c'}$, N_{R,f_y} and N_{R,A_s} represent three independent standard normal (Gaussian) distributed random values assigned to the uncertainty variables f_c', f_y and A_s, respectively. $COVf_c'$, $COVf_y$ and $COVA_s$ are the Coefficient of Variation (COV) for the variables f_c', f_y and A_s, respectively.

It is expected that the average and standard deviation of the normal distributed random numbers ($N_{R,f_c'}$, N_{R,f_y} or N_{R,A_s}) over all MC samples are approximately equal to 1 and 0, respectively. Thus, the mean value of each selected uncertainty variable among all generated samples equals the default value that is originally utilised in a structural modelling. These equations have been successfully utilised in the previous study for braced steel frames (Hajirasouliha et al., 2016).

Step 3: Seismic performance evaluation

A damage index involving a displacement-based ductility ratio is adopted as the performance parameter, its average value among all generated MC samples is calculated as output value in this uncertainty study. The average result is considered here because it helps minimise errors in predicting structural seismic responses across all generated MC samples. Since the sum of the deviations of each random value from the mean value is approximately equal to zero, and the uncertainty variables are assumed to be perfectly normally distributed. The mean and median values of the uncertainty variables are almost identical in this case. It is reasonable to consider the average seismic response as the central tendency in the sample set. Additionally, the median response value may not accurately capture extremely large or small seismic performances due to the effect of selected uncertainty sources.

In this uncertainty analysis, the structural seismic performances are evaluated under two different seismic hazard levels, namely: (i) Design Basis Earthquake (DBE), defined as a 10% probability of exceedance in 50 years, and (ii) Maximum Considered Earthquake (MCE), defined as a 2% probability of exceedance in 50 years. PGA is considered as the parameter to represent earthquake intensity here, because it primarily relates to earthquake characteristics and is not affected by structural properties (e.g. stiffness and mass) in different design solutions. It is more suitable for the comparison purpose between code-based and optimum designs. In accordance with the relationship between the seismic hazard level and corresponding PGA value as indicated in Table 3-1 in Chapter 3, the magnitude of PGA is decided as 0.4g and 0.65g at DBE and MCE levels, respectively.

The damage index in each storey level (D_i) is first evaluated based on the maximum inter-storey drift that can help quantify damage in both structural and non-structural components, following the "demand versus capacity" concept as suggested by (Powell and Allahabadi, 1988):

$$D_i = \left(\frac{\delta_c - \delta_t}{\delta_u - \delta_t}\right)^b \tag{5.4}$$

where constant parameter b is determined based on experimental data, suggested as 1.5 for reinforced concrete frames (Cosenza and Manfredi, 2000), δ_c, δ_t and δ_u are maximum inter-storey drift ($\Delta_{max,i}$), yielding drift ($\Delta_{yield,i}$) and ultimate drift capacity ($\Delta_{ultimate,i}$) in i^{th} storey, respectively. $\Delta_{max,i}$ is evaluated by using the non-linear time history analysis. $\Delta_{yield,i}$ is estimated through pushover analysis, where a monotonically increasing load is applied only to the i^{th} storey while ensuring all nodes below this storey are fixed. This approach eliminates uncertainty arising from the applied fixed lateral load pattern (e.g. triangular or uniform load pattern). The applied load is increased in a displacement-controlled manner until the observed displacement in a certain storey reaches its pre-determined maximum value. A bilinear capacity curve, based on the rule of “same energy absorption”, is further constructed using results from the pushover analysis to evaluate the $\Delta_{yield,i}$. $\Delta_{ultimate,i}$ is assumed as target limiting value of inter-storey drift corresponding to “collapse prevention” performance level according to performance-based seismic design guidelines (e.g. ASCE/SEI 41-06, (2007)).

The global damage index (D_g) is evaluated as weighted average of damage indexes (D_i) at multiple storey levels, while the weight (w_i) depends on amount of energy dissipation in the i^{th} storey:

$$D_g = \frac{\sum_{i=1}^{N} D_i w_i}{\sum_{i=1}^{N} w_i} \quad (5.5)$$

where N is the total number of storeys. w_i is the weight factor for i^{th} storey. The global damage index (D_g) ranges from 0 (undamaged) to 1 (severely damaged).

5.4. Analytical modelling and assumption

5.4.1. Selected reinforced concrete frames

In this study, 3-, 5-, 10- and 15-storey RC frames were selected as referenced structures, their geometry details and optimum cross-sectional detailing are presented in Figure 5-1. All the selected frames have a storey height of 3m, and three bays with a uniform width of 5m. These buildings were assumed to be situated on soil type C, as specified in Eurocode 8 (CEN, 2004a), and were designed to represent residential structures (important class I) in regions with medium to high seismic activity. In addition to seismic loads, the buildings were also designed to withstand constant dead and live loads. Specifically, the intermediate storeys were subjected for dead and live loads of 4.6 kN/m2 and 2 kN/m2, respectively, while the roof was designed for dead and live loads of 4 kN/m2 and 0.7 kN/m2, respectively. More details on the designs using the Eurocode 2 and 8 (CEN, 2004a, 2004b) and the design optimisations based on the concept of uniform damage distribution, they can be found in author’s previous paper (Dong et al., 2023). A summary and comparison of the total concrete volume and reinforcement steel weight for both initial and optimum designs are provided in Table 5-2.

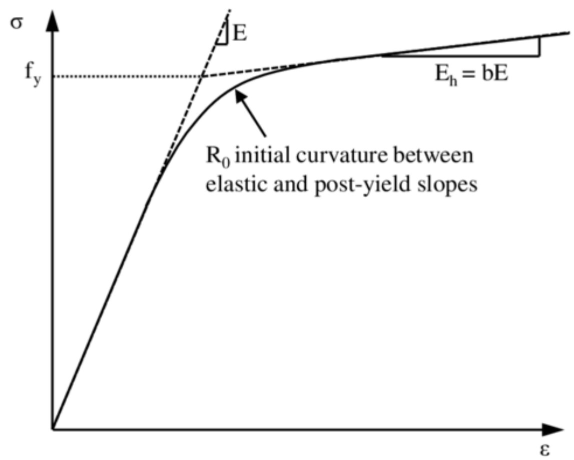
Figure 5-1: Geometry and dimensions of optimum beam and column members of 3-, 5-, 10- and 15-storey RC frames (Beams: "height × width"; Columns: "square dimension")

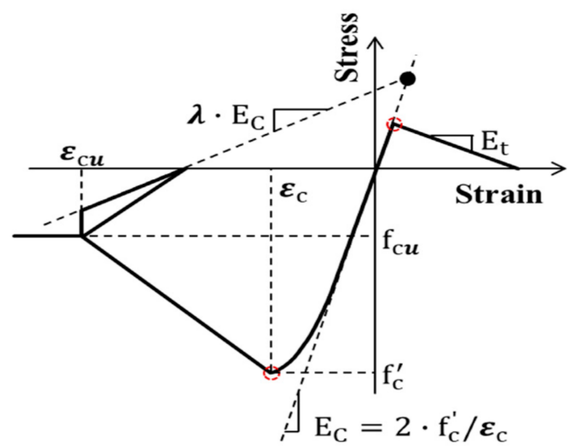
Table 5-2: Total concrete volume and reinforcement weight for initial and optimum designs

RC Frame	Total Concrete Volume (m3)			Total Reinforcement Steel Weight (kg)		
	Initial design	Optimum design	Gain (%)	Initial design	Optimum design	Gain (%)
3-storey	9.66	9.72	-0.5%	1318.8	1373.7	-4.2%
5-storey	18.30	19.38	-5.9%	2480.6	1428.7	42.4%
10-storey	43.95	35.14	20.0%	5809.0	3697.3	36.4%
15-storey	72.40	58.56	19.1%	10063.7	6413.4	36.3%

Numerical models of the RC frames were implemented using the finite element software OpenSees (McKenna et al., 2006). "Concrete02" and "Steel02" stress-strain models were utilised for concrete and steel, respectively. As shown in Figure 5-2**,** the concrete model fully expressed stress-strain relations for both confined and unconfined concretes (Mander et al., 1988; Mohd Yassin, 1994). The steel model (Giuffre-Menegotto-Pinto) incorporates several important non-linear characteristics of the reinforcement steel (Filippou et al., 1983). Beam and column elements were modelled by employing with force-based beam-column element ("*forceBeamColumn*") with the "Modified Gauss-Radau" integration method (Scott and Fenves, 2006). The element model assumes that the plasticity is distributed within the specific plastic hinge region whose physical length can be evaluated in accordance with the diameter of reinforcement in beams (d_{bL}), concrete strength (f_c') and steel strength (f_y).

P-Delta effects were taken into account in the analysis processes, Rayleigh damping with a constant ratio of 5% was assigned to the first mode and any other mode whose cumulate mass participation exceeds 95%. More details on the analytical model in this study can be found in the author's previous papers (Dong et al., 2023).





(a) (b)

Figure 5-2: Stress(σ)- strain(ε) curves for OpenSees models: (a) *Steel02* (Limbert et al., 2021); (b) *Concrete02* (Kim et al., 2020)

5.4.2. Earthquake ground motions

The effect of material and geometrical uncertainties is investigated in isolation by evaluating performance under a single artificial earthquake record (i.e. SIM02) whose elastic response spectrum has the closest approximation to Eurocode 8-based design response spectrum over a range of periods that encompasses fundamental periods of the four selected RC frames, as shown in Figure 5-3.

To investigate the additional effect of uncertainty in seismic input, fifteen independent natural earthquake records with a wide range of frequency contents and amplitudes are selected from the Pacific Earthquake Engineering Research Centre (PEER) online database and the SIMBAD database (Smerzini et al., 2014). The elastic response spectra of the un-scaled natural earthquakes, the mean response spectrum and Eurocode-based design spectrum are presented in Figure 5-4. Table 5-3 summarises the No. and characteristics of the selected natural records. For each specific earthquake level, the chosen artificial and natural earthquake records are simply scaled to achieve the desired magnitude of PGA.

Table 5-3: Characteristics of the independent natural earthquakes

Natural EQ No.	Earthquake	Station	Mw	PGA (g)
1	1990 Manji Abbar	MANJIL/ABBAR-T	7.4	0.496
2	1992 Cape Mendocino	CAPEMEND/PET000	6.9	0.590
3	1999 Duzce	DUZCE/DZC270	7.2	0.535
4	1979 Imperial Valley	IMPVALL/HE04140	6.5	0.485
5	1995 Kobe Hyogo	JMA/y	6.9	0.832
6	1999 Kocaeli	KOCAELI/DZC270	7.5	0.356
7	2005 NW Off Kyushu	FKO006/y	6.6	0.279
8	1989 Loma Prieta	LOMAP/G03000	6.9	0.555
9	1989 Loma Prieta	LGPC/x	6.9	0.531
10	2007 Niigata prefecture	NIG018/x	6.6	0.506
11	1994 Northridge	ST_24279/x	6.7	0.583
12	1994 Northridge	NORTHR/NWH360	6.7	0.590
13	2000 South Iceland	ST_109/y	6.4	0.706
14	1987 Supersition Hills	SUPERST/BICC000	6.7	0.358
15	2000 Tottori Prefecture	TTR009/y	6.6	0.611

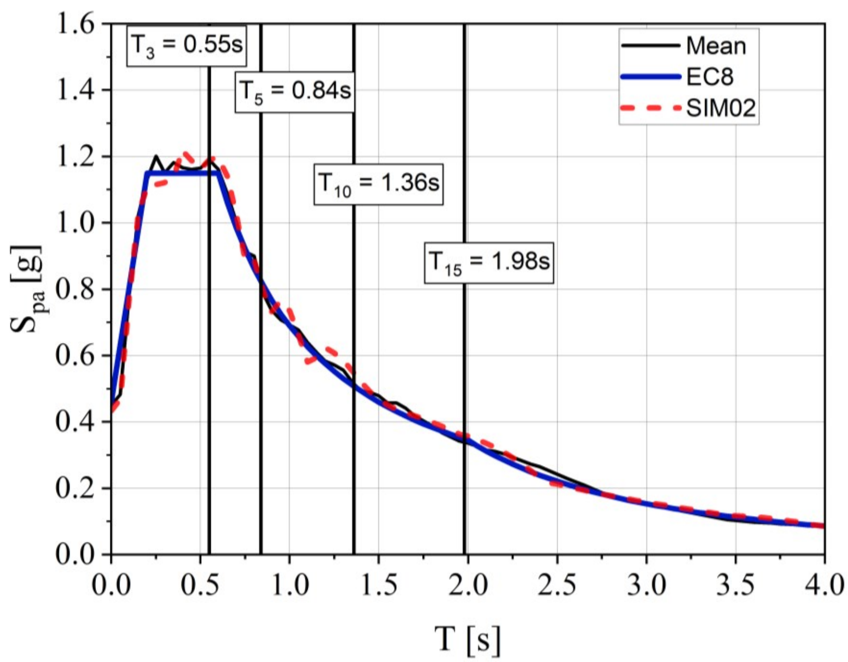


Figure 5-3: Eurocode 8-based design response spectrum and acceleration spectra of SIM02 artificial record

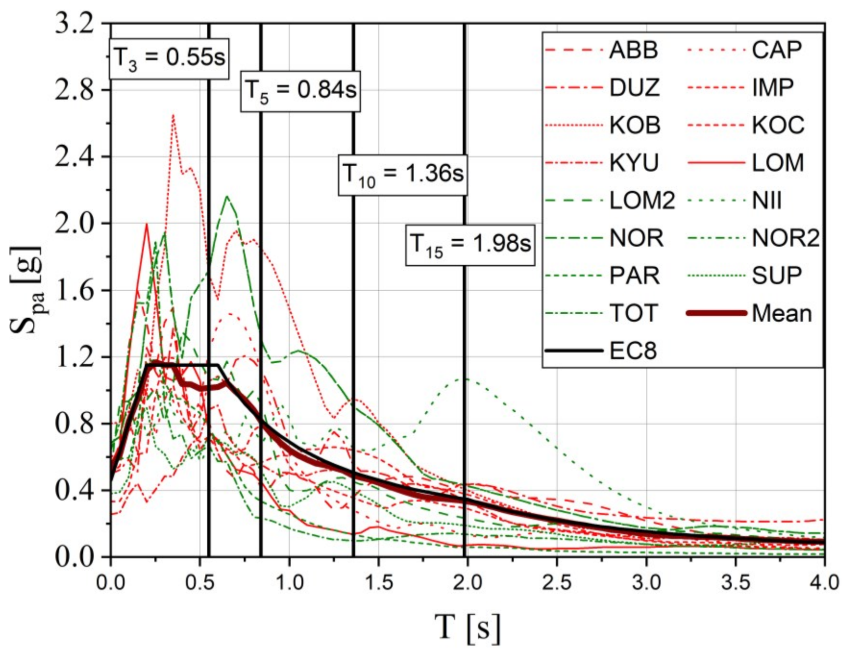


Figure 5-4: Eurocode 8-based design response spectrum and acceleration spectra of fifteen natural records

5.5. Sensitivity analysis

5.5.1. Sensitivity to sample size

To investigate the effect of sample size in the MCS process, Figure 5-5 presents the average and standard deviation of the global damage index (D_g) for a 5-storey optimally designed RC frame as it varies with different sample sizes (ranging from 10 to 150 samples). The non-linear time analysis was conducted for a single artificial earthquake record, SIM02, at a specific intensity level (i.e. DBE level). Random combinations of the selected three independent uncertain variables are considered as input in each generated MC sample, with COV values of 0.15 for f_c', 0.03 for f_y and 0.04 for A_s.

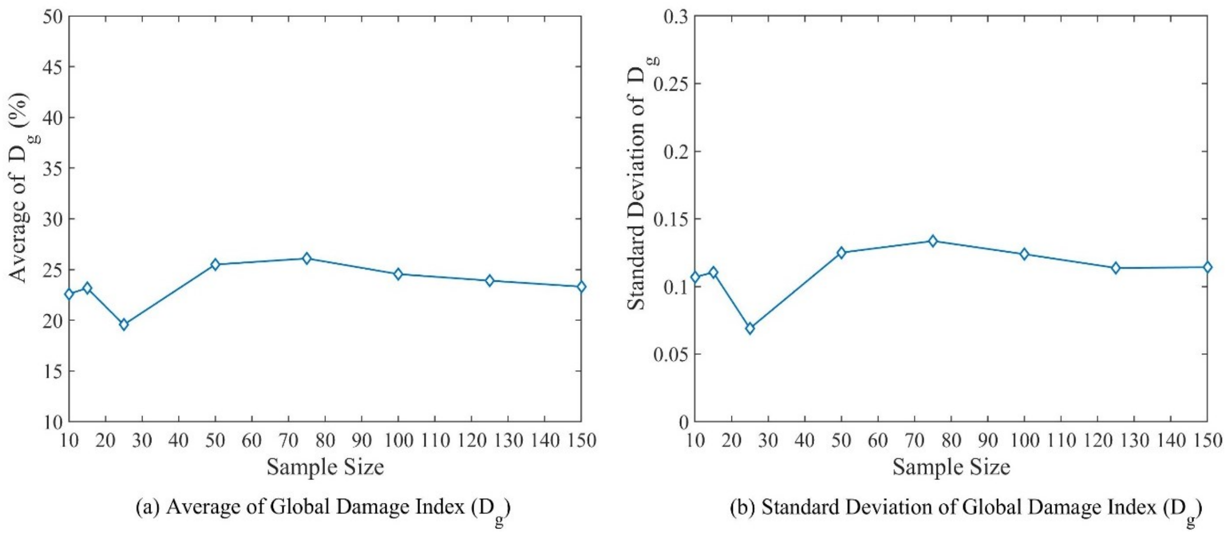


Figure 5-5: Effects of sample size on the seismic performance of 5-storey optimally designed frame subjected to an artificial earthquake

The results indicate that at least 125 MC samples are required to ensure the accuracy and reliability of the analysis results and avoid obvious fluctuation. Therefore, 125 MC samples are used in the following analyses.

5.5.2. Sensitivity to different design variables

In this section, sensitivity analysis is employed to explore the impact of concrete strength, steel strength and rebar area on the performance-sensitivity, respectively, under earthquakes with multiple intensity levels. The uncertainty levels, defined using four COV to represent construction quality ranging from good to poor, are 0.075, 0.15, 0.225 and 0.3 for concrete strength, 0.015, 0.03, 0.045 and 0.06 for steel strength, and 0.02, 0.04, 0.06 and 0.08 for the cross-sectional area of reinforcement.

The sensitivity analysis utilises the average and the standard deviation of the D_g for both code-based and optimum 5-storey RC frames under DBE and MCE levels, respectively. As mentioned earlier, the maximum value of D_g is limited to 100%. The results shown in Figures 5-6, 5-7 and 5-8, indicate the average (Ave.) and average plus standard deviation (Ave.+ std) of the global damage index at each uncertainty level.

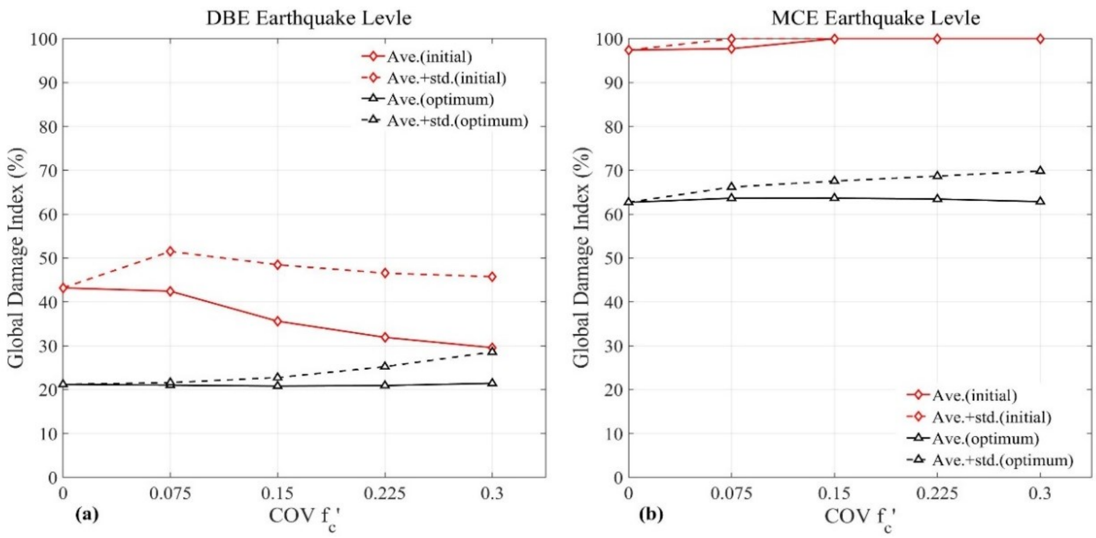


Figure 5-6: The global damage index (D_g) of 5-storey optimum and initial designs with the variations of $COV f_c'$, under (a) DBE and (b) MCE earthquake levels

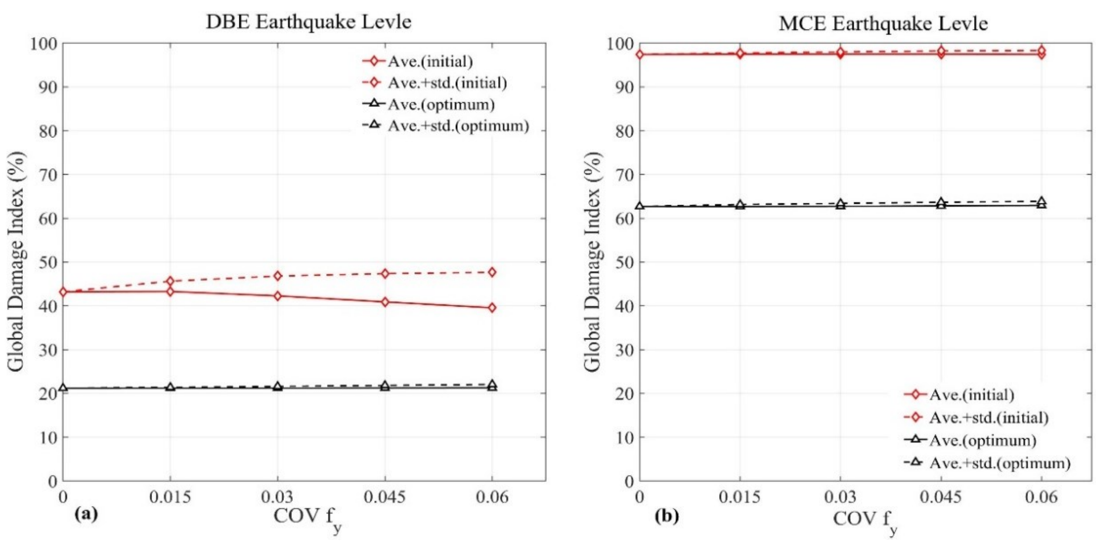


Figure 5-7: The global damage index (D_g) of 5-storey optimum and initial designs with the variations of $COV f_y$, under (a) DBE and (b) MCE earthquake levels

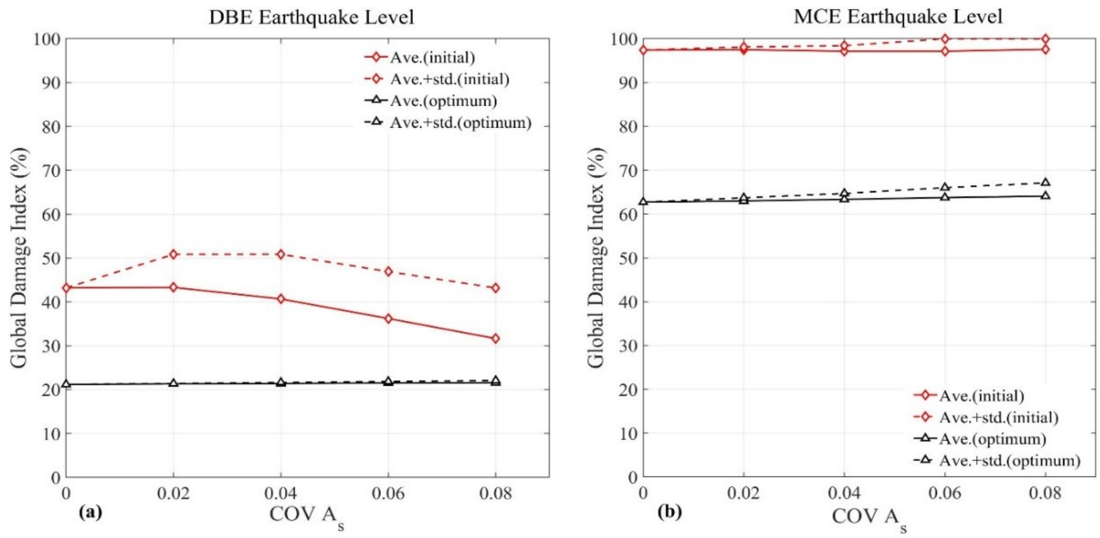


Figure 5-8: The global damage index (D_g) of 5-storey optimum and initial designs with the variations of $COV A_s$, under (a) DBE and (b) MCE earthquake levels

The results show that, (i) the structural performance of both code-based and optimum designs is affected by all the selected uncertainty variables, (ii) the variations in concrete compressive strength have the most significant impact on the estimated global structural damage, and (iii) the uncertainty in steel yield strength exerts a less obvious influence on the variation in the global damage index. These findings align with conclusions in a previous uncertainty study on RC frames, which also indicated the significant sensitivity of the structural seismic vulnerability at the global level to the uncertainty in the concrete compressive strength, particularly at damage control and collapse prevention limit states (Kim et al., 2020). This is because the concrete strength generally has substantial influence on the structural lateral stiffness, which indirectly affects the distribution of inertia force, the structural period, and amplification characteristic of an RC building under seismic loads. Additionally, the concrete strength has a relatively larger coefficient of variation (COV) compared to the steel strength.

The reinforcement cross-sectional area has less significant impact on the global seismic performance compared to the concrete strength uncertainty, but still, its influence is still noticeable especially for the initial building. This can be justified as the conventionally designed structure generally exhibits more concentrated maximum seismic response and non-uniform heigh-wise drift distribution. The steel reinforcement can in turn play a more dominant role in controlling the structural behaviour within the inelastic range in this case. However, in general, the effect of variations in the rebar area in each storey level is less than one in the concrete strength, because the areas of the steel reinforcements have inherently smaller COV (around 4 times less than the COV for the concrete strength) due to their more accurate manufacturing processes.

5.6. Effect of material and section properties variability

In this section, the effect of uncertainty on seismic performance is attributed to a combination of uncertainties in concrete compressive strength, steel yielding strength and rebar area. Figures 5-9, 5-10, 5-11 and 5-12 present the effect of the uncertainties on the average (Ave.) and average plus standard deviation (Ave. + std.) of the global damage indices for both conventionally and optimally designed frames, under multiple seismic hazard levels for the reference frames with 4 different storey heights, respectively.

To combine the effects of three uncertainty variables in the Monte Carlo estimations, a uniform uncertainty level is defined ranging from "low" to "high", described by the utilised COV values ($COV_{utilised}$) that are equal to 50%, 100%, 150% and 200% of the expected reference COV values ($COV_{expected}$), as presented in the Table 5-1. Each specific uncertainty level in these figures is represented as the ratio ($COV_{utilised}/COV_{expected}$), as shown on the horizontal axis of the figures. It assumes that even through the three variables are independent of each other, but they always have the same level of variability.

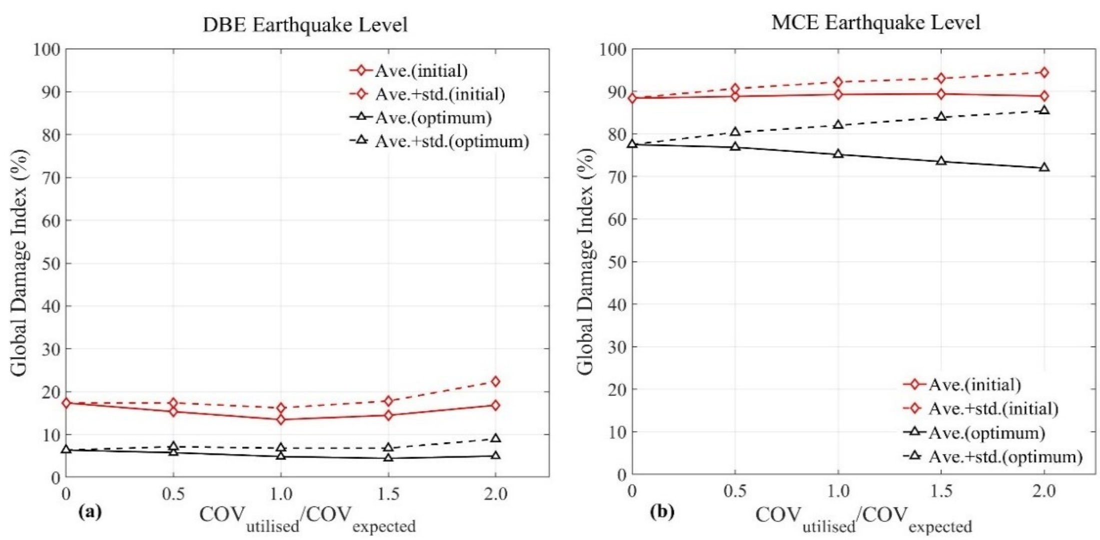


Figure 5-9: The global damage index (D_g) of 3-storey optimum and initial frames vary with uncertainty levels (quantified by $COV_{utilised}/COV_{expected}$), under (a) DBE and (b) MCE levels

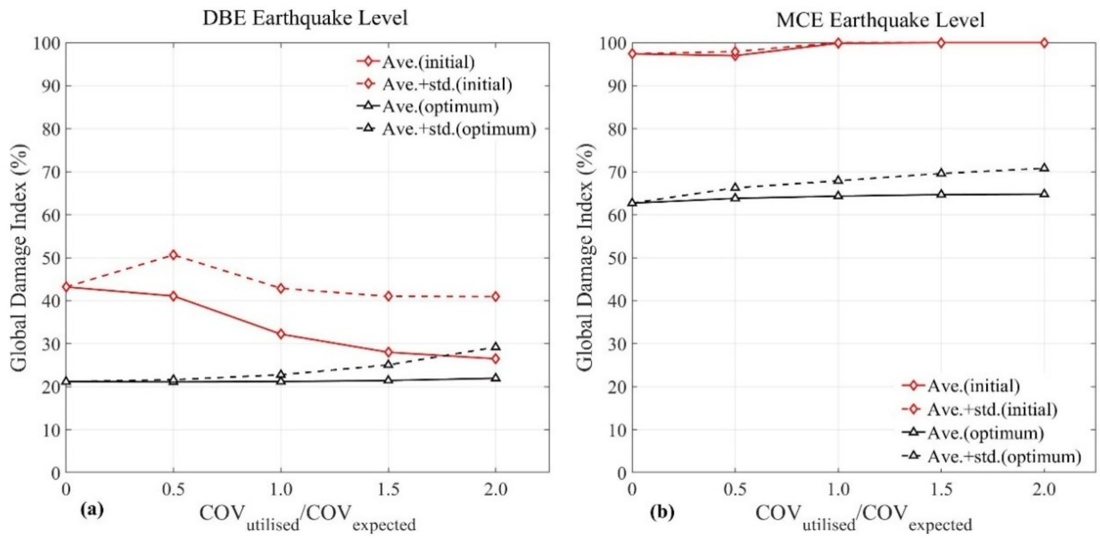


Figure 5-10: The global damage index (D_g) of 5-storey optimum and initial frames vary with uncertainty levels (quantified by $COV_{utilised}/COV_{expected}$), under (a) DBE and (b) MCE levels

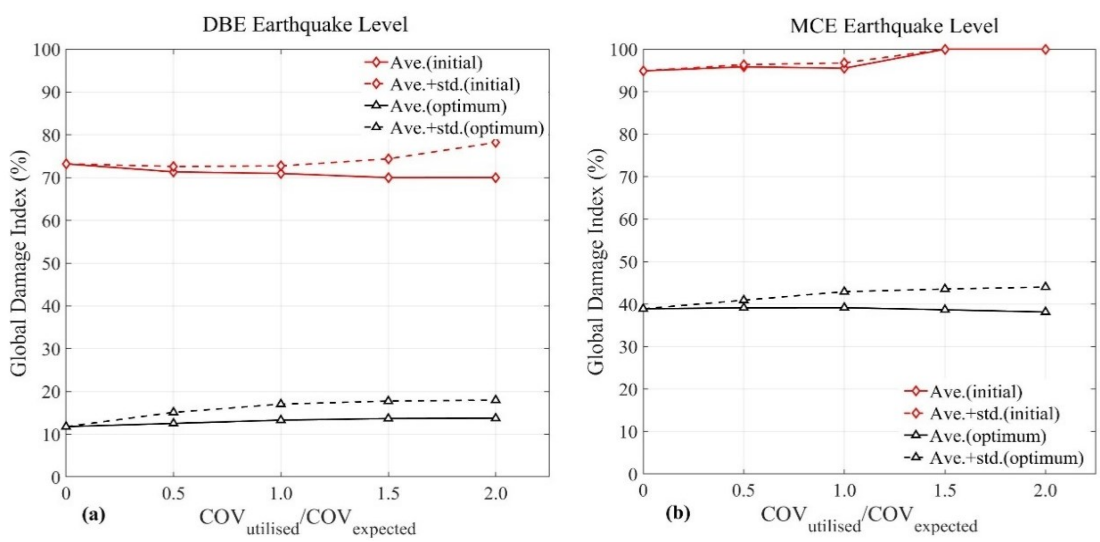


Figure 5-11: The global damage index (D_g) of 10-storey optimum and initial frames vary with uncertainty levels (quantified by $COV_{utilised}/COV_{expected}$), under (a) DBE and (b) MCE levels

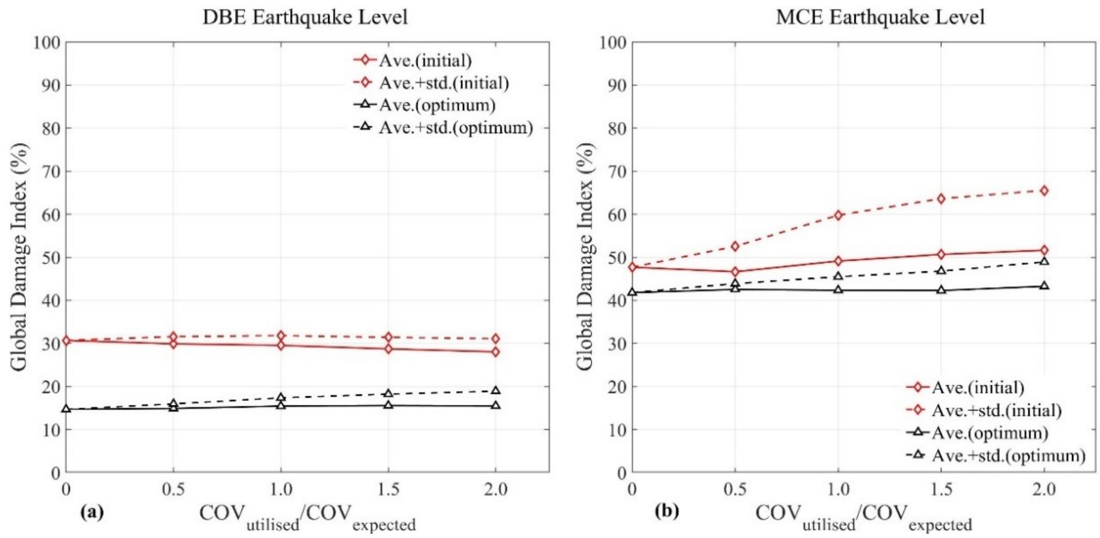


Figure 5-12: The global damage index (D_g) of 15-storey optimum and initial frames vary with uncertainty levels (quantified by $COV_{utilised}/COV_{expected}$), under (a) DBE and (b) MCE levels

It is evident that, the proposed optimisation framework consistently delivers designs with lower global damage indexes (D_g) compared to the Eurocode-based designs, regardless of the levels of variation in the uncertainty sources considered. The reduction in D_g is substantial, with differences of up to 70.3%, 48.6%, 82.4% and 50.2% observed for 3-, 5-, 10- and 15-storey RC frames, respectively. This outcome indicates that optimised structures reduce the risk of severe structural damage. Additionally, the average plus standard deviation results of D_g, represented by the dash line in the figures above, generally increase with the increasing uncertainty level of the selected uncertainty variables, quantified as the ratio $COV_{utilised}/COV_{expected,}$ for both code-based and optimum structures. However, such variation is more obvious for the conventionally code-based design.

From these observations, it is concluded that: (i) the UDD-based optimisation method consistently produces safer designs with significantly reduced global structural damage across multiple seismic hazard levels; (ii) the optimum solutions demonstrate relatively lower sensitivity to the selected uncertainty variables, thus always exhibiting less structural damage in reality considering the uncertainties arising from construction quality.

In general, the uncertain variables exert a more obvious influence on the D_g for structures that have already experienced heavy localised damage in certain storeys. This phenomenon occurs because even slight reductions in structural properties may result in significantly larger inter-storey drifts, especially in the storeys that have already sustained substantial damage. As a consequence, larger local and global damage indexes are evaluated in certain MC samples. This effect is particularly prominent in the code-based design, where structural damage tends to be more concentrated, especially when the structure becomes highly inelastic. This can be clearly seen in the 15-storey code-based RC frame at MCE level. As shown in the Figure 5-12, large uncertainties in the material and section properties (large COV values for $f_c', f_y, and\ A_s$) has a little impact on average D_g, but significantly affect the dispersion of the performance results.

For the optimum designs, structural damage is more uniformly distributed along the height of the structures, with less concentrated maximum inter-storey drifts and plastic rotations. In these optimum designs, the capacities of structural materials in most storeys are fully exploited. Therefore, even large vibrations in material and structural properties do not affect significantly the damage indexes at either the storey or structural level.

It should be noted that, in certain cases, such as the 5-storey frame in this study, increased COV can lead to a slight reduction in the global damage index for code-based design. This may occur because variability in material and sectional properties can occasionally lead to more efficient use of materials in certain stories. This may also change the pattern of drifts (or damage) distributing along the storeys, present the initial design form the concentrated localised damage and “soft-storey” failure. Nevertheless, the initial structure still experiences at least 17.1% larger D_g than the optimum 5-storey frame, under multiple seismic hazard levels.

5.7. Effect of uncertainty in seismic ground motions

In general, randomness of seismic input is the main source of uncertainty in the seismic design of structures (Kwon and Elnashai, 2006; Lee and Mosalam, 2005). To manage the uncertainty in seismic ground motions, previous studies have suggested that a more reliable optimum solution can be obtained by using the average responses corresponding to a set of earthquake records during the design optimisation process (De Domenico and Hajirasouliha, 2021; Hajirasouliha et al., 2012). This concept is adopted in the proposed optimisation framework, in which the reference RC frames are optimised based on the average response results (i.e. maximum inter-storey drift and maximum plastic rotation) for the six spectrum-compatible artificial earthquake records (namely “Mean artificial optimum design”).

To investigate the influences of seismic input uncertainty on the effectiveness of the proposed optimisation methodology, the obtained mean artificial optimum design is analysed under the fifteen independent natural earthquake records (see in Table 5-3). Moreover, Aslani (2005) found that the deviation of seismic response generally increases as the earthquake intensity increases, especially in lower storeys where deformation demands concentrated. To observe the uncertainty effect varying with earthquake intensities, three performance objectives relating to three earthquake intensity levels are considered in this section: Immediate Occupancy (IO) under frequent earthquakes, Life Safety (LS) under design basic earthquake (DBE) and Collapse Prevention (CP) under maximum considered earthquake (MCE).

Figures 5-13 and 5-14 present the height-wise distributions of maximum inter-storey drift ratios (Δ_{max}) for the 5- and 10-storey RC frames, respectively. Δ_{max} is evaluated through non-linear time history analysis using each selected independent natural earthquake record separately, considering multiple seismic hazard levels represented by performance objectives IO, LS and CP, highlighted in different colours (green, blue and red) in the figures. The dash lines represent the response results of the initial designs, while the solid lines represent the response results of the optimum designs. The title of each figure lists the name of the applied earthquake record, more details of these earthquake records, including earthquake magnitude, corresponding PGA value, response spectra and the recorded station, can be found in section 5.4.2 above.

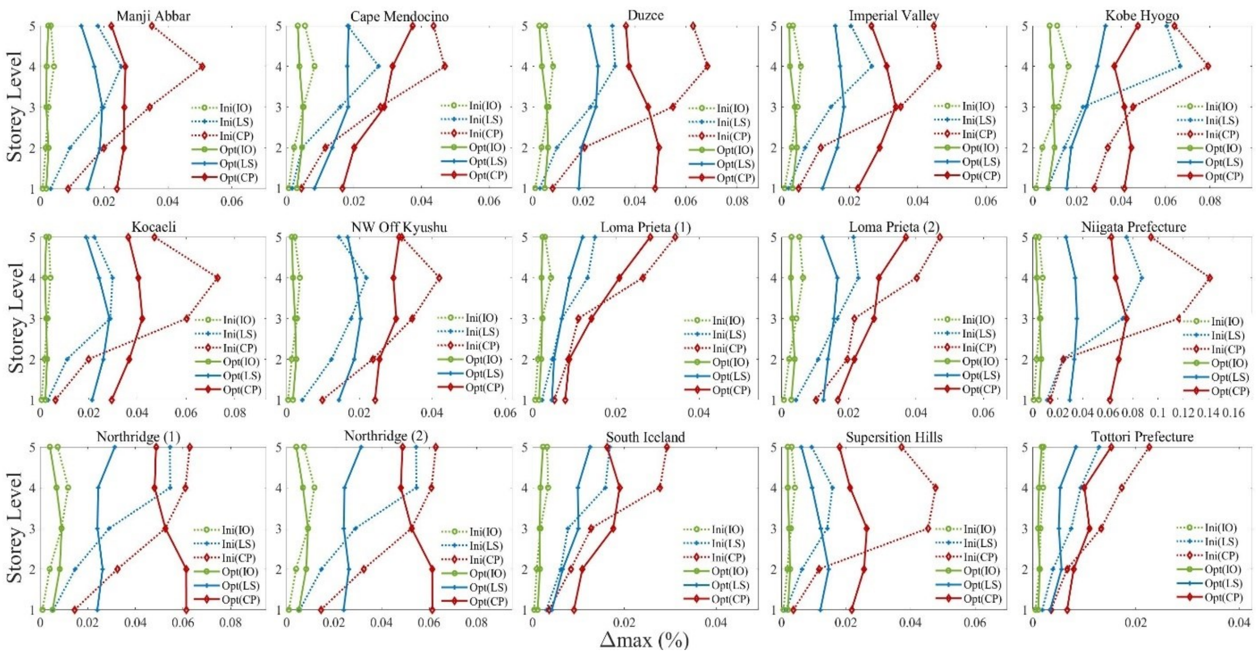


Figure 5-13: Height-wise distribution of maximum inter-storey drift ratios for optimum and initial design solutions for 5-storey RC frame, under fifteen natural earthquakes, at IO, LS, CP performance levels

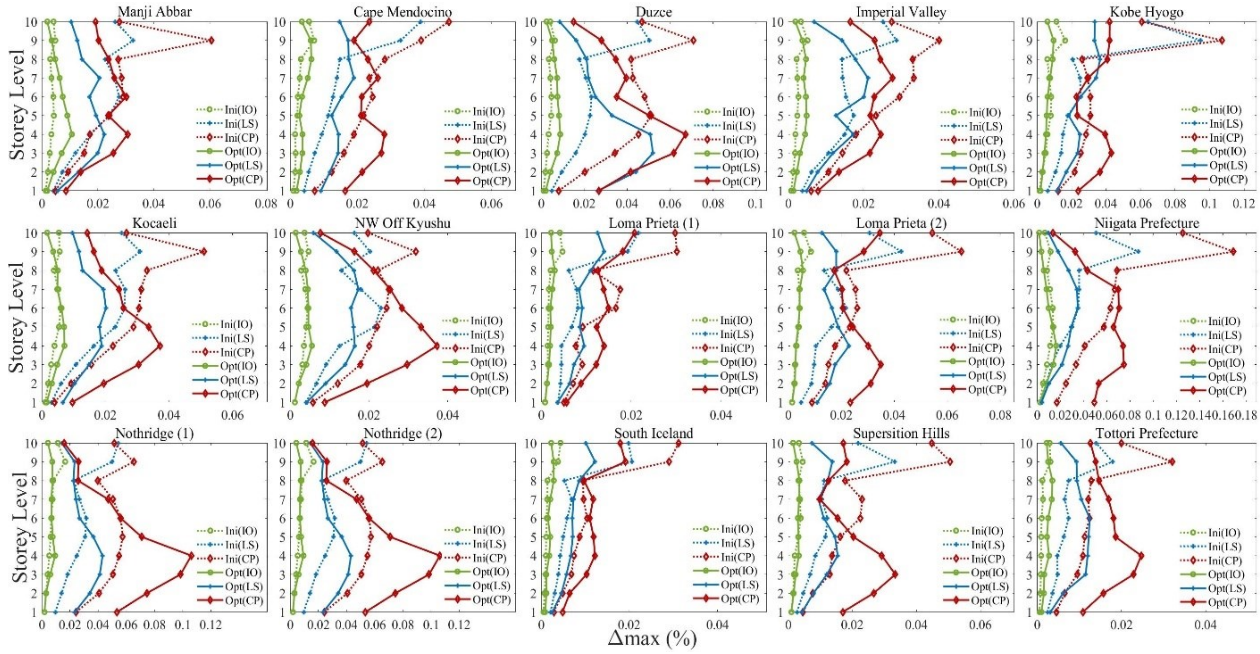


Figure 5-14: Height-wise distribution of maximum inter-storey drift ratios for optimum and initial design solutions for 10-storey RC frame, under fifteen natural earthquakes, at IO, LS, CP performance levels

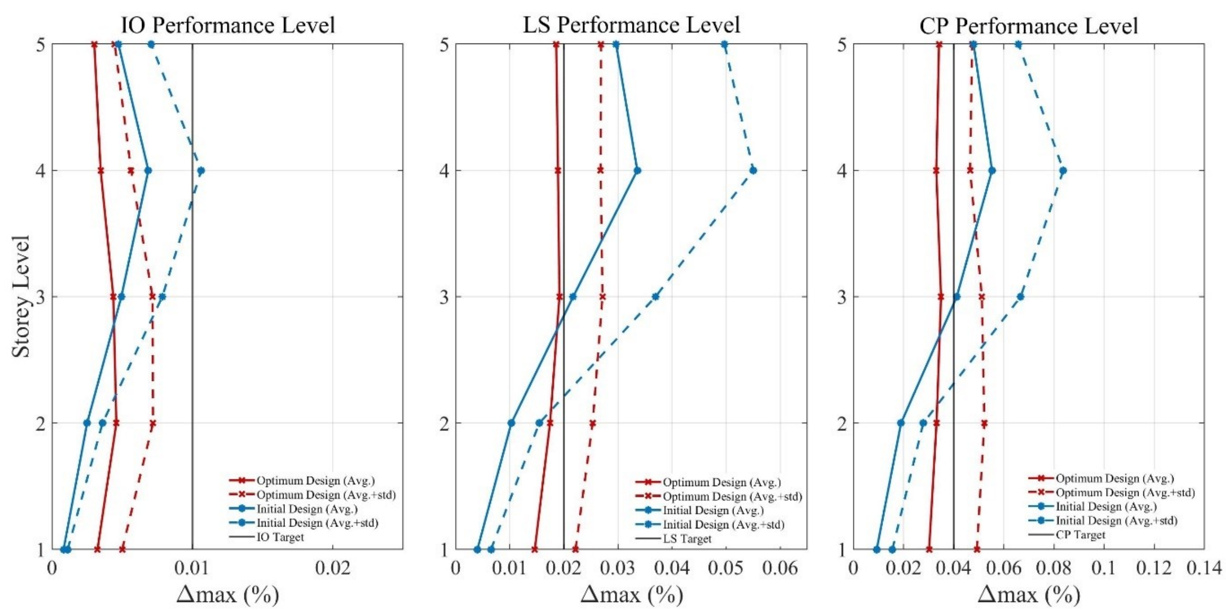


Figure 5-15: Height-wise distribution of maximum inter-storey drift ratios for optimum and initial 5-storey RC frame, average results for fifteen natural records under IO, LS, CP performance levels

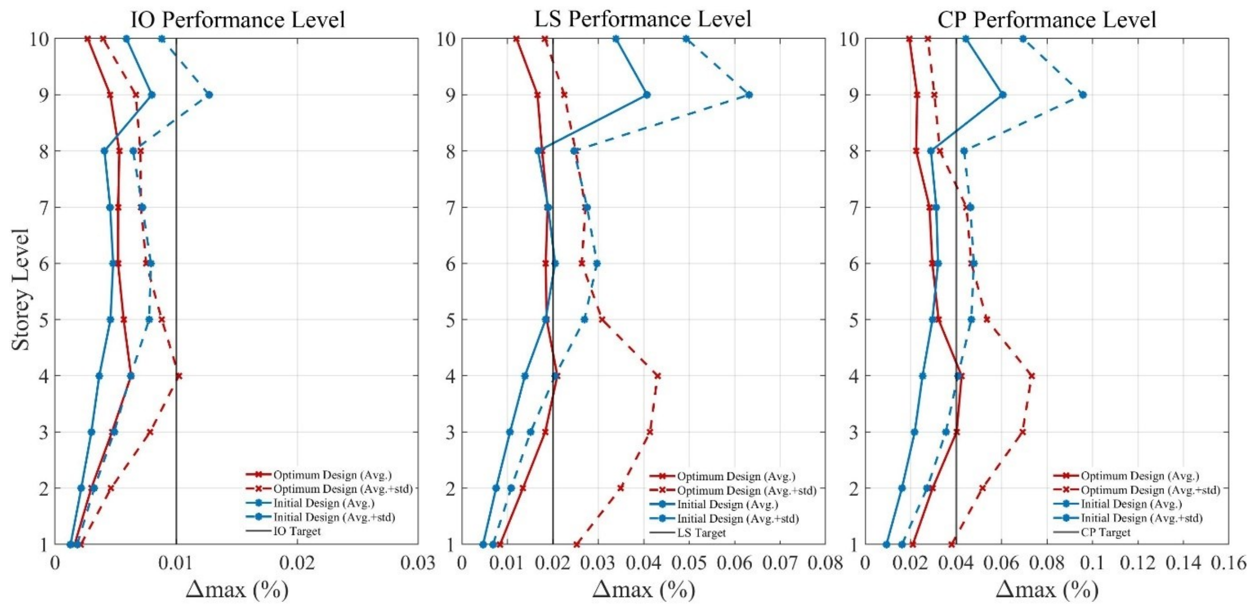


Figure 5-16: Height-wise distribution of maximum inter-storey drift ratios for optimum and initial 10-storey RC frame, average results for fifteen natural records under IO, LS, CP performance levels

The results in the Figures 5-13 and 5-14 clearly demonstrate that, compared to the code-based designs, the mean artificial optimum designs generally exhibit more uniform drift distribution and less concentrated maximum inter-storey drifts under each independent natural record. However, because these natural records are independent and have different dynamic characteristics (e.g. frequency content, amplitude, duration of significant ground motion, and spectrum profile) from the artificial earthquakes utilised during the optimisation process, certain individual natural earthquakes induce seismic responses that exceed the pre-determined performance-based target limits. This concern is particularly relevant when the structure behaves within the inelastic range under severe earthquakes (e.g. at CP performance level). Typical examples include the drift distribution results for the 5-storey frame under the “Northridge” and “Niigata Prefecture” earthquakes; as well as the results for the 10-storey under the “Duzce” earthquake.

For better understanding, Figures 5-15 and 5-16 present the average results of the Δ_{max} for the mean artificial optimum design under the fifteen natural earthquakes with different intensity levels. Overall, the average drift results for the selected independent earthquake records indicate that both the 5- and 10-storey optimum designs consistently meet the prescribed drift limits at all pre-determined performance levels. They also achieve great reductions in the Δ_{max} with up to 42.9% and 48.5% reductions, respectively. More details on the maximum inter-storey drift for both initial and optimum designs at multiple performance levels are provided in the following.

Figures 5-17 and 5-18 illustrate the structural local responses for the mean artificial optimum designs under each natural record with DBE and MCE levels, respectively. The local responses are expressed in terms of the maximum plastic rotation ratio ($\theta_{max,C}/\theta_{target,C}$) (the ratio of the maximum plastic hinge rotation ($\theta_{max,C}$) to the element plastic rotation capacity ($\theta_{target,C}$)) for all beam-column joints. It can be seen that the optimum designs generally exhibit less concentrated localised damage than the code-based frame during each natural earthquake event and reduce the maximum plastic rotation ratios up to 82.5% and 81.1% for 5- and 10-storey frames, respectively.

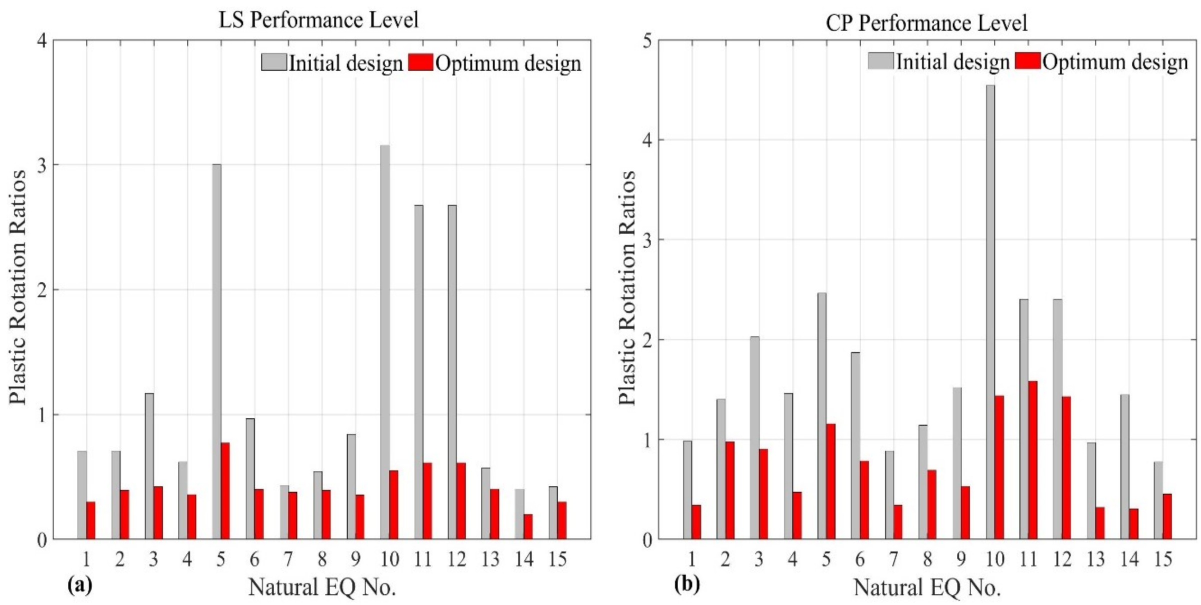


Figure 5-17: Maximum $\theta_{max,C}/\theta_{target,C}$ of 5-storey initial and optimum frames, results under fifteen natural earthquakes at (a) LS and (b) CP performance levels

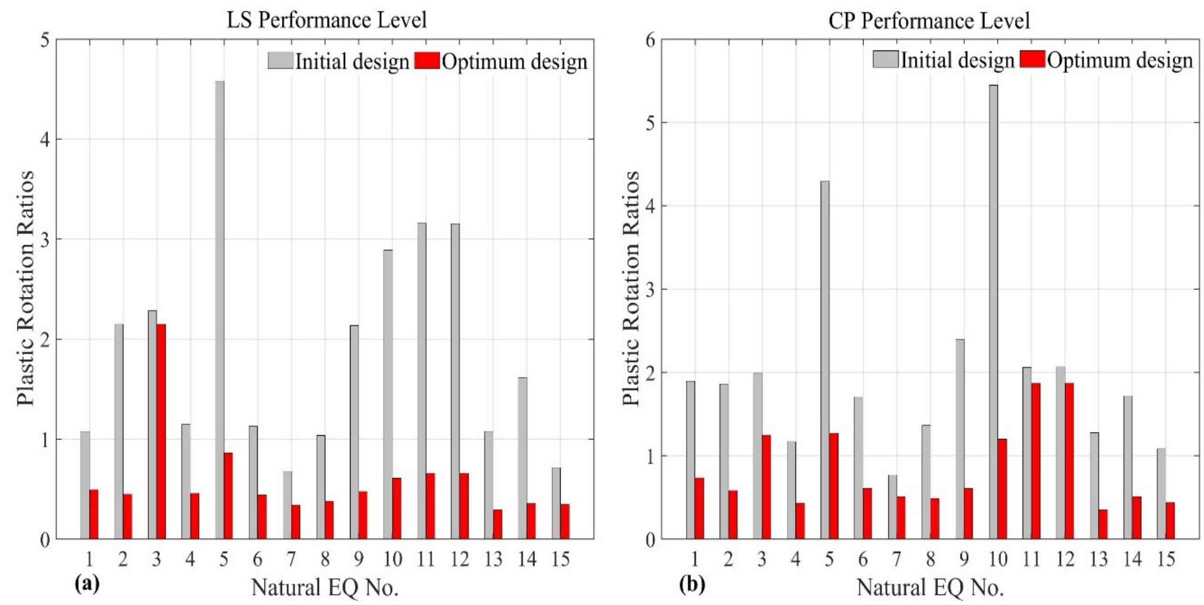


Figure 5-18: Maximum $\theta_{max,C}/\theta_{target,C}$ of 10-storey initial and optimum frames, results under fifteen natural earthquakes at (a) LS and (b) CP performance levels

Tables 5-4 - 5-8 summarise the average and the average plus the standard deviation response results associated with the maximum inter-storey drift and the maximum plastic rotation ratios in columns, under the fifteen independent earthquakes. These tables also compare the response results between code-based initial and optimum designs at multiple performance levels (i.e. IO, LS and CP). The results indicate that, compared to the code-based frames, the mean artificial optimum designs experience up to 14%, 42.9%, 48.5% and 10.3% less maximum inter-storey drift (Δ_{max}) for 3-, 5-, 10- and 15-storey frames, respectively, under multiple seismic hazard levels. They also exhibit less maximum plastic rotation ratio ($\theta_{max,C}/\theta_{target,C}$) by up to 20.8%, 38.7% and 58.2% for 3-, 5- and 10-storet frames, respectively. For the 15-storey RC frame, the optimum design exhibits slightly larger inter-storey drifts and plastic rotation ratios than the initial design especially within the inelastic range, but these responses still satisfy the prescribed performance constraints. Meanwhile, it should be noted that the optimum 15-storey frame requires considerably less total material usages, as shown in the Table 5-2. It is concluded that the earthquake record uncertainty does not significantly affect the maximum seismic responses at both local and global levels, and do not have an obvious influence on the efficiency of the optimum designs.

Table 5-4: Average and average plus standard deviation results of maximum inter-storey drift ratio (%) of 3-, 5-, 10- and 15-storey frame under fifteen natural earthquakes at IO level

RC frames	Optimum Design		Initial Design		Gain
	Avg.	Avg. + std	Avg.	Avg. + std	
3-storey	0.40	0.65	0.44	0.68	9.1%

5-storey	0.46	0.72	0.69	1.06	33.3%
10-storey	0.63	1.02	0.80	1.27	21.3%
15-storey	0.52	0.77	0.58	0.87	10.3%

Table 5-5: Average and average plus standard deviation results of maximum inter-storey drift ratio (%) of 3-, 5-, 10- and 15-storey frame under fifteen natural earthquakes at LS level

RC frames	Optimum Design		Initial Design		Gain
	Avg.	Avg. + std	Avg.	Avg. + std	
3-storey	2.22	3.49	2.58	3.97	14.0%
5-storey	1.92	2.71	3.36	5.50	42.9%
10-storey	2.10	4.30	4.08	6.32	48.5%
15-storey	2.05	3.59	2.19	3.04	6.4%

Table 5-6: Average and average plus standard deviation results of maximum inter-storey drift ratio (%) of 3-, 5-, 10- and 15-storey frame under fifteen natural earthquakes at CP level

RC frames	Optimum Design		Initial Design		Gain
	Avg.	Avg. + std	Avg.	Avg. + std	
3-storey	3.97	6.17	4.60	6.96	13.7%
5-storey	3.50	5.12	5.53	8.37	36.7%
10-storey	4.23	7.32	6.05	9.58	30.1%
15-storey	3.48	5.33	3.07	4.35	-13.3%

Table 5-7: Average and average plus standard deviation results of maximum plastic rotations demand to capacity ratios in columns of 3-, 5-, 10- and 15-storey frame under fifteen natural earthquakes at LS level

RC frames	Optimum Design		Initial Design		Gain
	Avg.	Avg. + std	Avg.	Avg. + std	
3-storey	0.61	0.96	0.77	1.31	20.8%
5-storey	0.73	1.14	1.19	2.19	38.7%
10-storey	0.76	1.96	1.82	2.85	58.2%
15-storey	0.78	1.37	0.66	1.02	-18.2%

Table 5-8: Average and average plus standard deviation results of maximum plastic rotations demand to capacity ratios in columns of 3-, 5-, 10- and 15-storey frame under fifteen natural earthquakes at CP level

RC frames	Optimum Design	Initial Design	Gain

	Avg.	Avg. + std	Avg.	Avg. + std	
3-storey	0.91	1.57	1.06	1.66	14.1%
5-storey	1.09	1.88	1.58	2.24	31.0%
10-storey	0.90	1.94	1.93	2.87	53.4%
15-storey	1.04	2.85	0.71	1.02	-31.7%

Figures 5-19 - 5-22 compare the global damage index (D_g) for the 3-, 5-, 10- and 15-storey frame under each selected natural earthquake (grey and red columns) and the corresponding average results (white and blue columns) at LS and CP performance levels, respectively. The results indicate that the optimum designs generally experience less global structural damage under each natural earthquake, and the average D_g is reduced by up to 22.3%, 31.4%, 52.6% and 6.8% for 3-, 5-, 10- and 15-storey frame, respectively. For the 15-storey frame, optimum and initial solutions exhibit almost identical structural damage, but meanwhile, the optimum design requires less total concrete volume and reinforcement weight, as summarised in the Table 5-2. There is evident that using the average responses corresponding to a group of spectrum-compatible earthquakes in the proposed design optimisation can efficiently manage the effect of uncertainty in the different earthquake records on the global structural damage. The effectiveness of the proposed UDD-based optimisation method, on average, is not significantly affected by the randomness in the earthquake records.

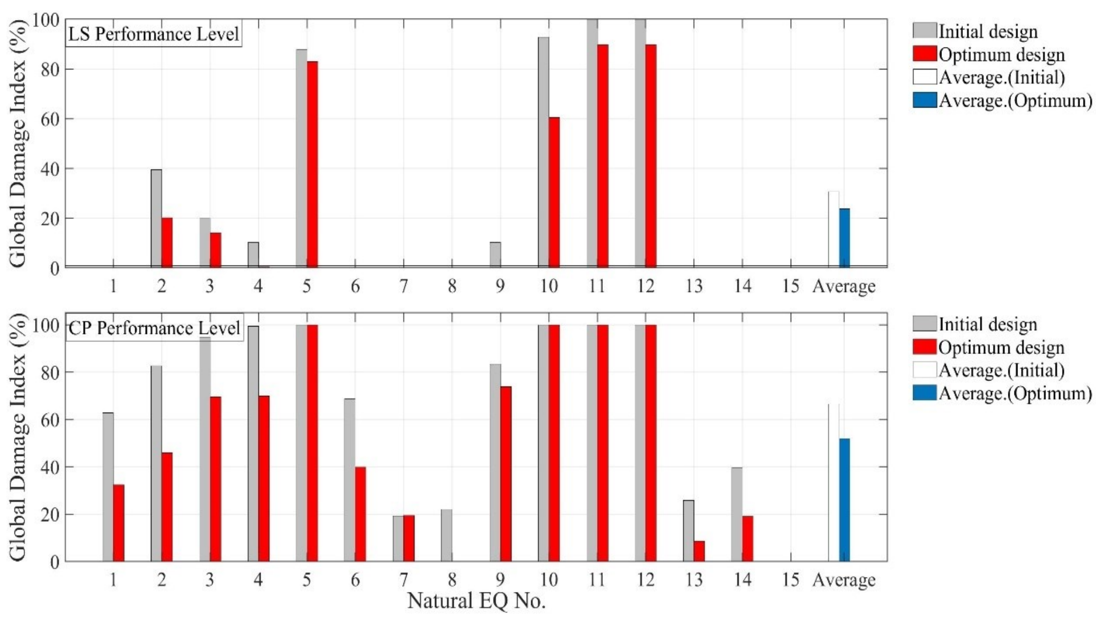


Figure 5-19: Global damage index (%) of 3-storey initial and optimum structures, individual and average results under fifteen selected natural records at LS (top) and CP (down) levels

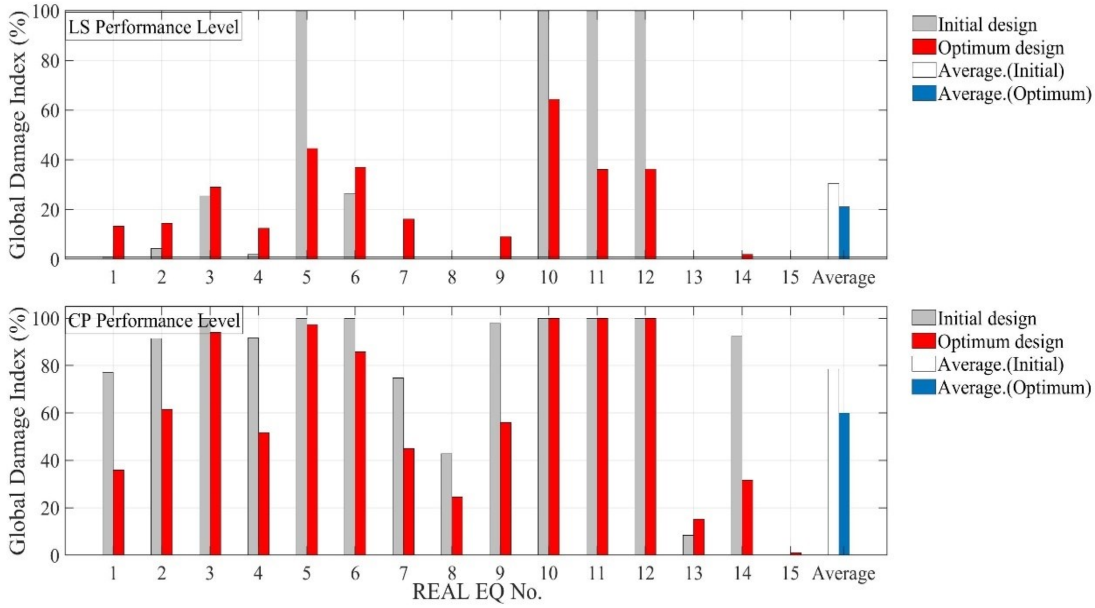


Figure 5-20: Global damage index (%) of 5-storey initial and optimum structures, individual and average results under fifteen selected natural records at LS (top) and CP (down) levels

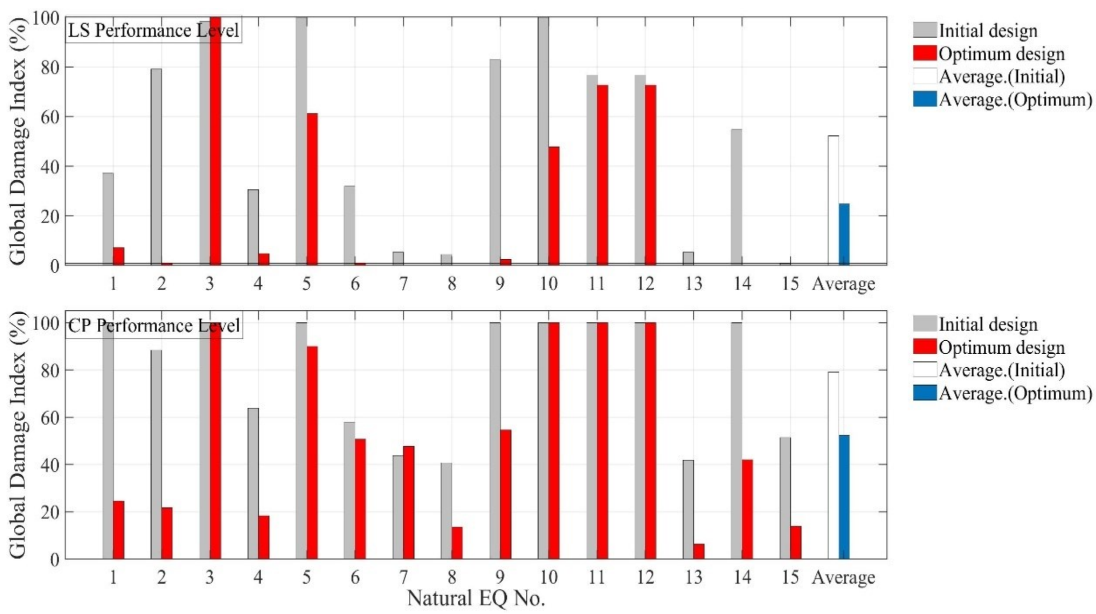


Figure 5-21: Global damage index (%) of 10-storey initial and optimum structures, individual and average results under fifteen selected natural records at LS (top) and CP (down) levels

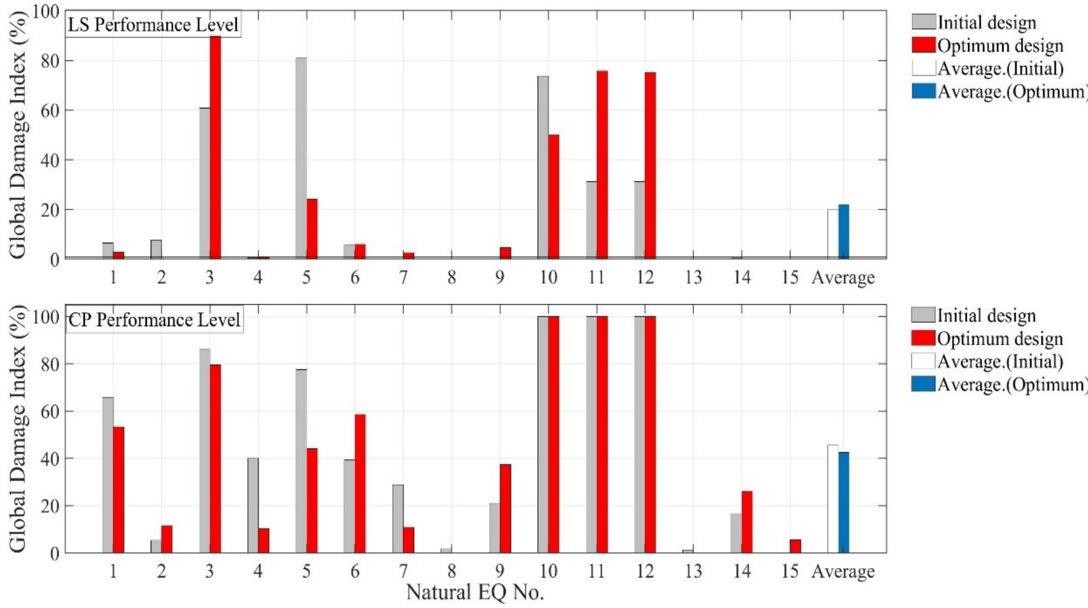


Figure 5-22: Global damage index (%) of 15-storey initial and optimum structures, individual and average results under fifteen selected natural records at LS (top) and CP (down) levels

5.8. Summary and Conclusions

A multi-level performance-based seismic design optimisation method based on the Uniform Damage Distribution (UDD) concept has been developed as a novel approach to achieve the optimum design solutions of RC frames located in high seismic regions. To address uncertainties arising from construction practices, three uncertain variables relating to the construction quality are considered as: concrete compressive strength (f_c',), steel yield strength (f_y) and cross-sectional area of reinforcement (A_s). An appropriate sample size was decided for the Monte Carlo estimation to ensure efficient and reliable uncertainty analysis. The study evaluated the seismic performance sensitivity to these uncertain variables for both initial and optimum designs. Using the Monte Carlo simulation method, this study investigated the uncertainty effects by combining material and section property variabilities for the initial code-based and optimum designs. The global damage index served as the performance parameter, assessed through non-linear time history analysis under a selected single artificial earthquake with multiple hazard levels. Finally, this study considered the uncertainty arising from earthquake records, assessing its impact by analysing both initial and optimum structures under a group of independent natural earthquakes with different dynamic characteristics from those used in the design optimisation. From the results presented in this study, the major findings are summarised as following:

- The efficiency of both optimum and code-based designs is affected by all selected uncertainty variables. Concrete compressive strength uncertainty has the most significant impact, while the steel yield strength has the least influence on seismic performance variability.

- Increasing the Coefficients of Variation (COVs) for the uncertainty variables affects the efficiency of both code-based initial and optimum structures. The proposed UDD optimisation always resulted in safer designs with lower global damage indexes, up to 70%, 49%, 82% and 50% for 3-, 5-, 10- and 15-storey RC frame, respectively. Furthermore, the optimum designs were relatively less sensitive to increased uncertainty level (quantified by COVs) at both DBE and MCE levels.

- Compared to the code-based designs, the optimum frames exhibited more unform drift distributions, less concentrated maximum inter-storey drifts, and less localised structural damage (i.e. lower plastic rotation in columns) under most selected individual independent natural earthquakes at multiple performance levels (i.e. IO, LS and CP).

- The average responses for the independent natural earthquakes demonstrated that, compared to the code-based designs, the optimum structures experienced up to 49% less maximum inter-storey drift and up to 58% less maximum plastic rotation ratios in columns, while satisfying all prescribed performance-based constraints corresponding to different seismic hazard levels.

- The average global damage indexes for the fifteen natural seismic excitations indicated that the optimum solutions exhibited up to 22%, 31%, 53% and 7% less global structural damage for 3-, 5-, 10- and 15-storey RC frames, respectively, compared to the initial code-based designs.

These results validate the effectiveness of the proposed UDD-based optimisation method and highlight that utilising average response under a set of seismic records can effectively manage the uncertainty in earthquake records in the proposed design optimisation.

CHAPTER 6 : Conclusions and Recommendations for Future Work

6.1. A Restatement of Research Problem and Main Objectives

The main purpose of this study is to develop a practical and computationally efficient multi-level performance-based optimisation methodology for seismic design of multi-storey RC frames, with objectives of minimising initial material costs and structural damage. The research objectives outlined in Section 1.3 have been comprehensively achieved throughout the researches conducted in this thesis. Beginning with a critical review of existing studies on the optimum seismic design of RC frames in Chapter 2, the chapter summarised the existing challenges and research gaps in this field.

To address the identified research gaps, Chapter 3 introduced the developed optimisation methodology. This optimisation approach is founded on the concept of Uniform Damage Distribution (UDD), while incorporating design criteria derived from performance-based seismic design guidelines, such as ASCE 41. The methodology simultaneously controlled both local and global structural seismic responses to satisfy multiple performance objectives, in terms of Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). The optimisation approach iteratively modified multiple design variables, including cross-sectional sizes and longitudinal reinforcement amounts, in both elastic and plastic phases. The successful implementation of this proposed optimisation methodology involved developing suitable nonlinear material and element models in Finite Element software OpenSees (McKenna et al., 2006) and applying non-linear time history analysis. In this Chapter, initial code-based designs for 3-, 5-, 10- and 15-storey RC frames were constructed, significant improvements in the seismic performances of the referenced buildings and reductions in structural total material usages were observed through a comparison between initial Eurocode-based and optimum seismic designs. In Chapter 3, iterative optimisation steps were completed by writing a routine in

MATLAB (MATLAB R, 2020). The computational efficiency of the proposed methodology was demonstrated by completing the iterative structural optimisation process within 40 steps.

Results in Chapter 3 and 4 indicated that using a group of spectrum-compatible earthquake records, rather than a single earthquake record, was necessary in this optimisation framework. It was also found that the efficiency of the framework was minimally affected by variations in earthquake records, whether artificial or natural seismic ground motions. Additionally, the cost-efficiency of the proposed optimisation method was evaluated in Chapter 4, by thoroughly assessing initial construction costs and expected total life-cycle costs separately for both code-based and optimum design solutions. It was observed that the costs saving was achieved without compromising structural seismic performance.

Chapter 5 focused on quantifying the effects of uncertainties on the performances of conventionally and optimally designed RC frames. Through the uncertainty analysis, it was ensured that the optimised designs remained efficiency when considering several key uncertainty sources in structural sectional and material properties, as well as in unpredictable seismic events.

6.2. A Summary of Findings

From the results obtained in Chapters 3, 4 and 5 in this study, the following conclusions can be drawn:

6.2.1. Optimum Design Under the Artificial Earthquake Records

- The efficiency of the optimisation method was first demonstrated by optimising 3-, 5-, 10- and 15-storey RC frames under six spectrum-compatible artificial earthquakes. Compared to the conventionally designed frames (using Eurocode 8), the optimum designs of the reference frames exhibited lower maximum inter-storey drift ratios (up to 58%), lower maximum plastic rotation ratios (up to 78%), and less overall structural damage (up to 88%), quantified by global damage index. Additionally, the optimum frames also, in general, experienced more uniform height-wise distributions of inter-storey drift ratios and plastic rotation ratios. Overall, it prevented “soft-storey” failures and reduced localised damage.

- The optimum designs solutions generally required considerably less total material usages by more efficiently utilising the cross-sectional dimensions and steel reinforcement ratios in each storey. This was particularly evident in the case of tall buildings (i.e. 10- and 15-storey RC frames), where total concrete volumes and total reinforcement weights were reduced by around 20% and 36%, respectively. For the 3-storey frame, the material usages were slightly increased (up to 4%) to satisfy the multiple pre-determined performance requirements.

- In the UDD formula, the convergence parameter (α) was found to have the greatest impact on the efficiency and the computational speed of the design optimisation. A value of 0.2 was used to achieve optimum answers within 40 iterative steps. This confirms the computational efficiency of the proposed optimisation methodology.

- The effect of earthquake record sections in the proposed optimisation approaches was assessed by repeating the same design optimisation process under a randomly selected single earthquake record. The investigation results, including total material usages and seismic performances, indicated that relying on a single spectrum-compatible earthquake may lead to less economic designs with less satisfactorily structural performance, especially for tall buildings. For more robust and economically efficient solutions, it is recommended to use the average response results from a group of spectrum-compatible seismic records in the proposed optimisation framework.

6.2.2. Optimum Design Under the Natural Earthquake Records

- The efficiency of the optimisation method was further demonstrated by optimising 3-, 5-, 10- and 15-storey RC frames under fifteen spectrum-compatible independent natural earthquake records. Compared to optimum designs under artificial records, using natural earthquakes in the design optimisation process resulted in optimum designs with slightly higher total material usage. Nevertheless, they still required less total concrete volumes (up to 19%) and total reinforcement weights (up to 42%) compared to the initial code-based designs.

- When subjected to a group of independent natural earthquake, both the mean artificial optimum designs and the mean natural optimum designs exhibited the following compared to the initial code-based designs: (i) up to 48% and 49% reduction, respectively, in maximum inter-storey drifts; (ii) up to 58% and 60% reduction, respectively, in maximum plastic rotation ratios in columns, and (iii) up to 53% and 47% reduction, respectively, in global structural damage. This confirms that, using the proposed optimisation method under both natural and artificial earthquake sets led to design answers with similar and satisfactory performances. They both closely approached the drift-based limiting values and rotation-based limits (within 5%) in each storey at multiple performance levels. However, using natural earthquakes led to optimum designs with similar or larger global structural damage (up to 14%) and higher standard deviations of the global damage indexes than the mean artificial optimum designs, due to the more random earthquake characteristics and a wider range of variability in the natural seismic inputs.

- Overall, it is concluded that using artificial earthquakes in the optimisation framework led to relatively more economic design solutions (less reinforcement weights and concrete volumes). A set of spectrum-compatible artificial earthquakes are recommended to be utilised as seismic inputs in proposed design optimisation process. This is especially beneficial when there are only very limited real earthquake records for a selected seismic region.

6.2.3. Life-cycle Cost Assessment of Initially and Optimally Designed RC Frames

- The Incremental Dynamic Analysis (IDA), as first step in this life-cycle cost assessment, its results showed that optimum designs resulted in up to 85% reduction in global damage index than the conventionally designed frames under a wide range of earthquake PGA levels, quantified by PGA values varying from 0.05g to 1.0g. The results from IDA were further expanded into the fragility analysis. It indicated that, the optimum designs were consistently efficient in reducing limit-state probabilities (fragilities) corresponding to several predetermined damage states ranging from slight damage to destroyed.

- The optimum designs required less initial construction costs (up to 15%), whilst over the building's effective lifetime sustaining lower expected damage costs (by up to 87%) and having up to 64% less total life-cycle costs than the code-based initial designs.

- For the 3-storey RC frame, although its optimum solution required slightly more initial construction cost (up to 0.6%) than the code-based design, its life-cycle cost was reduced by around 15%. This highlights that design optimisations with a single objective of minimising initial costs do not necessarily lead to an economic design when the total costs over the structure's life period are considered.

6.2.4. Effect of Uncertainties on Seismic Performance of Optimum Designs

- The seismic performances (i.e. global damage index) of both optimum and code-based design structures were affected by the considered uncertainty variables, including concrete compressive strength, steel yielding strength and rebar area, under both design basic earthquakes (DBE) and maximum considered earthquakes (MCE) levels. The greatest part of uncertainty in the seismic performance evaluation was derived from concrete strength variability, and the seismic performance was less sensitive to steel strength variability.

- When multiple sources of uncertainties in concrete and steel strengths and rebar design detailing were simultaneously considered in the referenced frames, increasing the uncertainty levels (quantified by Coefficient of Variation (COV) for the uncertainty variables) affected both code-based and optimised 3-, 5-, 10- and 15-storey RC frames. Compared to the code-based frames, the proposed optimisation method consistently led to safer designs with lower global damage index (up to 82%) considering a wide range of uncertainty levels. Furthermore, the optimum designs were relatively less sensitive to the increased COVs for the uncertainty variables, at both DBE and MCE levels.

- The effect of earthquakes uncertainties was assessed by analysing the initial and optimum designs under fifteen independent natural earthquakes. Compared to conventionally designed frames, the optimum designs still experienced lower maximum drift results and less global structural damage and prevented localised damage (lower plastic hinge rotations) under most individual natural earthquakes.

- The average response results for the fifteen independent natural earthquakes showed that the optimum frames exhibited less maximum inter-storey drift (up to 49%), less maximum plastic rotation (up to 58%) and lower global structural damage (up to 53%) than the code-based designs, and they still satisfied all targeted performance objectives corresponding to multiple seismic hazard levels. This confirms that utilising the average seismic response for a set of earthquake records in the proposed optimisation can effectively manage effect of uncertainty in seismic inputs.

6.3. Practical Implications and Further Impacts

In conclusion, the research conducted in this thesis has comprehensively and successfully addressed all listed objectives. This research significantly contributes to this field by understanding current research gaps and design challenges in the seismic design methods particularly for RC frames, and by advancing the practical application of optimum seismic designs. It provides direction for more efficient, resilient and economically viable structural seismic designs, particularly important for developing countries and regions prone to frequent earthquakes. The proposed optimum seismic designs can effectively control structural local and more global performances within the plastic phases, considering the effects of energy dissipation and strength degradation beyond the yielding of a structure. Thus, the optimisation methodology is especially beneficial for seismicity areas with higher hazard levels, it leads to practical design solutions with more accurate predictions on structural inelastic behaviours. Moreover, the structures were optimised not only by considering restrictions on structural seismic performances, but also by verifying compliance with design constraints outlined in current design guideline and realistic construction practices. This approach ensured that optimum designs were achieved with greater practical applicability.

In addition, this work has a broader impact on both academia and practice. The insights gained from this study can inform engineers and researchers in the development of seismic design guidelines. By encouraging the consideration of structural optimisation in future design codes, the optimisation can be applied at the end of the design procedure, after the structure is conventionally designed following specific codes. It can improve designs solely relying on the design guidelines, leading to the creation of safer, more sustainable structures, and more customised solutions in accordance with specific requirements from clients and community. The automatic optimisation algorithms, which are generally processed by writing a script through a programming language, can be added in current design tools. They iteratively modify design variables to closely approach specific design objectives and performance targets, also providing feasible direction to efficiently lead the best design solutions that may be difficult to obtain using the traditional "trial-verification-modification" design method. Meanwhile, the methodology developed here can be adapted to a wide range of structure systems and applied to achieve complex design objectives. Overall, this research contributes to mitigating the impacts of earthquake events on buildings and community, ultimately fostering a more sustainable and safe environment for next generations.

6.4. Recommendations for Future Work

- In the proposed design optimisation procedure, shear reinforcement was assumed to be approximately proportional to the amount of the longitudinal reinforcement. However, this is not necessarily true and shear reinforcement should be included as another design variable, considering its significant impact on the deterioration of the structural non-linear deformations of beams and column elements.

- In this study, the initial construction costs were calculated by mainly considering material costs and labour and fabrication costs for concretes, steel reinforcements and formwork. For practical applications, more cost components, such as transport and storage fees, foundation construction costs, should be included in the evaluation of the initial costs.

- The performance-based constraints subjected in the proposed optimisation framework were generally expressed in deterministic form, such as constant values specified in ASCE 41-06. In future studies, it is proposed to use constraints in a probabilistic form (namely reliability constraints), so that exceedance probability of the performance-based target limits is evaluated and uncertainties in the performance-based seismic design can be explicitly dealt with in the optimisation problem.

- This study only assesses the life-cycle cost assessments for the optimum frames that are obtained based on "initial cost" objective. To develop sustainable seismic design solutions, minimising the total life-cycle costs should be considered as an alternative objective in the proposed optimisation, and sustainability parameters in terms of amounts of greenhouse gas emissions and global CO_2 emissions should be included by using penalty clauses.

- Other epistemic uncertainties, such as parameters that affect the characteristics of members' plastic hinge rotations and their corresponding capacities (i.e. effective slab width, ultimate strength of steel), should be considered.

- Besides global damage index, the effects of uncertainty on the local performance of frames should also be investigated.

- The proposed low computational cost optimisation methodology can be developed for other structural systems, such as steel frames or RC frames equipped with dampers.

- AI can be integrated into structural seismic design optimisation in future studies, ultimately leading to more efficient and accurate seismic design solutions. For instance, machine learning algorithms, such as neural networks, can be embedded in the proposed optimisation framework to iteratively adjust several groups of design variables and achieve more complex and practical optimum designs that satisfy multiple conflicting design objectives. AI techniques can also be applied to predict structural seismic performance in a more accurate and computationally efficient manner, where predictive models can be developed by training machine learning models with input historical seismic data and different design configurations. Furthermore, AI techniques in data-driven approaches can be applied to select the most sustainable earthquake records subjected to the optimisation framework to provide the most reliable optimum designs.

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APPENDIX A

Summary on Previous Relevant Optimisation Studies

Table A.1. Research developments in design optimisation with aims of minimizing structural damage

Researcher	Year	Structure	Optimisation methodology	Seismic analysis	Design Variables
Varughese et al.	2014	RC frames	Chao lateral load distribution pattern	Non-linear dynamic analysis	Lateral load in each storey
Li et al.	2019	RC frames	Uniform Damage Distribution	Non-linear dynamic analysis	Shear strength in each storey
Hajirasouliha et al	2012	RC frames	Uniform Damage Distribution	Non-linear dynamic analysis	Longitudinal reinforcement ratio
Bai et al.	2016	RC frames	Uniform Deformation Distribution	Consecutive pushover analysis (Non-linear static analysis)	Reinforcement areas
Bai et al.	2020	RC frames	Optimality Criteria (OC)	Non-linear dynamic analysis	Rebar areas, section dimensions
Hashmi et al.	2018	regular and irregular RC frames	Uniform Deformation Distribution	Linear elastic analysis	Depth of beam and column
Hashmi et al.	2022	Irregular RC farmes	Uniform Damage Distribution	Non-linear dynamic analysis	Reinforcement ratio

Arroyo and Gutiérrez	2017	RC frames	Genetic algorithms, homogenization method	Response calculated based on elastic mode	Dimensions of structural members
Arroyo et al.	2018	RC frames	Genetic algorithms, homogenization method	Response calculated based on elastic mode	Dimensions of structural members

Table A.2. Research developments in design optimisation with aims of minimising costs

Researcher	Year	Structure	Optimisation methodology	Seismic analysis	Design Variables
Ganzerli et al.	2000	RC frames	Intermediate optimisation cycle	Non-linear static analysis	Cross-section size, reinforcement area
Chan and Zou	2005	RC frames	Optimality Criteria (OC), Lagrangian function, gradient-based solution	Non-linear static analysis	Structural member sizes, longitudinal reinforcement
Zou and Chan	2005	RC frames	Optimality Criteria (OC), Lagrangian function, gradient-based solution	Non-linear static analysis	Structural member sizes, longitudinal reinforcement
Hajirasouliha et al.	2012	RC frames	Uniform Deformation Distribution	Non-linear dynamic analysis	Longitudinal reinforcement weight in each storey
Fragiadakis and Papadrakakis	2008	RC frames	Evolution Strategies	Non-linear dynamic analysis	Cross-section size, steel reinforcement
Li et al.	2010	RC frame-shear-wall structures	A hybrid of Genetic Algorithm (GA) and Optimality Criteria (OC)	-	Section size of structural member
Akin and Saka	2012	RC frames	Harmony Search algorithm	Performances calculated through matrix displacement method	section dimensions and arrangement of longitudinal reinforcement (i.e. number and diameter of rebar)

Akin and Saka	2015	RC frames	Harmony Search algorithm	Performances calculated through matrix displacement method	section dimensions, longitudinal and transverse reinforcement
Gharehbaghi and Khatibinia	2015	RC frames	Particle Swarm Optimisation (PSO)	Average response calculated through intelligent regression model	section dimensions, longitudinal reinforcement
Gharehbaghi et al.	2023	RC frames	Three improved metaheuristic optimisation	Non-linear static analysis	section dimensions, area of steel reinforcement
Esfandiari et al.	2018	RC frames	A hybrid of Multi-criterion Decision-making (DM) and PSO	Non-linear dynamic analysis	section sizes, number and diameter of reinforcement at specific locations
Mergos	2017	RC frames	Genetic Algorithm (GA)	Linear dynamic analysis, Non-linear dynamic analysis	section dimensions, longitudinal and transverse reinforcement
Mergos	2018	RC frames	Genetic Algorithm (GA)	Non-linear static analysis, Non-linear dynamic analysis	section dimensions, longitudinal and transverse reinforcement
Mergos	2020	Regular RC frame and RC frame with setbacks	Genetic Algorithm (GA)	Non-linear static analysis, Non-linear dynamic analysis	section dimensions, longitudinal and transverse reinforcement
Razmara Shooli et al.	2019	Moment-resisting RC frames	A hybrid of GA and PSO	Non-linear static analysis, Non-linear dynamic analysis	sectional dimensions, longitudinal reinforcements
Liu et al.	2010	RC frames	Gradient-based first and second order optimisation	Response calculated based on Newmark-β method	Width and depth of structural member
Zhang and Tian	2019	RC frames	A feasible region boundary for corresponding variables	Non-linear static analysis	overall system stiffness (factor) and overall system strength (factor)
Gholizadeh and Aligholizadeh	2019	RC frames	Chaotic Enhanced Colliding Bodies Optimisation (CECBO)	A metamodel composed of NN (neural network techniques) and WBP (wavelet	sectional dimensions, arrangement of reinforcements

				back propagation)	
Seify Asghshahr	2021	RC frames	Genetic Algorithm (GA)	Linear static analysis	Cross-section sizes
Lavan and Wilkinson	2017	3D Irregular RC frames	Analysis-Redesign approaches	3D non-linear dynamic analysis	Normal flexural strength of structural member
Lagaros and Fragiadakis	2011	3D Regular/ Irregular RC frames	Evolutionary Strategies Algorithm	Non-linear static analysis	dimensions of beam and column, longitudinal reinforcements
Razavi and Gholizadeh	2021	RC frames	Improved black hole algorithm	Non-linear static analysis	Cross-section dimensions and number of reinforcing bars

Multi-objective optimisation:

Table A.3. Research developments in design optimisation with multiple objectives

Researcher	Year	Structure	Optimisation methodology	Seismic analysis	Design Variables
Lagaros and Papadrakakis	2007	3D RC frame	Non-dominated Sorting Evolution Strategies Algorithm	Linear static analysis, Non-linear static analysis	Section dimensions of columns
Gharehbaghi	2018	RC frame	Particle Swarm Optimisation (PSO)	Non-linear dynamic analysis	Sectional dimensions, renforcements ratio
Möller et al.	2009	RC frame	A search-based numerical algorithm	Response calculated using neural network	geometric and structural properties, earthquake characteristics
Möller et al.	2015	RC frame	A search-based numerical algorithm	Response calculated using neural network	Section dimensions, reinforcement, earthquake characteristics
Khatibinia et al.	2013	RC frame	Gravitational search algorithm	A metamodel composed of weighted least squares support vector machine and wavelet kernel function	sectional dimensions, diameters of longitudinal reinforcements

Yazdani et al.	2017	RC frame	Modified discrete Gravitational search algorithm	A metamodel composed of weighted least squares support vector machine and wavelet kernel function	section dimensions, diameters of longitudinal reinforcements
Zou et al.	2007	RC frame	Optimality Criteria algorithm, ε-constraint method	Non-linear static analysis	section dimensions, reinforcements quantities
Mitropoulou et al.	2011	3D regular and irregular RC frame	Non-dominated Sorting Evolution Strategies Algorithm	Non-linear static analysis, Non-linear dynamic analysis	section dimensions, longitudinal and transverse reinforcement
Asadi and Hajirasouliha	2020	RC frame	Uniform Damage Distribution	Non-linear dynamic analysis	Area of longitudinal reinforcement
Nouri et al.	2020	RC frame	Analysis-Redesign approaches	Response predicted by simple response function	section dimensions, reinforcements ratios

APPENDIX B

Initial Design Results (Eurocode-based)

Table B.1. Cross-section design details of Beams, 3-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcements at Top			Reinforcements on Bottom		
		Ratio, (%)	Total Number	Rebar Diameter, (mm)	Ratio, (%)	Total Number	Rebar Diameter, (mm)
1	40 x 35	0.67	3	20	0.43	3	16
2	30 x 30	1.10	5	16	0.45	2	16
3	30 x 30	1.10	5	16	0.45	2	16

Table B.2. Cross-section design details of Columns, 3-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcement Ratio (%)	Rebar Number Main / Intermediate / Corner	Rebar Diameter, Side / Corner (mm)
1	40 x 40	2.36	2 / 2 / 4	20 / 20
2	35 x 35	1.97	2 / 2 / 4	16 / 16
3	35 x 35	1.97	2 / 2 / 4	16 / 16

Table B.3. Cross-section design details of Beams, 5-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcements at Top			Reinforcements on Bottom		
		Ratio, (%)	Total Number	Rebar Diameter, (mm)	Ratio, (%)	Total Number	Rebar Diameter, (mm)
1	50 x 45	0.36	4	16	0.27	3	16
2	40 x 35	0.86	6	16	0.57	4	16
3	35 x 30	0.96	5	16	0.57	3	16
4	30 x 30	1.12	5	16	0.67	3	16
5	30 x 30	1.12	5	16	0.67	3	16

Table B.4. Cross-section design details of Columns, 5-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcement Ratio (%)	Rebar Number Main / Intermediate / Corner	Rebar Diameter, Side / Corner (mm)
1	50 x 50	2.01	3 / 3 / 4	20 / 20
2	40 x 40	2.36	2 / 2 / 4	20 / 20
3	35 x 35	1.97	2 / 2 / 4	16 / 16
4	30 x 30	2.68	2 / 2 / 4	16 / 16
5	30 x 30	2.68	2 / 2 / 4	16 / 16

Table B.5. Cross-section details of Beams, 10-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcements at Top			Reinforcements on Bottom		
		Ratio, (%)	Total Number	Rebar Diameter, (mm)	Ratio, (%)	Total Number	Rebar Diameter, (mm)
1	50 x 45	0.70	5	20	0.42	3	20
2	50 x 45	0.70	5	20	0.42	3	20
3	45 x 40	0.87	5	20	0.45	4	16
4	45 x 40	0.87	5	20	0.45	4	16
5	40 x 35	0.86	6	16	0.57	4	16
6	40 x 35	0.86	6	16	0.57	4	16
7	40 x 35	0.86	6	16	0.57	4	16
8	40 x 35	0.86	6	16	0.57	4	16
9	30 x 30	1.12	5	16	0.45	2	16
10	30 x 30	1.12	5	16	0.45	2	16

Table B.6. Cross-section design details of Columns 10-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcement Ratio (%)	Rebar Number Main / Intermediate / Corner	Rebar Diameter, Side / Corner (mm)
1	50 x 50	2.01	3 / 3 / 4	20 / 20
2	50 x 50	2.01	3 / 3 / 4	20 / 20
3	45 x 45	1.86	2 / 2 / 4	20 / 20
4	45 x 45	1.86	2 / 2 / 4	20 / 20
5	40 x 40	2.36	2 / 2 / 4	20 / 20
6	40 x 40	2.36	2 / 2 / 4	20 / 20
7	40 x 40	2.36	2 / 2 / 4	20 / 20
8	40 x 40	2.36	2 / 2 / 4	20 / 20
9	30 x 30	1.79	1 / 1 / 4	16 / 16
10	30 x 30	1.79	1 / 1 / 4	16 / 16

Table B.7. Cross-section design details of Beams, 15-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcements at Top			Reinforcements on Bottom		
		Ratio, (%)	Total Number	Rebar Diameter, (mm)	Ratio, (%)	Total Number	Rebar Diameter, (mm)
1	50 x 50	0.75	6	20	0.50	4	20
2	50 x 50	0.75	6	20	0.50	4	20
3	50 x 50	0.75	6	20	0.50	4	20
4	45 x 40	1.05	6	20	0.70	4	20
5	45 x 40	1.05	6	20	0.70	4	20
6	45 x 40	1.05	6	20	0.70	4	20
7	45 x 40	1.05	6	20	0.70	4	20
8	40 x 35	1.12	5	20	0.67	3	20
9	40 x 35	1.12	5	20	0.67	3	20
10	40 x 35	1.12	5	20	0.67	3	20
11	40 x 35	1.12	5	20	0.67	3	20
12	40 x 35	1.12	5	20	0.67	3	20
13	35 x 30	1.15	6	16	0.77	4	16
14	35 x 30	1.15	6	16	0.77	4	16
15	35 x 30	1.15	6	16	0.77	4	16

Table B.8. Cross-section design details of Columns, 15-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcement Ratio (%)	Rebar Number Main / Intermediate / Corner	Rebar Diameter, Side / Corner (mm)
1	55 x 55	1.66	3 / 3 / 4	20 / 20
2	55 x 55	1.66	3 / 3 / 4	20 / 20
3	55 x 55	1.66	3 / 3 / 4	20 / 20
4	45 x 45	1.86	2 / 2 / 4	20 / 20
5	45 x 45	1.86	2 / 2 / 4	20 / 20
6	45 x 45	1.86	2 / 2 / 4	20 / 20
7	45 x 45	1.86	2 / 2 / 4	20 / 20
8	45 x 45	1.86	2 / 2 / 4	20 / 20
9	40 x 40	2.36	2 / 2 / 4	20 / 20
10	40 x 40	2.36	2 / 2 / 4	20 / 20
11	40 x 40	2.36	2 / 2 / 4	20 / 20
12	40 x 40	2.36	2 / 2 / 4	20 / 20
13	35 x 35	1.97	2 / 2 / 4	16 / 16
14	35 x 35	1.97	2 / 2 / 4	16 / 16
15	35 x 35	1.97	2 / 2 / 4	16 / 16

APPENDIX C

Optimum Design Results

Table C.1. Cross-section design details of Beams, 3-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcements at Top			Reinforcements on Bottom		
		Ratio, (%)	Total Number	Rebar Diameter, (mm)	Ratio, (%)	Total Number	Rebar Diameter, (mm)
1	30 x 30	0.33	6	8	0.33	6	8
2	40 x 35	0.79	3	22	0.33	4	12
3	30 x 30	1.21	3	22	0.49	4	12

Table C.2. Cross-section design details of Columns, 3-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcement Ratio (%)	Rebar Number Main / Intermediate / Corner	Rebar Diameter, Side / Corner (mm)
1	40 x 40	3.42	3 / 2 / 4	22 / 22
2	40 x 40	1.10	3 / 3 / 4	12 / 12
3	30 x 30	3.07	1 / 1 / 4	22 / 20

Table C.3. Cross-section design details of Beams, 5-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcements at Top			Reinforcements on Bottom		
		Ratio, (%)	Total Number	Rebar Diameter, (mm)	Ratio, (%)	Total Number	Rebar Diameter, (mm)
1	30 x 30	0.33	6	8	0.33	6	8
2	45 x 40	0.33	4	14	0.33	4	14
3	40 x 35	0.33	4	12	0.33	4	12
4	40 x 35	0.35	6	10	0.35	6	10
5	30 x 30	0.73	6	12	0.44	5	10

Table C.4. Cross-section design details of Columns, 5-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcement Ratio (%)	Rebar Number Main / Intermediate / Corner	Rebar Diameter, Side / Corner (mm)
1	45 x 45	1.02	2 / 1 / 4	16 / 16
2	45 x 45	1.00	2 / 1 / 4	16 / 16
3	40 x 40	1.00	1 / 1 / 4	16 / 16
4	40 x 40	1.01	1 / 1 / 4	16 / 16
5	30 x 30	2.13	3 / 1 / 4	16 / 14

Table C.5. Cross-section details of Beams, 10-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcements at Top			Reinforcements on Bottom		
		Ratio, (%)	Total Number	Rebar Diameter, (mm)	Ratio, (%)	Total Number	Rebar Diameter, (mm)
1	45 x 40	0.32%	5	12	0.32%	5	12
2	45 x 40	0.32%	5	12	0.32%	5	12
3	40 x 35	0.32%	4	12	0.32%	4	12
4	35 x 30	1.22%	4	20	0.62%	6	12
5	35 x 30	1.46%	5	20	0.97%	5	16
6	35 x 30	0.97%	5	16	0.65%	6	12
7	35 x 30	1.01%	5	16	0.67%	6	12
8	30 x 30	1.17%	5	16	0.78%	6	12
9	35 x 30	0.98%	5	16	0.47%	6	10
10	35 x 30	0.35%	2	16	0.35%	2	16

Table C.6. Cross-section design details of Columns 10-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcement Ratio (%)	Rebar Number Main / Intermediate / Corner	Rebar Diameter, Side / Corner (mm)
1	45 x 45	1.00%	2 / 1 / 4	16 / 16
2	45 x 45	1.00%	2 / 1 / 4	16 / 16
3	40 x 40	1.00%	1 / 1 / 4	16 / 16
4	35 x 35	1.87%	3 / 3 / 4	14 / 12
5	35 x 35	2.44%	3 / 3 / 4	16 / 14
6	35 x 35	1.89%	3 / 3 / 4	14 / 12
7	35 x 35	1.53%	3 / 1 / 4	14 / 14
8	35 x 35	1.45%	3 / 3 / 4	12 / 12
9	35 x 35	1.53%	3 / 1 / 4	14 / 14
10	35 x 35	1.42%	3 / 3 / 4	12 / 12

Table C.7. Cross-section design details of Beams, 15-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcements at Top			Reinforcements on Bottom		
		Ratio, (%)	Total Number	Rebar Diameter, (mm)	Ratio, (%)	Total Number	Rebar Diameter, (mm)
1	45 x 40	0.60%	6	16	0.40%	4	16
2	45 x 40	0.60%	6	16	0.40%	4	16
3	45 x 40	0.60%	6	16	0.40%	4	16
4	45 x 40	0.80%	5	20	0.53%	5	16
5	40 x 35	0.80%	6	16	0.53%	7	12
6	35 x 30	0.81%	4	16	0.55%	5	12
7	40 x 35	1.09%	5	20	0.73%	5	16
8	35 x 30	0.30%	4	10	0.30%	4	10
9	40 x 35	1.30%	6	20	0.78%	6	16
10	40 x 35	0.60%	4	16	0.38%	5	12
11	40 x 35	0.81%	4	20	0.49%	6	12
12	35 x 30	1.10%	4	20	0.66%	6	12
13	35 x 30	1.25%	7	16	0.84%	3	20
14	30 x 30	1.48%	7	16	0.99%	3	20
15	30 x 30	0.61%	3	16	0.40%	2	16

Table C.8. Cross-section design details of Columns, 15-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcement Ratio (%)	Rebar Number Main / Intermediate / Corner	Rebar Diameter, Side / Corner (mm)
1	45 x 45	1.80%	3 / 1 / 4	20 / 20
2	45 x 45	1.00%	2 / 1 / 4	16 / 16
3	45 x 45	1.00%	2 / 1 / 4	16 / 16
4	45 x 45	1.00%	2 / 1 / 4	16 / 16
5	40 x 40	1.00%	1 / 1 / 4	16 / 16
6	40 x 40	1.18%	2 / 1 / 4	16 / 16
7	40 x 40	1.76%	3 / 2 / 4	16 / 16
8	40 x 40	1.00%	1 / 1 / 4	16 / 16
9	40 x 40	1.74%	3 / 2 / 4	16 / 16
10	40 x 40	1.06%	3 / 3 / 4	12 / 10
11	40 x 40	1.07%	3 / 3 / 4	12 / 10
12	35 x 35	1.76%	3 / 2 / 4	14 / 14
13	35 x 35	1.60%	2 / 1 / 4	16 / 16
14	30 x 30	3.56%	3 / 3 / 4	16 / 16
15	30 x 30	2.85%	2 / 2 / 4	20 / 14

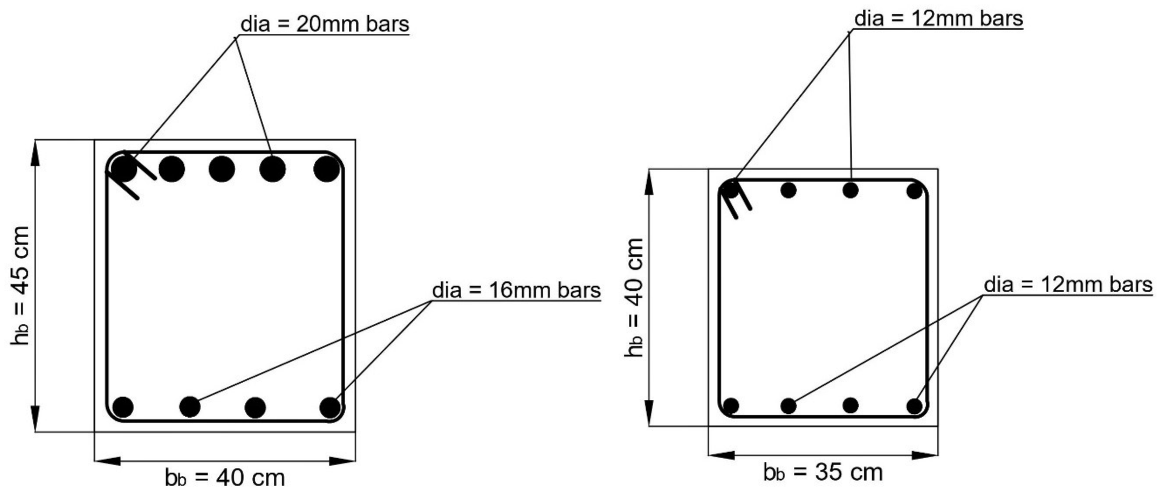

Figure C1: Cross-section Drawings of Beam Sections in 3rd Storey for 10-storey RC Frame, for Initial Design (left) and Optimum Design (right) (h_b: height of beam; b_b: width of beam; dia: diameter of rebar)

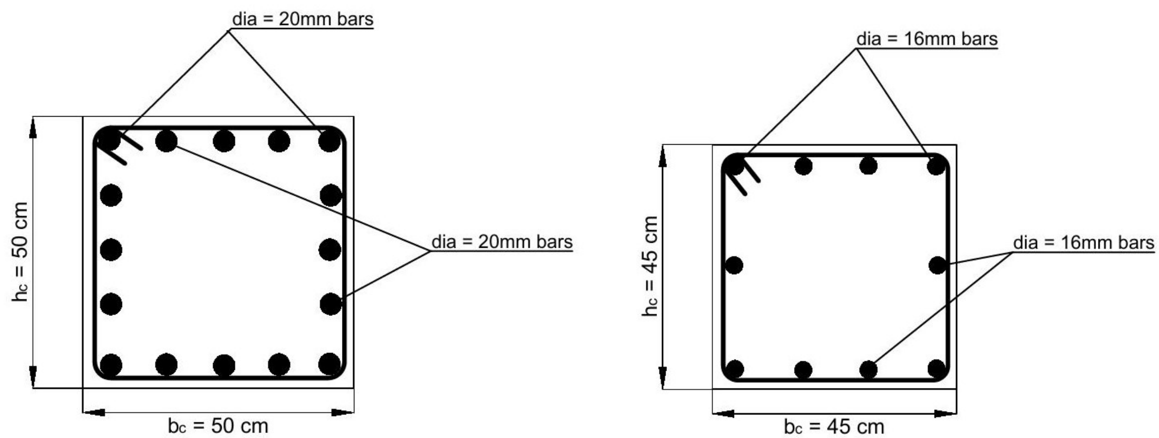

Figure C2: Cross-section Drawings of Column Sections in 1st Storey for 5-storey RC Frame, for Initial Design (left) and Optimum Design (right) (h_c: height of column; b_c: width of column; dia: diameter of rebar)

APPENDIX D

Mean Natural Optimum Design Results

Table D.1. Cross-section design details of Beams, 3-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcements at Top			Reinforcements on Bottom		
		Ratio, (%)	Total Number	Rebar Diameter, (mm)	Ratio, (%)	Total Number	Rebar Diameter, (mm)
1	30 x 30	0.33	6	8	0.33	6	8
2	40 x 35	1.02	4	22	0.42	5	12
3	30 x 30	1.12	5	16	0.46	2	16

Table D.2. Cross-section design details of Columns, 3-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcement Ratio (%)	Rebar Number Main / Intermediate / Corner	Rebar Diameter, Side / Corner (mm)
1	40 x 40	3.39	3 / 2 / 4	22 / 22
2	40 x 40	1.16	3 / 1 / 4	14 / 14
3	30 x 30	3.06	1 / 1 / 4	22 / 20

Table D.3. Cross-section design details of Beams, 5-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcements at Top			Reinforcements on Bottom		
		Ratio, (%)	Total Number	Rebar Diameter, (mm)	Ratio, (%)	Total Number	Rebar Diameter, (mm)
1	30 x 30	0.33	6	8	0.33	6	8
2	40 x 35	0.34	3	14	0.34	3	14
3	40 x 35	0.67	8	12	0.40	5	12
4	40 x 35	0.33	6	10	0.33	6	10
5	30 x 30	0.52	4	12	0.39	4	10

Table D.4. Cross-section design details of Columns, 5-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcement Ratio (%)	Rebar Number Main / Intermediate / Corner	Rebar Diameter, Side / Corner (mm)
1	45 x 45	1.34	3 / 2 / 4	16 / 16
2	40 x 40	1.00	1 / 1 / 4	16 / 16
3	40 x 40	1.10	1 / 1 / 4	16 / 16
4	40 x 40	1.00	1 / 1 / 4	16 / 16
5	30 x 30	1.92	2 / 1 / 4	16 / 14

Table D.5. Cross-section details of Beams, 10-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcements at Top			Reinforcements on Bottom		
		Ratio, (%)	Total Number	Rebar Diameter, (mm)	Ratio, (%)	Total Number	Rebar Diameter, (mm)
1	45 x 40	0.65	6	16	0.40	7	12
2	45 x 40	0.65	6	16	0.40	7	12
3	40 x 35	0.69	5	16	0.43	6	12
4	35 x 30	1.63	6	20	1.08	6	16
5	35 x 30	1.41	5	20	0.93	5	16
6	35 x 30	1.38	7	16	0.91	8	12
7	35 x 30	1.24	4	20	0.82	8	12
8	30 x 30	1.50	7	16	1.00	8	12
9	35 x 30	1.02	5	16	0.47	6	10
10	35 x 30	0.32	2	16	0.32	2	16

Table D.6. Cross-section design details of Columns 10-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcement Ratio (%)	Rebar Number Main / Intermediate / Corner	Rebar Diameter, Side / Corner (mm)
1	45 x 45	1.43	3 / 2 / 4	16 / 16
2	45 x 45	1.00	2 / 1 / 4	16 / 16
3	40 x 40	1.08	2 / 1 / 4	16 / 14
4	40 x 40	1.84	3 / 3 / 4	16 / 14
5	35 x 35	2.87	4 / 3 / 4	16 / 16
6	35 x 35	2.64	5 / 4 / 4	14 / 12
7	35 x 35	2.01	4 / 2 / 4	14 / 14
8	35 x 35	1.71	2 / 2 / 4	16 / 14
9	35 x 35	1.95	3 / 3 / 4	14 / 14
10	35 x 35	1.15	2 / 2 / 4	12 / 12

Table D.7. Cross-section design details of Beams, 15-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcements at Top			Reinforcements on Bottom		
		Ratio, (%)	Total Number	Rebar Diameter, (mm)	Ratio, (%)	Total Number	Rebar Diameter, (mm)
1	45 x 40	0.90	5	20	0.60	6	16
2	45 x 40	0.90	5	20	0.60	6	16
3	45 x 40	0.90	5	20	0.60	6	16
4	45 x 40	1.10	6	20	0.80	5	20
5	40 x 35	1.10	5	20	0.80	6	16
6	35 x 30	1.45	5	20	1.01	5	16
7	35 x 30	2.07	7	20	1.55	5	20
8	40 x 35	0.73	5	16	0.49	5	14
9	40 x 35	1.13	5	20	0.76	5	16
10	35 x 30	1.39	5	20	0.92	5	16
11	35 x 30	1.27	4	20	0.85	6	14
12	40 x 35	0.76	5	16	0.46	4	14
13	35 x 30	1.17	6	16	0.78	4	16
14	30 x 30	0.95	4	16	0.64	5	12
15	30 x 30	0.79	6	12	0.53	6	10

Table D.8. Cross-section design details of Columns, 15-storey frames

Storey Level	Height x Width (cm x cm)	Reinforcement Ratio (%)	Rebar Number Main / Intermediate / Corner	Rebar Diameter, Side / Corner (mm)
1	45 x 45	3.29	5 / 4 / 4	20 / 20
2	45 x 45	2.00	3 / 2 / 4	20 / 16
3	45 x 45	2.00	3 / 2 / 4	20 / 16
4	45 x 45	2.00	4 / 4 / 4	16 / 16
5	40 x 40	2.00	4 / 2 / 4	16 / 16
6	40 x 40	2.35	4 / 4 / 4	16 / 14
7	40 x 40	2.10	2 / 2 / 4	20 / 16
8	40 x 40	1.27	4 / 3 / 4	12 / 12
9	40 x 40	1.78	4 / 1 / 4	16 / 16
10	40 x 40	1.14	4 / 2 / 4	12 / 12
11	40 x 40	1.02	1 / 1 / 4	16 / 16
12	40 x 40	1.07	2 / 1 / 4	16 / 14
13	35 x 35	1.81	4 / 4 / 4	12 / 12
14	35 x 35	1.64	2 / 1 / 4	16 / 16
15	30 x 30	2.56	3 / 3 / 4	14 / 12

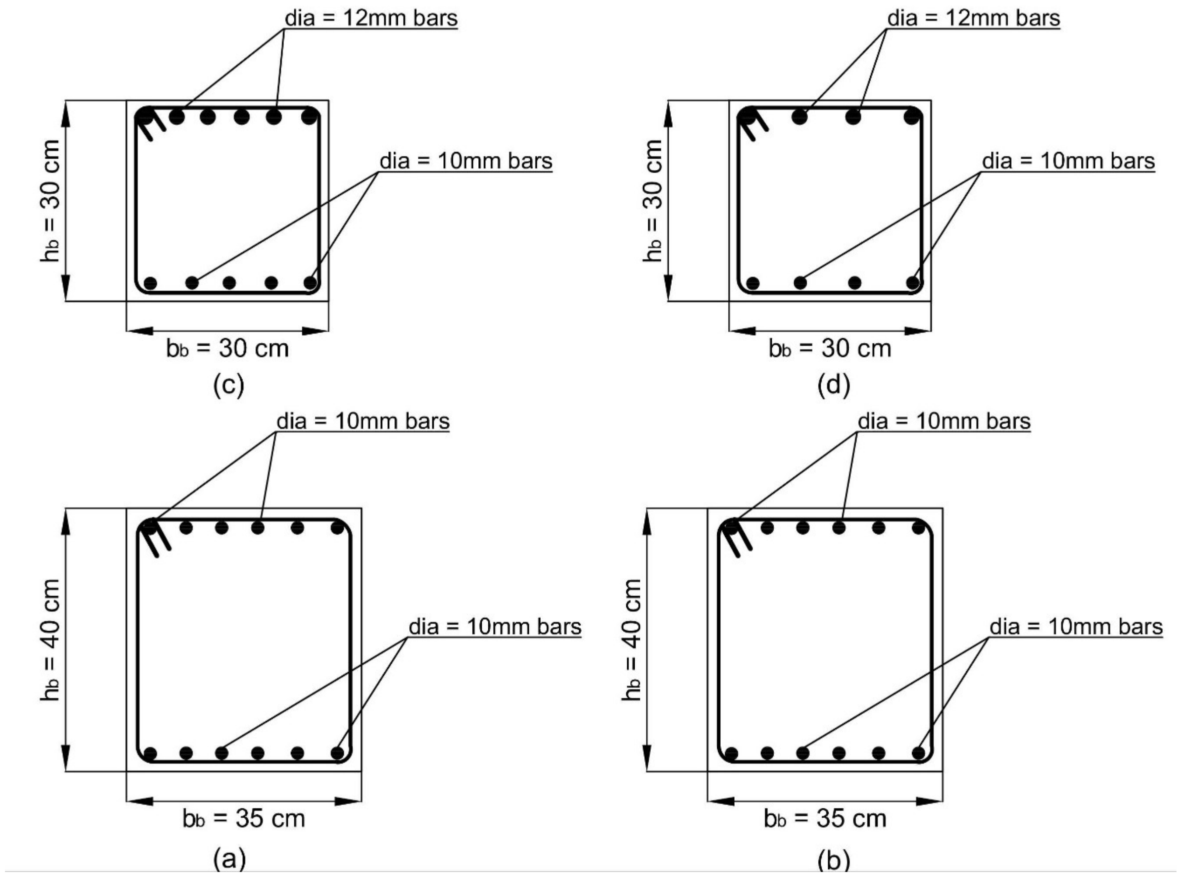

Figure D1: Cross-section Drawings of Beam Sections for 5-storey RC Frame, for (a) Artificial Optimum Design in the 4th storey, (b) Natural Optimum Design in the 4th storey, (c) Artificial Optimum Design in the 5th storey, (d) Natural Optimum Design in the 5th storey (h_b: height of beam; b_b: width of beam; dia: diameter of rebar)

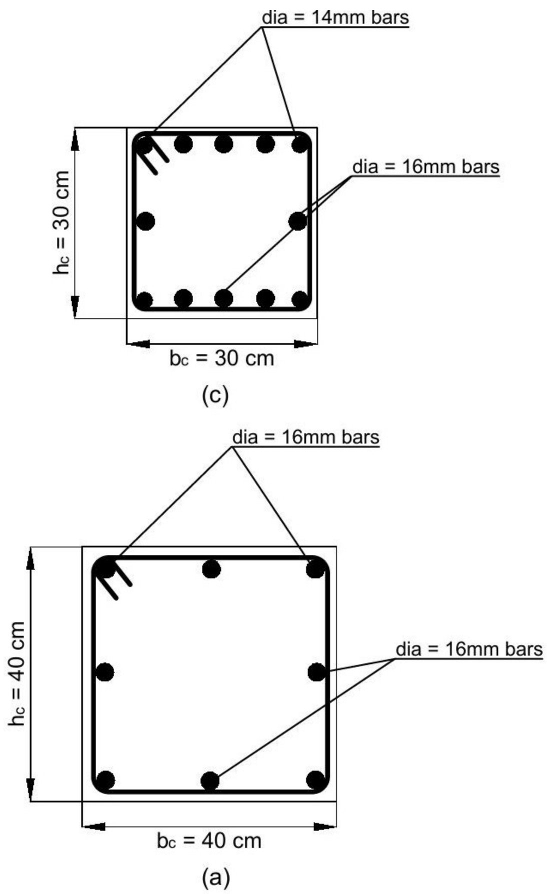

Figure D2: Cross-section Drawings of Column Sections for 5-storey RC Frame, for (a) Artificial Optimum Design in the 4th storey, (b) Natural Optimum Design in the 4th storey, (c) Artificial Optimum Design in the 5th storey, (d) Natural Optimum Design in the 5th storey (h_b: height of beam; b_b: width of beam; dia: diameter of rebar)