

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

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ABSTRACT

Reinforced concrete (RC) frames are extensively used in construction of buildings worldwide. The conventional design of these buildings typically relies on a "trial and error" method, starting with initial dimensioning followed by validation of the preliminary design. This approach makes it difficult to calculate potential cost savings in materials, while maintaining structural safety. In recent decades, the need for more efficient, safe and rational seismic design procedures has led to an increasing interest in structural optimisation of RC frames. However, due to the complex non-linear behaviour of RC frames, arriving at their optimum design under earthquake excitations is challenging. Even at serviceability limit state, concrete cracking can lead to significant stiffness changes and redistribution of inertial forces, phenomena that are normally expected after steel yielding. To mitigate these drawbacks, numerous combinations of section dimensions and reinforcement arrangements can be considered as design variables during a structural size optimisation process. However, this is seldom done for RC frames, as it requires seismic analyses with a high level of accuracy, which can be computationally expensive. Meanwhile, design optimisation has been identified as a feasible way to reduced embodied carbon emissions in construction, and therefore, a change of design paradigms is necessary. The article aims to critically review the major developments in recent seismic design optimisation studies on RC buildings, focusing on structural size optimisation, with the goal of identifying the main achievements and limitations. Different design objectives in current seismic design optimisation procedures are first summarised. The key steps in structural optimisation are then discussed, including design variables, design constraints, application of optimization methodologies, and evaluation of seismic performance. Finally, the research gaps in this field are identified, and suggestions are provided for future research directions. This article contributes towards the development of more efficient seismic design methods for RC structures, which in turn can lead to more sustainable construction.

1. Introduction

Substandard reinforced concrete (RC) buildings that are gravity load designed without considering seismic design guidelines have experienced irreparable structural damage during previous seismic events, as confirmed in previous studies [1,2]. In recent years, the seismic design of the RC buildings has been re-evaluated following destructive earthquakes around the world. Indeed, whilst modern seismic design codes have been adopted in most seismic-prone regions, post-earthquake surveys indicate that the seismic performances of many RC frames are still susceptible to soft-storey failures, particularly in lower stories and under severe earthquakes [3–6]. This observation is also confirmed by further experimental and numerical studies by Feng et al. [7] and Lu

et al. [8] that indicated code-based structures do not generally 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 [9], 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. Furthermore, economic loss due to structural and non-structural damage can be unexpectedly high, even if the design solution successfully ensures life safety. As an example, the earthquake

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occurred in 2012 in Northern Italy resulted in 27 casualties, while caused significant damage to public and private buildings with an estimated overall economic loss of approximately 13 billion Euro [10].

The increasing demand for safe and cost-efficient seismic designs, has driven the development of structural optimisation of buildings. Moreover, in light of the push for net-zero construction, structural optimisation is deemed a feasible solution to reduce the embodied carbon of building constructions [11]. The general steps required in design optimisation problems are summarised by Arora [12]. The first step is described as “*statement of work*” (or *design problem*), in which the overall objectives of the projects and corresponding requirements are stated. However, usually there are several feasible designs for a specific structural system, while the efficiency of each design is identified by an “*objective function*” that mathematically describes the design problem formulated based on the selected “*design variables*”. The best solution is then obtained by minimising (or maximising) the numerical value of this function in an optimisation problem. An optimal design should meet both general and specific design requirements, called “*design constraints*”, which often depend on the selected design variables. To solve the optimisation problem once it is formulated, different “*optimisation methods*” can be used. An important step is to assess the performance of the initial and candidate designs during the optimisation process, which is achieved through “*analysis procedures*”.

In structural size optimisation, the structural properties of each element (e.g. dimensions of the cross sections, and longitudinal and transverse reinforcement ratios) are optimised to achieve the best solution of the optimisation objective function, while imposing a range of geometry and boundary constraints. In this case, the location and number of structural elements are fixed. Seismic design problems are generally dealt with in the category of size optimisation, where the shape and topology of structural elements and the structural material properties are pre-determined and kept constant throughout the optimisation process [13,14].

To address the limitations inherent in conventional “force-based” seismic design methods, which generally rely on “trial-verification-modification” processes, previous studies have proposed alternative seismic design methods. For instance, Priestley and Kowalsky [15] developed a direct displacement-based seismic design approach for determining base shear forces and a more accurate distribution of the shear force along the storey height to meet specified displacement-based limit states, without the need for iterative analysis. The principles of displacement-based design were further developed by Gentile and Calvi [16] into a loss-based seismic design approach for RC buildings, aiming to achieve a targeted level of economic loss corresponding to building damage within a few iterations. While this method can reduce computational efforts associated with repetitive applications of the “trial-verification-modification” processes, it is more preferable for conceptual or preliminary design phases.

Performance-based seismic design (PBSD) can be seen as an evolution of the displacement-based design principle, and it is adopted in several seismic design guidelines (e.g. [17–20]). Compared to the conventional “force-based” seismic design method, PBSD expresses design criteria directly related to local (i.e. element deformations) or global (i.e. inter-storey drift) structural responses so that they meet specific performance requirements for buildings (e.g. immediate occupancy, life safety and collapse prevention). Hence, PBSD offers a more rational approach to control both structural and non-structural damage, satisfying different performance objectives corresponding to multiple seismic hazard levels. PBSD can be incorporated into structural optimisations to produce more reliable designs that achieve specific design objectives and satisfy multiple performance-based objectives. However, implementing performance-based structural optimisation for RC frames can be challenging, as the structural response within the non-linear range can be significantly affected by various design variables. For example, previous studies have been shown that while an increase in reinforcement ratio will increase the construction cost, it does not

necessarily improve the seismic performance [19–22].

To date, only a few review papers have specifically focused on the structural optimisation of engineering structures under seismic loads [23,24]. Moreover, current research in the field is fragmented and sometimes inconsistent. Besides, there is a lack of a critical and comprehensive review focused on the seismic design optimization of RC frames, which represent a complex structural system with multiple interdependent design variables that significantly affect the optimal solutions. As such, this paper critically reviews the major developments in recent seismic design optimizations specifically for RC buildings. The presented work primarily focuses on structural size optimization, aiming to provide detailed explanations of the strengths and limitations of previously adopted optimization approaches and discussing different design objectives, including structural safety, cost efficiency, and sustainability. The research gaps and future directions are then identified, and practical suggestions are provided for identifying the most suitable seismic analysis and optimisation methods for regular and irregular buildings located in either low or high seismic region to achieve safer and more efficient seismic designs of RC structures. Fig. 1 summarises the different concepts of seismic design optimisation process discussed in this study, which will be elaborated on in the following sections.

2. Objective of seismic design optimisation

In structural optimisation, an optimisation formula is needed to describe the design problem and define the design objectives and constraints. The design objectives can be single or multiple, with the latter being generally more complex. Fig. 2 summarises the design objectives used in the existing literature on optimum design of RC structures, categorising them into single-objective and multi-objective optimisation. The figure also provides details on the procedures adopted to achieve each design objective in each case. The following sections expand on the different items shown in the figure.

2.1. Single-objective optimisation

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

$$\text{Min} F = F(x) \quad (1)$$

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

$$x \in \{x_1, x_2, \dots, x_j\} \quad j = 1, 2, \dots, N_j$$

where $F(x)$ represents the design objective function, g_i denotes the inequality design constraints, N_i is the total number of constraints, x represents the design vector containing all the selected design variables, and N_j is the total number of variables.

“Single-objective optimisation” 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.

2.1.1. Minimum structural damage optimisation

Direct minimum structural damage optimisations generally aim to reduce the local concentration of seismic demand and obtain a more uniform distribution of damage. In this case, the objective function $F(x)$

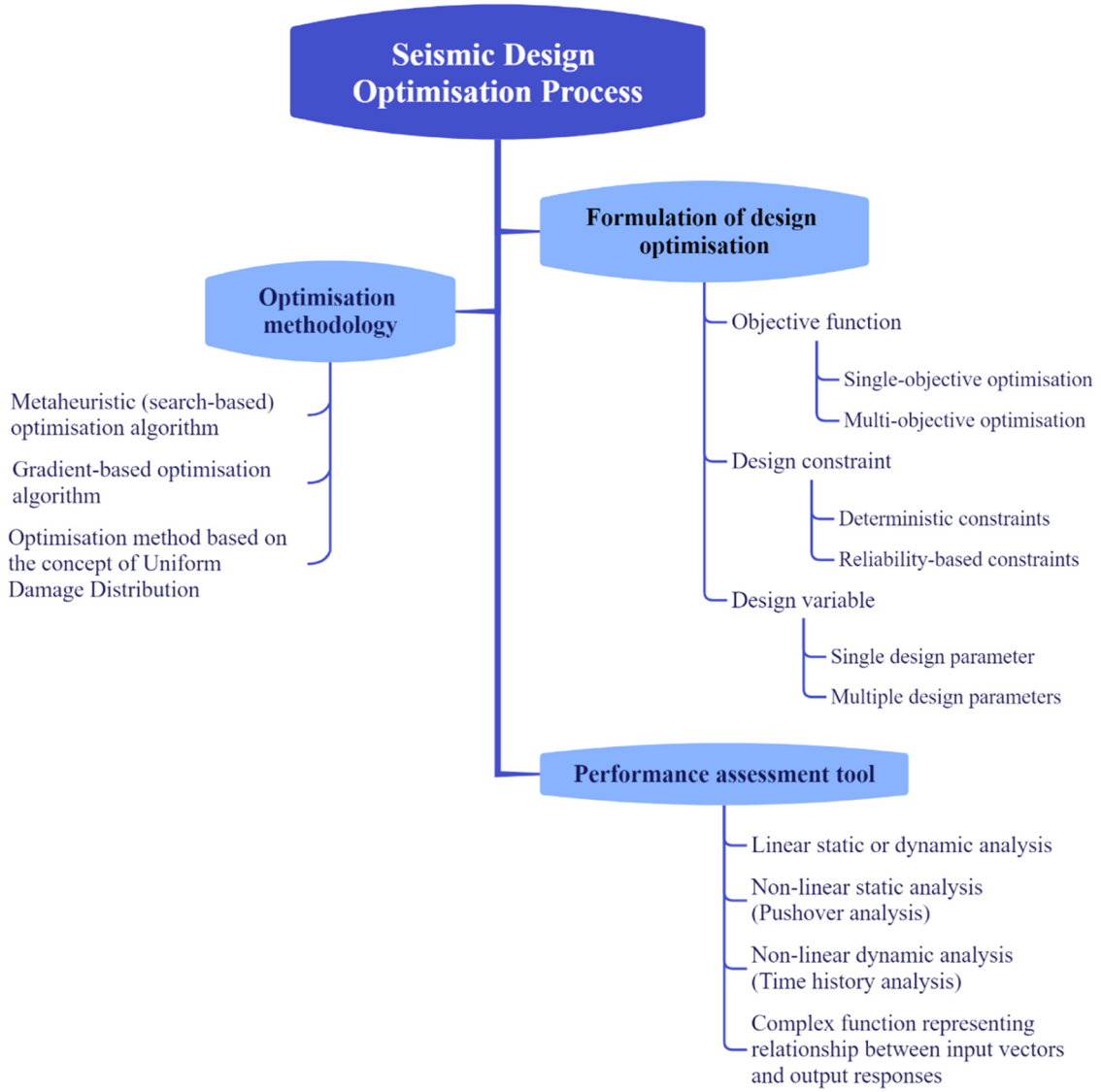


Fig. 1. Concepts of seismic design optimisation process discussed in this study.

is usually formulated with reference to the selected structural seismic response as follows:

$$\text{Min}F(x) = \frac{IDR_i}{IDR_0} \quad (2)$$

where IDR_0 and IDR_i are the structural maximum responses for the initial design and for the design in i^{th} iterative step, respectively. The parameter IDR can also be any performance parameter, such as inter-storey drift ratio, plastic hinge rotations, storey ductility demand, local or global damage index, and coefficient of variation (COV) of the seismic response.

2.1.2. Optimum lateral load pattern

Most force-based seismic design guidelines use equivalent static lateral forces to represent seismic excitations. The distribution of these lateral forces along the height is mainly derived based on elastic vibration response of the system, and can 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 the inelastic range [25,26]. 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, while the design procedure remains unchanged.

In one of the relevant studies, Varughese et al. [27] 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 (i.e. Chao load distribution) experienced more uniform damage and inter-storey drift distributions. This can be explained as the Chao load distribution considers the contribution of higher modes that are especially important in high-rise buildings. To increase structural resistance capacity at collapse state under earthquakes, Li et al. [28] 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 then used to determine the optimum lateral load pattern. The results demonstrated that, compared to code-based designs, frames designed based on optimum lateral force patterns were less likely to collapse and met storey drift limits under multiple seismic intensity levels.

2.1.3. More uniform damage distribution

Most force-based design codes (e.g. Eurocode 8 [9] and IBC 2021 [29]) determine the resistance of members under design forces derived

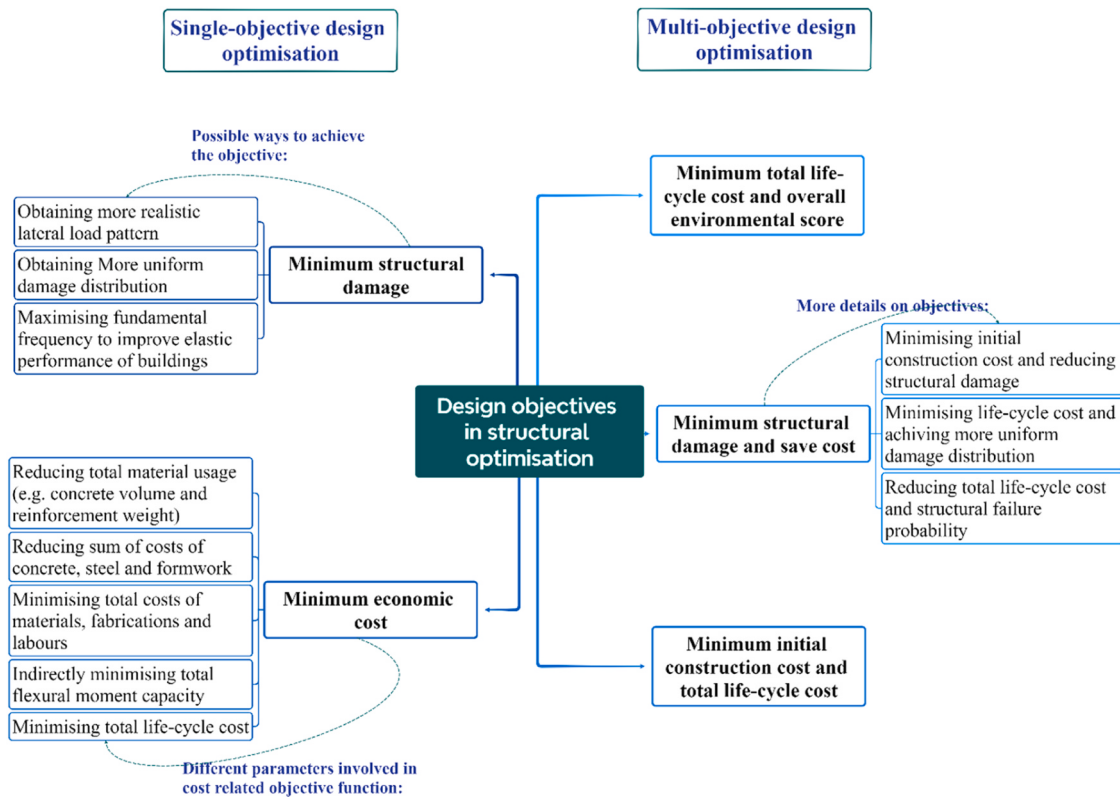


Fig. 2. Details on different design objective(s) selected in the optimisation frameworks reviewed in this study.

from a linear elastic system. Displacement demands are then checked at end of the design process, but without explicitly considering the effect of nonlinear behaviour of the structure on the calculated design forces [25, 30]. To address this issue, PBSO methods directly control structural damage under multiple seismic hazard levels, and hence can be used to attain a more uniform structural damage and better use of materials to withstand seismic loads.

As an example, Hajirasouliha et al. study [31] adopted a PBSO optimisation method to minimise structural damage by limiting inter-storey drifts of multi-storey RC frames to a Life Safety (LS) performance level. The total material usage was kept constant through the optimisation process to ensure the initial costs were not significantly affected. The results indicate that, compared to RC frames conventionally designed using IBC-2009, optimum solutions significantly reduced global damage by up to 30 % and achieved a near uniform inter-storey drift distribution. Similar conclusions were drawn by Bai et al. [32], where the structural damage of RC moment-resisting frames was minimised by redistributing the area of flexure reinforcement from components with less damage to 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 a study by Bai et al. [33] to redistribute material with the ultimate aim of reducing damage. It was shown that optimum design solutions could reduce maximum inter-storey drift ratios by up to 35 %. The proposed optimisation also achieved more uniform deformation distributions under different seismic hazard levels, with a marginal increase (5 %–10 %) in material costs. Similarly, Hashmi et al. [34] presented a PBSO optimisation that aimed to achieve more efficient use of structural members and more uniform distribution of inter-storey drifts in both regular and irregular frames at serviceability limit state. In the performance-based optimum design of irregular RC frames proposed by Hashmi et al. [35], the damage was controlled 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.

2.1.4. Modified fundamental period

Reducing the fundamental (first-mode) period of an elastic structural (named as eigenfrequency optimisation) has been used as an indirect approach to minimise structural damage [36,37]. The objective function $F(x)$ is expressed as:

$$\text{Max}F(x) = \omega_n(x) \quad (3)$$

where ω_n represents the n^{th} eigenfrequency of the selected regular or irregular RC frame, and design variable x is considered as sectional dimensions of beam and column elements, while maintaining the same concrete volume before and after the optimisation. The physical justification of the eigenfrequency optimisation [36,37] is that for the same amount the concrete as the initial design, the elastic performance of the building can be improved by maximising structural fundamental frequency through redistribution of structural material (i.e. structural dimensions) leading to a delay in the initiation of structural inelastic behaviour. The results of these studies demonstrated that, in general, optimised buildings exhibited greater overstrength and ductility capacity, as well as less inter-storey drift and susceptibility to collapse compared to the initial design.

2.1.5. Minimum cost optimisation

2.1.5.1. Minimum material usage optimisation. Most design optimisation studies try to save computational cost by minimising the initial construction cost only in terms of total materials used in the construction of the structure (concrete volumes and/or reinforcement weights). The objective function $F(x)$ in this case can be written as:

$$F(x) = V_c \bullet C_{co} + m_s \bullet C_{so} = \sum_i^{N_i} b_i h_i L_i C_{co} + \sum_i^{N_i} A_{s,i} L_i C_{so} \quad (4)$$

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

Ganzerli et al. [38] proposed an optimum seismic design incorporating PBSO criteria that limited plastic rotations at beam and column elements. A simple 2D RC frame was analysed to minimise structural cost, which was assumed to be proportional to the total amount of concrete and reinforcement. In more recent optimisation studies of multi-storey RC structures [39–44], the design objective of “minimum structural material cost” was achieved by minimising the total concrete volume and reinforcing steel weight. Both section dimensions and steel reinforcement amounts were considered as design variables. Additionally, to ensure structural safety under selected seismic hazard levels, the objective function was subjected to a set of design constraints to limit structural seismic responses (e.g. inter-storey drift). Consequently, the optimum solutions provided less total direct cost and achieved an improved control on maximum seismic response values.

To minimise total weight of longitudinal reinforcement in RC frames, while controlling inter-storey drift in each storey for the selected performance level, the optimisation methodology incorporating with PBSO criteria was developed in studies by Hajirasouliha et al. [31] and Hashmi et al. [35]. 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 reduced the amount of reinforcement steel by up to 33 % and simultaneously satisfied multiple performance objectives in terms of Life Safety (LS) and Collapse Prevention (CP). In another study, Seify Asghshahr [45] developed a reliability-based optimisation framework to achieve a minimum initial material cost, while the reliability index of RC frames was minimised by subjecting probabilistic constraints in the objective functions. To minimise initial construction cost of a RC frame, Zhang and Tian [46] developed a simplified approach by reducing the number of design variables into overall system stiffness and overall system strength to reduce computational cost. 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).

2.1.5.2. Reducing construction costs using more parameters. Besides total concrete volume and reinforcement weight, some studies have also considered the amount of formwork used during the construction and its associated costs [47–54]. The general objective function $F(x)$ utilised in these studies can be expressed as:

$$F(x) = V_c \bullet C_{co} + m_s \bullet C_{so} + A_f \bullet C_{fo} \quad (5)$$

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. This indicated that the initial construction cost of optimum design solution depends heavily on the unit prices of concrete, steel and formwork [52].

The review of the existing literature indicates that almost half of the previous studies simplified the optimisation process by changing the objective from “minimum economic cost” to “minimum structural material use”. However, using structural weight to represent initial cost is questionable as minimising total material use in RC buildings does not necessarily lead to the minimum cost, particularly if labour and fabrication costs are considerable. In a relevant study, Li et al. [55] investigated the initial cost of RC frame-shear-wall structures by minimising costs of materials (i.e. concrete and steel), fabrication, labour and

formwork. To achieve a practical optimum design that could satisfy 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 separate databases were constructed for beam and column sections. The results indicate that the proposed optimisation minimised the total cost and provided a practical design solution that could be directly adopted by engineers. A similar optimisation procedure was proposed by Esfandiari et al. [56], who established a “minimum cost” objective function considering costs of materials, labour and placement of concrete and reinforcement, while satisfying design code requirements, as well as constructional, architectural and reinforcement detailing constraints.

2.1.5.3. Indirectly minimising economic cost optimisation. An alternative way to minimise structural initial construction cost of 3D irregular RC structures was proposed by Lavan and Wilkinson [57]. In this study, the objective of the design optimisation was to minimise total flexural moment capacity of all beam and column members, while satisfying constraints assigned on inter-storey drift and ductility. The assumption was that, if element dimensions remain constant, the total volume of steel (which is directly related to element flexural strength) would be the main component affecting costs.

2.1.5.4. Minimising total life-cycle cost optimisation. In an optimum performance-based design framework proposed by Lagaros and Fragiadakis [58], a single objective was considered to minimise total life-cycle costs of 3D regular and irregular RC structures. Costs were expressed as the sum of the initial costs and the expected limit-state costs over the life span of the structure. Similarly, Razavi and Gholizadeh [59] proposed a single-objective optimisation of RC frames. However, two different objectives, including minimum initial costs and minimum total life-cycle costs, were considered independently in their optimisation study for comparison purposes. The results of their study indicate that the optimisation considering the total cost provided a more efficient design solution in terms of economy and seismic collapse safety, compared to the case that only the initial cost was minimised.

2.2. 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:

$$\text{Min}[F_1(x), F_2(x), \dots, F_N(x)] \quad (6)$$

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

$$x \in \{x_1, x_2, \dots, x_j\} \quad j = 1, 2, \dots, N_j$$

where $F_1(x)$, $F_2(x)$, ..., F_N represent multiple design objectives relating to optimisation design problems, N is the total number of the objectives in the optimisation, 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.

In a multi-objective optimisation, generally there is not a 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 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 balances conflicting objectives while satisfies practical design constraints.

2.2.1. 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. While

reducing the total amount of materials can minimise the initial construction cost, “blindly” reducing materials may compromise the capacity of certain structural elements, thereby increasing structural responses (e.g. floor accelerations, inter-storey drifts). To balance these conflicting objectives, Lagaros and Papadrakakis [60] expressed a multi-objective function in the optimisation framework for a 3D RC frame as:

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

where C_{IN} is the initial construction cost (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. The acceptable solutions of the optimisation problem in Lagaros and Papadrakakis [60] study were presented on a Pareto front curve, indicating the locus of all optimum designs across different values of the specific 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 PBSO procedures.

Gharehbaghi [61] 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. A modified Park-Ang damage index was adopted as the performance parameter to quantify damage at both element and structural levels. It was found that code-based design solutions required more construction costs (up to 4 %) and experienced more damage (30 % on average) under severe earthquakes. Similarly, Asadi and Hajirasouliha [21] introduced a practical performance-based optimisation methodology based on the concept of Uniform Damage Distribution (UDD) for RC frames. The methodology aimed to minimise both structural and non-structural damage in terms of inter-storey drift, and total life-cycle cost. The results from incremental dynamic analysis (IDA) confirmed that, compared to frames that were initially designed in accordance with ASCE 07–16 and ACI 318–14, the optimum design solutions reduced total life-cycle cost (up to 45 %) and inter-storey drift ratios (up to 50 %) at Life Safety performance level. Asadi and Hajirasouliha [21] also highlighted that optimisations aiming to minimise initial cost do not necessarily lead to the optimum solutions when the life-cycle cost is considered in the objective function. In other studies, Möller et al. [62, 63] proposed optimisation frameworks that aimed to minimise both life-cycle cost and structural failure probability of RC frames under earthquake excitations. The failure probabilities of the optimum solutions were limited by applying reliability constraints at each selected performance level. In addition to initial construction costs and damage repair costs involved in the life-cycle cost, Möller et al. [63] also introduced social cost in the cost objective function, which was associated with costs to human and economic losses after earthquakes.

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

During seismic design optimisation, the term “economic cost” can extend to a broader definition that consists of: (i) initial costs including material and fabrication costs during construction, and (ii) 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 the sum of initial construction cost and expected damage cost:

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

where C_{IN} is the initial cost; 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:

$$C_{LS} = C_{idam} + C_{icon} + C_{iren} + C_{iinc} + C_{iinj} + C_{ifat} \quad (9)$$

where the following costs are included: damage repair (C_{idam}), contents cost (C_{icon}) due to structural damages (generally quantified in terms of maximum inter-story drifts and floor accelerations), loss of rental (C_{iren}), loss of income (C_{iinc}), cost of injuries (C_{iinj}) and cost of human fatalities (C_{ifat}).

It should be noted that in the calculations of life-cycle cost during an optimisation process, the “initial construction cost (C_{IN})” and “expected damage cost (C_{LS})” conflict with each other. Considering that C_{LS} is calculated based on both structural and non-structural damage, a reduction in structural damage can result in lower overall expected damage costs. However, this reduction is generally achieved by using additional materials, which in turn will increase the initial construction costs (C_{IN}). To address this drawback, Zou et al. [64] developed a multi-objective function for seismic design optimisation of RC frames as follows:

$$\text{Min}[F_1(x) = C_{IN}(x), \quad F_2(x) = C_{LS}(t, x)] \quad (10)$$

where all variables are as defined before. The above-mentioned objectives can be used to minimise the total life-cycle cost ($C_{TOT}(t, x_j)$). In Zou et al. [64] study, section dimensions of RC members and reinforcement quantities were considered as design variables to minimise concrete costs and steel reinforcement costs, respectively. The proposed multi-objective optimisation function was solved by first transferring it into a single-objective function through the ϵ -constraint method. Subsequently, a Pareto optimal set that contained a set of non-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 other studies [65, 66], where annual probabilities of non-performance (failure) were also limited by subjecting reliability-based constraints to the objective function. Using a multi-objective performance-based seismic optimisation approach, Mitropoulou et al. [67] simultaneously minimised $C_{IN}(x_j)$ and $C_{TOT}(t, x_j)$ in 3D regular and irregular RC buildings. Compared to a single-objective design optimisation, where $C_{IN}(x_j)$ was minimised, the solutions obtained from the multi-objective optimisation problem required more structural material but led to lower life-cycle cost. This reduced structural vulnerabilities to earthquakes, especially when the initial costs were the dominant factor in choosing optimum solutions. The results showed that neglecting the effects of uncertainties in material properties and section dimensions can significantly underestimate (by up to 30 %) seismic damage indices and total life-cycle costs. It was also concluded that a sufficient number of earthquake records is required to obtain reliable life-cycle cost results.

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

Optimisation objectives can also consider minimising environmental impacts caused by material and energy consumption, greenhouse gas emissions and CO₂ emissions, during construction and/or operation periods. An optimisation approach for design of RC frames was proposed by Nouri et al. [68], focusing on both cost savings and environmental impact for the entire structural life-cycle period. Optimum designs were achieved by considering three objective functions: (i) minimising the sum of initial construction costs (C_{IN}) and expected damage costs (C_{LS}), (ii) minimising the total life-cycle costs (C_{TOT}) and overall environmental score ($Score_{TOT}$) (quantifying the environmental impact), and (iii) minimising the sum of life-cycle costs (C_{TOT}) and environmental scores at initial construction and operational stages. The results showed that, compared to code-based designs: (i) optimum designs obtained considering the first objective reduced total life-cycle costs by up to 9 %, but with a slight increase in initial costs; (ii) if “environmental impact” was considered in the objective function, the obtained optimum design

led to higher life-cycle costs (up to 5.5 %) but lower environmental scores (up to 22 %). Optimisation studies on RC frames developed by Mergos [69] also confirmed that design for seismic loads could considerably increase the CO₂ emissions of RC frames. The study also concluded that optimum design aiming to minimise environmental impacts would generally depend considerably on the pre-determined ductility class of buildings located in high-seismic regions and the specified seismic hazard levels.

Fig. 3 shows the percentage of studies for different optimisations grouped in quinquennials. Whilst this topic has attracted increasing research attention, the review shows that much less research exist on multi-objective seismic design optimisations of RC frames. For better comparison, Fig. 4 shows the percentage of the relevant optimisation studies reviewed in this article for each optimisation objective. A comprehensive overview of the reviewed design optimisations of RC frames, including target structures, applied optimisation methodologies, implemented seismic analysis methods, and optimised design variables are provided in the Appendix A, where tables are categorised based on number and details of design objectives.

In this section, different seismic design optimisation problems of RC frames were discussed by focusing on their design objectives. Overall, the findings in this section suggest that single-objective optimisation studies, particularly emphasising on minimum economic cost, generally do not take into account some detailed objective factors, such as costs associated to fabrication, labour, transport and storage in different urban environments. In practice, multiple factors should be considered for a design, such as safety, cost-effectiveness, and sustainability. This implies that multi-objective optimisation studies are align more closely with realistic engineering requirements and should attract more attentions. However, previous multi-objective optimisation processes generally resulted in more than one optimum solutions, and therefore, it is necessary to develop tools to help engineers choose a solution that balances multiple conflicting objectives.

3. Optimisation formulations: design variables and constraints

This section provides more details about the fundamental components involved in the formulations of single objective and multiple objectives optimisation problems.

3.1. Design variables

Design variables are parameters that are modified in the optimisation process to achieve an optimisation objective. In accordance with the number of selected parameters modified in the structural size optimisation, the design variables can be divided into two categories: *single* and *multiple* design parameters. The values of the variables are generally classified as *discrete* or *continuous*. For instance, in previous research studies, the cross-section dimensions and amount of flexural

reinforcement of RC elements were both considered as multiple design parameters and modified as variables, since their values are directly related to the total weight of the structures and their optimisation can reduce structural damage during seismic events.

3.1.1. Single design parameter

Longitudinal steel reinforcement ratio has been widely used as a sole design parameter in many of previous optimisation studies, since it is a key parameter in controlling inelastic structural responses under earthquake excitations. Bai et al. [32] highlighted that the longitudinal reinforcement of elements in a specific story i can be also influenced by the reinforcement designs at immediate top and bottom storeys:

$$[(A_{scol})_{ik}^j] = \omega_1^1 * [(A_{scol,1})_{ik}^j] + \omega_1^2 * [(A_{scol,2})_{ik}^j] + \omega_1^3 * [(A_{scol,3})_{ik}^j] \quad (11)$$

$$[(A_{sbeam})_{ik}^j] = (1 - \beta) * [(A_{sbeam,1})_{ik}^j] + \beta * [(A_{sbeam,2})_{ik}^j] \quad (12)$$

where ω_1^1 , ω_1^2 , ω_1^3 are reinforcements contribution factors; subscripts “1”, “2” and “3” denote longitudinal reinforcement of column in an immediate lower storey, current storey and immediate upper storey, respectively; β is the reinforcement contribution factor for beam elements (normally assumed as constant $\beta = 0.5$); $[(A_{sbeam})_{ik}^j]$ represents the modified steel reinforcement area for the j^{th} beam element in the i^{th} storey and at the k^{th} iteration; $A_{sbeam,1}$ and $A_{sbeam,2}$ are the longitudinal reinforcement of the beams in the i^{th} storey and the immediate upper storey, respectively.

When only the amount of longitudinal steel reinforcement is considered as the design variable, the sectional dimension will be another important parameter affecting the structural performance of RC buildings. The dimensions of the elements are normally determined at the initial stage of the design in accordance with gravity loads and serviceability requirements in design codes [31,33,57]. Previous optimisation studies typically maintained constant section dimensions throughout the entire design optimisation procedure, and these were only enlarged if the modified reinforcement exceeded the limiting values specified in the selected design guidelines or if practical limitations required so. However, these design variables are not independent, as structural capacity and ductility under seismic excitations are affected by both section size and amount of reinforcement. In such optimisation approaches, it is also assumed that sufficient transverse reinforcement exists 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 [36,37,55], sectional dimensions of column and beam elements are selected as the only design parameter, modified independently to find their best values in all structural elements. This single design parameter is generally utilised in optimisation problems dealing with the elastic behaviours under minor earthquakes or for stiffness optimisation. This is because the size of concrete section controls the lateral stiffness and deformations of buildings within the elastic range. Using this approach, detailing of the steel reinforcement is determined following conventional design procedures and only after the section dimensions were optimised.

3.1.2. Multiple design parameter

For “minimum structural cost”, the objective function generally consists of multiple design parameters that account 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 parameters are needed, as concrete and steel influence stiffness and strength of structural members, respectively.

In structural optimisation, one of the ways to modify multiple design parameters as key variables is to use databases that contain pre-determined beam and column elements with various cross-sectional sizes and amounts of longitudinal and transverse reinforcement. The boundaries of each database are chosen by applying design constraints

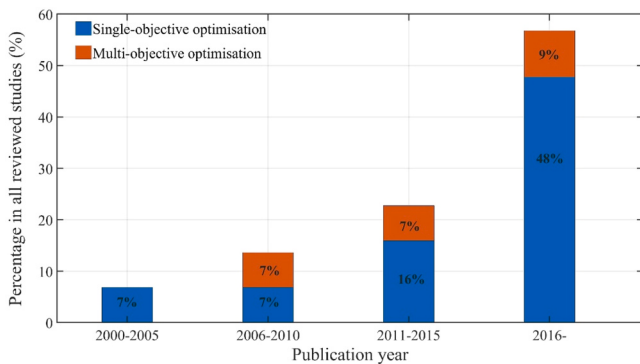


Fig. 3. Percentage of objective function types in the past studies reviewed in this study (Total reviewed studies: 42).

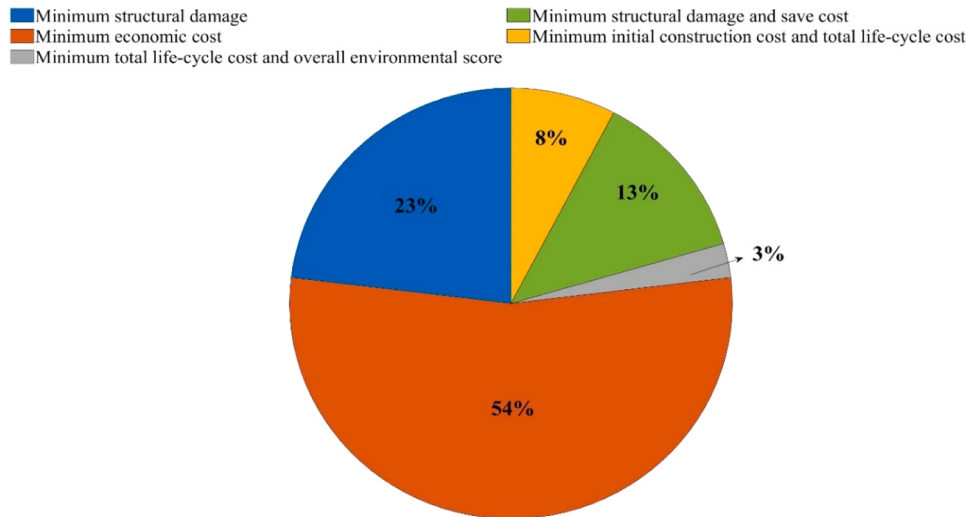


Fig. 4. Proportion of different design objectives in all reviewed studies (Total reviewed studies: 42).

to the selected objective function. A search-based optimisation method can be then used to find optimum answers of these design variables from the pre-determined values in the database. This indicates that the proper selection of the search domain (database) affects the accuracy of optimum design solutions. Mergos [50,51] divided the vector of design variables (i.e. 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. In another study, Mergos [48] categorised the selected design variables into primary and secondary variables. Cross-section dimensions and longitudinal reinforcement amounts were considered as primary design parameters and were selected from a pre-determined search space. The transverse reinforcement, as a secondary parameter, was chosen to fulfil design requirements with respect to performance, serviceability and construction practices after the optimiser obtained the primary variables in each iteration.

Some optimisations studies [42–44,64] dealing with multiple design parameters suggested to divide the entire optimisation process into: (i) an elastic design optimisation and (ii) a plastic design optimisation. In most of these studies, the cross section sizes of elements were considered as the only design parameter in the elastic phase to ensure structural serviceability or Immediate Occupancy (IO) under minor earthquakes. Once the optimum section dimensions were chosen at end of the elastic optimisation, they were kept unchanged during the plastic phase, where cross-section areas or arrangement of longitudinal reinforcements were optimised as primary designs variable under rare earthquakes.

3.2. Design constraints

A set of design constraints or “checks” can be used to ensure that each candidate design meets design code requirements and practical limitations. Design code requirements may include deformation demands, structural geometry and detailing. Structural behaviour constraints on strength, ductility and displacement can also be used to ensure structural safety under seismic loads. For example, Akin and Saka [52,54] adopted design constraints in terms of shear and flexural strength, ductility, serviceability and seismic performances requirements according to design provisions in ACI 318–05.

In addition, the integration of PBSO in design optimisation will require certain design constraints using performance-based target limits. These limits quantitatively describe the desired structural safety at selected performance levels. By employing this approach, structural

performance is more directly and effectively controlled under different earthquake intensity levels, ensuring that structures maintain specific performance objectives.

In general, design constraints used in structural optimisation under seismic loads can be categorised into deterministic and reliability-based constraints.

3.2.1. Deterministic design constraints

A typical deterministic design constraint can be expressed as:

$$g_i(x) \geq 0 \quad i = 1, 2, \dots, N_i \quad (13)$$

where $g_i(x)$ is the constraint relating to a design variable x in a full vector of design variables, and N_i is the total number of design constraints that should be satisfied in the optimisation process. Besides inequality constraints, equality constraints can also be adopted:

$$g_i(x) = 0 \quad i = 1, 2, \dots, N_i \quad (14)$$

Previous optimisations generally used inter-storey drift ratio to describe structural damage, while the response was limited using deterministic constraints [31,42–44,47,61]. 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 [70]. Conversely, some other studies [46,47] used plastic hinge rotation at each structural element as performance constraints, where a target limiting value at each specific performance level was decided following load information and section properties at each optimisation iteration, as recommended in ASCE/SEI 41–13 [71].

3.2.2. Reliability-based design constraints

In current seismic design codes, seismic uncertainty is commonly addressed by applying a series of coefficients when deciding seismic design loads. These coefficients account for site soil conditions, loading characteristics, the importance of structure, and seismic nonlinear behaviour. The values of these coefficients are generally chosen using expert judgement and empirical evidence, but they are not always realistic and rational [72]. On the other hand, uncertainties in other parameters such as structural properties, material properties and numerical modelling can also have significant impact on seismic responses. Therefore, a deterministic-based optimisation approach that ignores the effects of sources of uncertainties does not necessarily produce reliable evaluation of seismic responses, and hence may lead to unsafe design solutions.

To achieve more reliable optimum solutions, the effects of diverse sources of uncertainties have been considered in few previous design

optimisation studies. Möller et al. [62,63] developed seismic design optimisation frameworks for RC frames, by taking into account uncertainties from structural capacities and seismic demands. It was shown that the proposed optimisation frameworks considering the effects of the uncertainties lead to more accurate seismic response predictions and avoid unexpected failure probabilities, compared to deterministic-based structural optimisation. To address this issue, in studies by Khatibinia et al. [65] and Yazdani et al. [66] considered uncertainties in material properties of concrete, steel, and soil, as well as in input earthquake characteristics and their effects on seismic responses, by employing probabilistic constraints in the performance-based design optimisation process.

In general, the reliability-based constraints can be expressed by different parameters including reliability index, annual failure probability, and mean annual frequencies (rate) of exceedance. For instance, Seify Ashghshahr [45] and Zou et al. [73] expressed the reliability constraint as a reliability index β_k . The design constraint referring to structural reliability index was then expressed as:

$$\beta_k \geq \bar{\beta}_k \quad (k = 1, 2, 3, \dots, N_k) \quad (15)$$

where N_k is the total number of performance objectives specified in the optimisation problem; $\bar{\beta}_k$ is a minimum target value on reliability index; and k represents the total number of the considered performance-based objectives.

Furthermore, the reliability constraint ($g_R^i(X_u)$) in the optimisation formula can be expressed as a probabilistic constraint using the following equation:

$$g_R^i(X_u) = \frac{P_{np}^i}{P_{np.all}^i} - 1 \leq 0 \quad (16)$$

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

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 explained in the following sections.

First-order reliability-based method.

The first-order second-moment was adopted as an efficient approach to evaluate the reliability of a seismic design solution in a previous optimisation study by Zou et al. [73]. The main target of this method is finding the most probable failure point that has minimum distance to limit-state surface within the space of variables. It is thus necessary to have an exact expression on limit-state function or limit-state surface. They calculated the reliability index (β_k) based on the seismic responses and the corresponding target limits at selected performance levels:

$$\beta_k = \frac{\mu_{\Delta\bar{u}}^N - \mu_{\Delta\bar{u}}^N}{\sqrt{(\sigma_d^N)^2 + (\sigma_{\Delta\bar{u}}^N)^2}} \quad (17)$$

where $\mu_{\Delta\bar{u}}^N$ and $\sigma_{\Delta\bar{u}}^N$ represent equivalent mean and equivalent standard deviation for an inter-storey drift ($\Delta\bar{u}$), respectively; and μ_d^N and σ_d^N are the equivalent mean and equivalent standard value for allowable drift limit (d) specified in PBSD criteria, respectively. The corresponding failure probability (P_f) can be in turn calculated as follows:

$$P_f = 1 - \Phi(\beta_k) \quad (18)$$

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

Monte-Carlo simulation method.

The Monte-Carlo simulation method has been 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 unsuitable [65,66]. This method can simultaneously consider limit state functions introduced in PBSD guidelines at different performance levels. Equations utilized in the Monte-Carlo method are as follows:

$$P_{np} = \frac{1}{N} \sum_{i=1}^N I_i(X_u) \quad (19)$$

$$I_i(X_u) = \begin{cases} 1 & \text{if } G^i(X_u) \leq 0 \\ 0 & \text{if } G^i(X_u) > 0 \end{cases} \quad (20)$$

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

$$G^i(X_u) = R_{limit}^i - R^i(x_u) \quad (21)$$

where $R^i(x_u)$ is the probabilistic structural seismic response (e.g. maximum inter-storey drift) and R_{limit}^i is the corresponding limiting value.

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, to save on computational time, previous study by Gholizadeh and Aligholizadeh [49] predicted relevant structural seismic performance mathematically using metamodels instead of nonlinear time history analysis.

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) [41]. The MAF ($\nu(\text{EDP} > \text{edp})$) has been 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). The MAF can be calculated using the following equation [41]:

$$\nu(\text{EDP} > \text{edp}) = \int_0^\infty [1 - P(\text{EDP} > \text{edp}(\text{IM} = \text{im}))] \left| \frac{dv}{d\text{IM}} \right| d\text{IM} \quad (22)$$

where $P(\text{EDP} > \text{edp}(\text{IM} = \text{im}))$ is the 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 ($\text{IM} = \text{im}$), and $\left| \frac{dv}{d\text{IM}} \right|$ is the mean annual rate of the earthquake intensity. The EDP has been generally expressed as inter-storey drift ratios or other performance parameters that represent local or global structural damage. The limit-state probability $P(\text{EDP} > \text{edp}(\text{IM} = \text{im}))$ is then calculated as:

$$P(\text{EDP} > \text{edp}(\text{IM} = \text{im})) = \Phi \left[\frac{\ln(\widehat{\text{edp}}) - \ln(\widehat{\vartheta_{\max}})}{\delta} \right] \quad (23)$$

where $\ln(\widehat{\vartheta_{\max}})$ and δ are the logarithmic mean and standard deviation of the response ($\vartheta_{\max}$); $\ln(\widehat{\text{edp}})$ is the logarithmic mean of the pre-determined target limit of the response. In the previous study by Fragiadakis and Papadrakakis [41], the probability distribution of the seismic response variable was assumed as logarithmic.

It should be noted that, to reduce computational time, reliability-based design optimisation methods generally include only the sources of uncertainty that broadly affect the structural performance of the design solutions into reliability constraints. Other uncertain parameters can be considered 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, the inherent uncertainties in other design parameters, such as the those describing the characteristics of plastic hinge rotation, the mass of the structure, and the geometry information (i.e. storey heights and member lengths) were ignored. However, this simplification can reduce the accuracy in calculation of structural failure probability, and hence affect design checks in accordance with reliability-based constraints. Furthermore, this review found that when the reliability indexes (β_k) or failure probabilities (P_f) were utilised to express reliability constraints, there were no design criteria for target limiting values for the reliability index. In previous studies, these limiting values were decided based on achieving a balance between economic considerations and acceptable risks levels. However, relying on user-defined limiting values within the reliability constraints does not consistently lead to rational and reliable design solution.

This section provided a detailed review on diverse modified design variables and different design constraints considered in previous optimisation studies. It was concluded that considering multiple design parameters as variables in optimum seismic design of RC frames could provide more realistic solutions, especially when both elastic and inelastic structural performances are controlled in the optimisation. However, this approach may result in a time-consuming process, highlighting the need for more computationally efficient optimisation methods. The importance of reliability-based design optimisation was also discussed to address the limitations of the deterministic-based design approaches.

4. Optimisation methodologies

In the field of structural optimisation under seismic loads, three categorises of optimisation methodologies are commonly adopted: (i) search-based optimisation; (ii) gradient-based optimisation; (iii) optimisation using “optimality criteria” (e.g. Uniform Damage Distribution (UDD)). Appendix A summarises the optimisation methodologies adopted in different studies, whereas Fig. 5 compares their relative share in the current literature. It is shown that the most popular method for seismic design optimisation of RC frames is the search-based optimisation, followed by those used the concept of UDD. The following subsections summarise the main characteristics of each methodology and discuss their strength and shortcomings.

4.1. Search-based optimisation methodology

Search-based optimisation algorithms (or metaheuristic optimisation algorithms) are generally inspired by natural phenomena such as the movement of individuals in a bird flock, or by natural selection process. They include: genetic algorithm (GA) [36,37,39,40,45,48,51,55], evolution strategies (ES) [41,58,60,67], chaotic enhanced colliding bodies optimisation (CECBO) [49], harmony search (HS) [52,54], particle swarm optimisation (PSO) [39,53,56], gravitational search algorithm [65,66], improve multi-verse (IMV), improved black hole (IBH) and

modified newton metaheuristic algorithm (MNMA) [47,59]. The above algorithms are widely used in structural optimisation and mainly aim to improve objective values through iterations.

As an example of search-based methodologies, Fig. 6 shows the optimisation procedure adopted in an ES algorithm, including details on generating new population and deciding a termination criterion. Generating new individuals in each generation is an essential component in 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 create 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 the position and velocity of each particle (individual) to search for its best position within the search space. In PSO, multiple candidate designs are generally generated at each iterative step. These designs are evaluated and compared, and the design with the 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 can handle both continuous and discrete design variables; (ii) several design parameters can be modified as variables in the optimisation approach; (iii) there is no need of gradient information or exact relationships for objective functions and design constraints, hence they can be easily implemented if obtaining the gradient of objective functions proves difficult; and (iv) the algorithms generally avoid local optimum answers, when the pre-determined search space is fully explored.

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 also highly dependent on the size and selection of the search space. Such optimisation methods are normally not suitable for complex structures under multiple load cases and when using time-consuming analysis methods (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. [74] pointed out that standard metaheuristic algorithms struggle to deal 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 [75], there is no unique metaheuristic optimisation approach that can provide best answers for all optimisation problems.

To reduce computational costs, Li et al. [55] used a hybrid GA and Optimal Criteria (OC) optimisation method that combines the

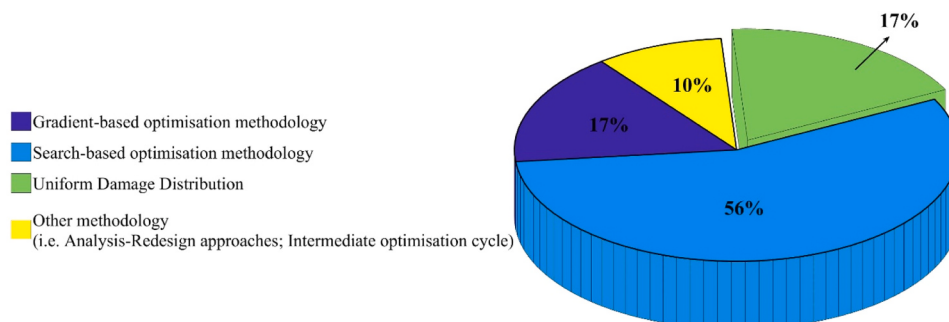


Fig. 5. Percentage of optimisation methodologies utilised in the previous studies (Total reviewed studies: 42).

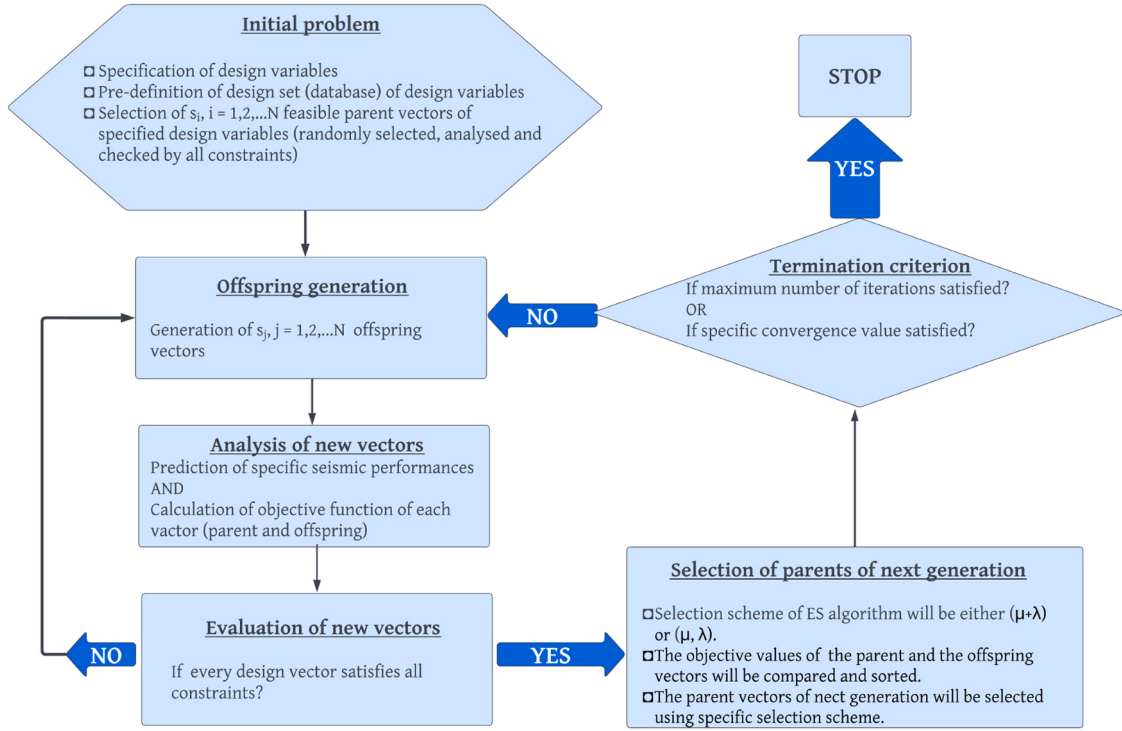


Fig. 6. Optimisation procedure with Evolution strategies (ES) algorithm.

advantages of both methods. This aimed to solve a practical design problem of RC frames with a large number of design variables. The strength and stiffness design optimisation were processed by the GA and OC algorithms, respectively, and consequently, an optimum design that met both strength and stiffness constraints was achieved. Esfandiari et al. [56] adopted a similar approach, where the hybrid multi-criterion decision-making (DM) and PSO were combined to accelerate the optimisation convergence and simplify the optimisation process. Razmara Shooli et al. [39] also implemented a hybrid GA and PSO optimisation technique to improve populations, which were initially generated by GA. The hybrid optimisation achieved an optimum solution for a complex design problem with less computational costs compared to the conventional methods. Razmara Shooli et al. [39] also proposed combining non-linear static analysis and non-linear dynamic analysis to obtain an optimum search space for optimum answers.

4.2. 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 classified: (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 the gradients of the objective functions and the gradients of specific constraints are required in these algorithms. To reduce computational costs and obtain gradient information for a complex optimisation formula, gradient-based optimisation algorithms generally convert a constrained design problem including objective functions with time-dependent performance constraints into approximate unconstrained functions, namely Lagrangian functions. The design variable x_i can be indirectly modified by finding stationary condition of the Lagrangian function, which is assumed as the time point when the first derivative of the function equals zero [44].

Some optimisation frameworks convert the objective function subjected to inequality constraints into an unconstrained one by using a

suitable transformation method such as Lagrange multiplier method [42,44] or exterior Penalty function [40,59]. For example, the Lagrange multiplier method utilised by Zou and Chan [42] transformed an objective function ($F(x)$) subjected to performance constraints $g_i(x)$ into an unconstrained Lagrangian function (L) expressed by the following formulation:

$$L(x, \lambda_i) = F(x) \pm \sum_{i=1}^{N_i} \lambda_i g_i(x) \quad (24)$$

where x represents any design variable considered in a structural system; N_i is total number of performance constraints (e.g. inter-storey drift constraints, plastic rotation constraints, etc) considered in design optimisation; and λ_i is the Lagrangian multiplier in accordance with i^{th} design constraint.

The stationary condition of the Lagrangian function is then expressed as:

$$\frac{\partial F(x)}{\partial x} \pm \sum_{i=1}^{N_i} \lambda_i \frac{\partial g_i(x)}{\partial x} = 0 \quad (25)$$

Considering the remodify design variables:

$$S_i = \frac{\sum_{i=1}^{N_i} \lambda_i \frac{\partial g_i(x)}{\partial x}}{\frac{\partial F(x)}{\partial x}} - 1 \quad (26)$$

$$x^{v+1} = x^v \times \left[1 + \frac{1}{\eta} S_i \right] \quad (27)$$

where $\frac{\partial F(x)}{\partial x}$ is the derivative of the objective function ($F(x)$) with respect to design variable (x); λ_j is a parameter used to convert constrained problems into unconstrained ones; $\frac{\partial g_i(x)}{\partial x}$ is the derivative of the i^{th} constraint ($g_i(x)$); N_i is the total number of performance-based constraints; η is a parameter that controls the convergence speed; v is the iterative step; and S_i is the search direction in the optimisation process.

The major advantage of gradient-based optimisation methods is that random searching within the identified search domain is avoided. As the optimisation approach ensures that any design variable violating design

constraints is not included in the optimisation, the search space of design variables can be reduced, and hence the search direction is more directed. In general, gradient-based algorithms lead to smooth convergence solutions since the convergence rate is commonly controlled by a parameter in the gradient calculation. However, gradient-based methodologies may lead to local optimum designs if the search direction is not well defined, while it is not easy to assess if a global optimum answer has been reached. Another limitation of gradient-based methodologies is that they are still computationally demanding due to the 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 dimensions and reinforcement ratios) are simultaneously optimised. Due to their high computational costs, previous gradient-based optimisation approaches avoided using time-consuming seismic analysis methods such as nonlinear dynamic analysis, which 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 the principle virtual work [44] or Newmark direct time integration method [40] 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 changes in natural frequencies. More details about such mathematical-based performance evaluation methods will be explained in the following section on seismic analysis methods.

4.3. 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) was proposed by Moghaddam and Hajirasouliha [25,26]. UDD optimisation utilises an adaptive iterative analysis-redesign process, in which design variables (e.g. section dimension, reinforcement area, damping coefficient in damper) are redistributed from slightly damaged components to heavily damaged components of a structure until a status of uniform damage distribution is achieved. UDD optimisation can also optimise lateral load patterns that are used to simulate the seismic effect in each storey in a building during seismic analysis [30,77,78]. An example of UDD formula used for the optimisation of RC frames [31] is:

$$[(A_{rein})_{i,k+1}^j] = \left(\frac{\Delta_i}{\Delta_{target}} \right)^\alpha * [(A_{rein})_{i,k}^j] \quad (28)$$

where $[(A_{rein})_{i,k+1}^j]$ is a specific design variable (here area of reinforcement in j^{th} element) of the i^{th} story at $(k+1)^{th}$ iteration; Δ_i is the seismic response result (here maximum lateral inter-storey drift) at the i^{th} storey level; Δ_{target} is the target value of the selected response parameters; α controls convergence speed in the optimisation process and ranges from 0 to 1. It is important to note that a large value of α may lead to divergences during the optimisation process. While there is no guarantee of convergence when UDD is applied to the non-linear structural systems that are highly sensitive to the modification of the design variable, by selecting a suitable value of convergence parameter (α) the convergence is generally achieved within a few steps without significant fluctuations. This optimisation method generally requires an additional sensitivity analysis to determine an appropriate value for the convergence parameter to strike a balance between the computational efforts and avoiding significant fluctuations. For RC building, previous studies have suggested using convergence parameter values around 0.2 when employing non-linear time history analysis [31,76,79].

In general, structural damage can be described by using performance parameters, such as a local Park & Ang damage index, storey ductility demands, maximum inter-storey drifts and plastic hinge rotations of structural elements. Using the UDD concept, structural damage can be

directly controlled and managed based on target limiting values at specific performance levels. This generally results in safer designs for RC frames, with less concentrated maximum seismic responses. Since the material capacities in most stories are fully exploited, UDD can potentially lead to more uniform damage distribution [31,35]. Besides, UDD optimisation can be implemented for practical design purposes to achieve different multiple objectives including minimum structural damage and minimum total life-cycle cost [21].

Compared to other optimisation methodologies, such as GA and PSO, UDD optimisation required up to 300 times less number of non-linear dynamic analysis [80]. Therefore, nonlinear time history analysis can be efficiently used when a UDD optimisation is adopted. However, most previous UDD-based optimisation studies only considered maximum inter-storey drifts as the single performance parameter to control damage, while only very limited research studies 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 all structural members. Furthermore, most of previous UDD-based optimisation studies only considered a single performance level (e.g. LS) under an earthquake with a specific recurrence rate (e.g. 475 year return period). However, this does not guarantee safety of the optimal structure in future earthquakes with higher intensities levels.

In this section, the optimisation methods utilised in previous structural sizing optimisation studies were categorised into search-based, gradient-based, and UDD. The strengths and shortcomings of each method were discussed. Overall, it was concluded that search-based methods can be used for most optimisation problems, without limitations on the selection of design variables or constraints. However, their computational efficiency notably decreases with an increasing the number of design variables, while their accuracy also depends on the pre-determined search space. Gradient-based methods can converge faster to the optimum solution by using a direct approach, but the required calculations of gradient information as per iteration are generally computationally expensive and can be mathematically complex. While the concept of UDD can be used to simply the complexity of the optimisation process and obtain a better design with enhanced convergence speed, its current development is still limited in terms of the choice of performance parameters and seismic hazard levels.

5. Seismic performance evaluation

When PBSO criteria are incorporated into the optimisation framework, analysis methods that can provide accurate and reliable predictions are essential. Current guidelines such as ASCE/SEI 41-17 [20] suggest four alternative procedures, as summarised in Table 1. The selection of a seismic analysis method depends on several factors, including target performance level, seismic hazard level, importance of the structure, and structural characteristics (e.g. regularity, complexity, frequencies and mode shapes) [81]. Fig. 7 shows that most optimisation studies to date have adopted nonlinear static (or push-over) and non-linear dynamic analyses, whereas only a few studies used linear analyses. The main characteristics and advantages and disadvantages of

Table 1
Alternative seismic analysis procedures suggested in ASCE/SEI 41.

Category	Analysis procedure	Analysis method	Seismic load
Linear	Linear static	Equivalent static analysis	Distributed static lateral load
	Linear dynamic	Response spectrum analysis/Linear dynamic 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

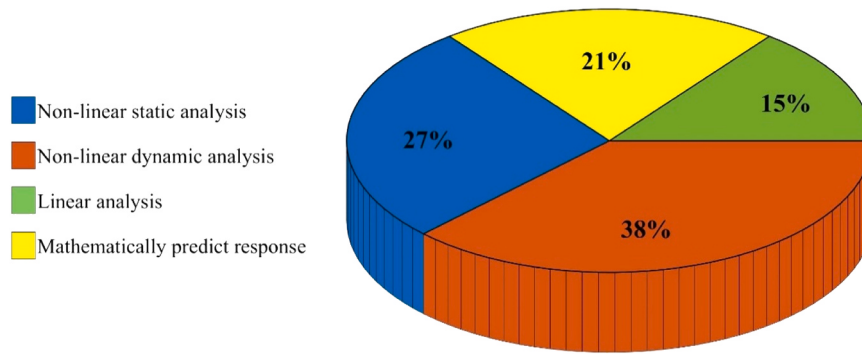


Fig. 7. Percentage of seismic analysis methods utilised in the previous studies (Total reviewed studies: 42).

the seismic analysis methods in Fig. 7 are discussed in the following sub-sections.

5.1. Linear static or linear dynamic analyses

In linear static or dynamic analyses methods, the equivalent static lateral force used to simulate seismic effects on a building is derived based on the expected structural behaviour of a linear elastic system. Non-linear ductile behaviours and energy dissipation capacity are accounted for indirectly by considering a response modification factor R in ASCE/SEI 7–10 [82] or a behaviour factor q in Eurocode 8 [9]. However, simple linear analyses cannot accurately assess the seismic performance of non-linear buildings, especially under strong earthquake events [25]. On the other hand, the distributions of equivalent static loads over the height of buildings become unrealistic when the lateral inertia forces redistribute after the occurrence of yielding. Only a few articles [34,45,83] adopted linear analysis methods to assess the structural seismic performance in elastic phase subjected to minor earthquakes.

5.2. Non-linear static analyses

Nonlinear static analyses (or pushover analyses) apply a monotonically increasing lateral load at each storey to push a structure until a collapse mechanism or target displacement at a control point is reached. Force or displacement can be used to control the increase of the lateral load. Previous optimisation frameworks that used pushover analysis adopted either the Displacement Coefficient Method [39,59] or Capacity Spectrum Method [32,42,44,46,50,64,67] to evaluate the expected target displacements and maximum seismic responses.

5.2.1. Displacement coefficient method

ASCE/SEI 41–17 [20] provides a displacement coefficient method to estimate the target displacement of a building (i.e. maximum displacement of the roof δ_t) using the following formula:

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \quad (29)$$

where C_0 is used to scale up the elastic displacement of a single degree of freedom (SDOF) system to the roof displacement of a multi-storey building; C_1 is a modification factor reflecting the ratio of the expected maximum inelastic displacement to the calculated structural elastic response; C_2 reflects influences of pinched hysteresis shape, strength deterioration and stiffness degradation on the maximum displacement response. S_a is the acceleration design response spectrum corresponding to the effective fundamental period T_e , where T_e is evaluated by modifying the fundamental period of the building (T) in the direction under consideration:

$$T_e = T \sqrt{\frac{k_i}{k_e}} \quad (30)$$

In the above equation, k_i and k_e are the building initial stiffness and effective lateral stiffness, respectively, evaluated by using an idealised force-displacement curve as suggested by ASCE/SEI 41–17 [20].

5.2.2. Capacity spectrum method

The capacity spectrum method assumes that the first vibration mode dominates the seismic response of a building. The method requires to first convert a pushover curve (in force-displacement format) into an acceleration-displacement response spectra (ADRS) format which is called capacity spectrum [81]. A demand curve is then obtained by scaling elastic response spectra of seismic actions to a demand spectrum. The intersection of the capacity and demand spectrum is defined as the “performance point”. At the performance point, seismic responses at both local and global levels are checked against target limits to ensure structural safety. As described in ATC 40 [17], the target displacement can be evaluated based on the deformation demand at the performance point.

Lagaros and Fragiadakis [58] compared the efficiency of different pushover analysis methods (displacement coefficient method in ASCE-41, capacity spectrum method in ATC-40, and the N2 method in Eurocode 8) for 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 construction costs (up to 3.7 %) and total life-cycle costs (up to 9 %). Conversely, the N2 and ASCE 41-methods provided relatively similar response results especially under low to medium earthquake intensity levels.

Pushover analyses were mainly adopted in previous optimisation problems when computationally demanding optimisation techniques (e.g. GA or PSO) were used, with the aim to reduce the high computational costs. 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 performance-based constraints can be easily calculated. This can be also achieved by utilising pushover analysis.

Most previous pushover analyses utilised invariant lateral load patterns (e.g. triangular and uniform shapes that are approximately proportional to the structural fundamental mode shape or floor mass). However, these load patterns may not be consistent with actual conditions as the structural inertia force is redistributed after some yielding in the structure. Likewise, using invariant load patterns may neglect the effects of higher modes, leading to less (underestimated) seismic responses at the upper stories of high-rise buildings.

A study by Mergos [48] discussed that the optimisation process incorporating conventional pushover analysis with either uniform or first mode-based lateral load patterns may not lead to optimal designs

that satisfy all performance requirements, especially at the local level, when their seismic performance is checked through non-linear dynamic analysis. Similarly, a study by Moghaddam and Hajirasouliha [84] highlighted that the limitations in pushover analysis can result in unreliable optimal solutions due to inaccurate evaluations of structural performance under strong seismic loads. Another limitation of conventional non-linear static analysis is that it can only be applied in the optimal seismic design of regular buildings [67]. To address this issue, previous optimisation framework proposed by Bai et al. [32] utilized an improved pushover analysis, called consecutive modal pushover analysis, which accounts for higher-mode effects.

5.3. Non-linear dynamic analysis

Nonlinear dynamic analyses can calculate seismic responses by directly solving the equations of motion under earthquake ground excitations [85]. To take into account the variability in the selection of earthquake records, due to uncertainties related to frequency contents and amplitudes of future earthquakes, and the variability in site classification, a group of earthquakes records can be used in the optimisation process to capture record-to-record variability [86]. In addition to adopting a group of ground motions, to manage earthquake record variability, the selected earthquake records are generally scaled so their mean spectrum compares well with the code response spectra at a specific seismic hazard level. Artificial earthquake records (mathematically derived from design response spectrum) have been also utilized for time history analysis in previous optimisation problems [30,80].

Non-linear dynamic analyses are deemed as the most accurate tool to predict the seismic performance of buildings since stiffness/strength degradation and hysteretic behaviour are explicitly considered [23]. Nonlinear dynamic analyses are also appropriate to assess the seismic response of irregular RC buildings as the effects of higher modes are directly included [57,67]. However, nonlinear dynamic analyses increase considerably the analytical complexity and computational costs, especially in optimisation methods where a large number of analyses are required.

5.4. Evaluate seismic response using mathematical equations

Some previous studies have adopted the displacement matrix method [52,54] to assess the seismic response of buildings. However, this method is based on the structural elastic behaviour, and therefore it cannot accurately predict inelastic responses. When dealing with nonlinear responses in the design optimisation, Gholizadeh and Aligholizadeh [49] employed a metamodel involving neural network (NN), based on the concept of machine learning, to predict seismic responses. 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 and section properties). Complex wavelet functions were then established to represent the relationship between input and output parameters. However, a large database of the design variables was required to ensure acceptable accuracy of the prediction results. A metamodel constructed based on the concept of artificial neural network was also employed by other studies [65,66] to predict average inelastic responses of a building and save computational costs. These proposed metamodeling frameworks consisted of a wavelet weighted least squares support vector machine (WWLS-SVM), and the responses were predicted by minimising the objective function through a gravitational search algorithm. However, in general, the prediction accuracy of these metamodels was shown to be significantly affected by the selection of parameters involved in the models (i.e. design variables, parameters of the utilised wavelet kernel theory). Furthermore, the computational efforts required 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.

As shown in Table 1, this section covered the most used analysis methods in structural seismic design optimisation problems. It was discussed that, in general, non-linear analysis methods (i.e. non-linear static and dynamic analyses) provide more accurate estimations of the structural performance compared to the linear methods, particularly under high-intensity earthquakes where the structures are expected to exhibit nonlinear behaviour. Non-linear static analyses are preferable when the computational efficiency is prioritised. However, the fixed load patterns utilised in the conventional non-linear static analyses may lead to inaccurate structural performance predictions. On the other hand, non-linear dynamic analyses can capture more realistic structural behaviour, but their limitation is the high computational cost that may limit their practical application.

6. Summary and conclusions

The comprehensive review conducted in this article confirms that although various methods have been proposed for the structural size optimisation of buildings, they generally involve the following main steps: (i) defining optimisation objectives, (ii) selecting design variables and constraints relevant to the objective formulation, (iii) applying optimisation methodologies, and (iv) analysing seismic responses. This article discussed different objective functions employed to address design problems, primarily focusing on minimising structural damage and economic costs, using single-objective and multi-objective optimisation. The most common design variables considered in the structural size optimisation, as well as deterministic and reliability-based design constraints in the optimisation formulations were reviewed. Subsequently, different optimisation methodologies and their advantages and limitations were discussed in depth. In general, the outcomes of the previous studies indicated that:

- Since the structural behaviour of RC frames in low-intensity seismic areas is expected to mainly remain in the elastic range, linear analyses methods such as linear static and linear time-history analysis, are viable alternatives to reduce the computational time of the optimisation process. However, in the case of irregular buildings linear time history analysis would be preferable as the effects of higher modes are more prominent. For irregular buildings in plan, it is also recommended to utilise three-dimensional models to take into account the torsional effects due to irregularity [87]. For design cases under minor earthquakes, most optimisation methods can be practically applied, since the number of design variables in the optimisation process is generally limited, and the computational efforts are not high for elastic analyses.
- For buildings located in medium to high-seismic regions, where inelastic structural behaviour is expected, linear seismic analysis methods cannot provide reliable predictions on structural performance. Therefore, nonlinear analyses including nonlinear static (pushover) and dynamic analyses are recommended. It should be noted that for irregular building and high-rise buildings where the effects of higher modes on structural performance are generally more dominant, previous studies have demonstrated that conventional pushover analyses with invariant load pattern may lead to inaccurate response predictions [84]. Additionally, considering the complexity and high computational costs of non-linear dynamic analysis, any optimisation methods that require significant number of iterations (e.g. Genetic Algorithm with large search spaces) or complex formulation between structural seismic responses and design variables (e.g. gradient-based optimisation) may not be practical. For such buildings, the concept of UDD optimisation can be efficiently used to simplify the complex optimisation process, and to obtain reliable optimum designs with a few iterative steps using the results of non-linear dynamic analyses. However, UDD optimisation methods, in general, have limitations in terms of modifying several variables simultaneously.

7. Research gaps and future directions

Based on the critical review conducted in this study, the following research gaps are identified aiming to achieve optimum design solutions for RC structures in seismic regions.

7.1. Formulation of the design optimisation problem

There is a lack of optimisation studies that take into account the detailed calculations of costs, such as those associated with labour, management, transport, storage, architectural design complexity, and site conditions. These cost parameters are generally influenced by the chosen construction method and the urban environment, and they can significantly affect the reliability of the optimal solutions. Additionally, the environmental impact of RC frames, such as CO2 emissions, should be incorporated when minimising construction costs. This is expected to lead to more sustainable design solutions for practical applications.

The seismic performance of RC buildings can be influenced by various interdependent parameters. While several design variables have been explored in previous studies, there remains a gap in research addressing elements and connections details that could impact structural seismic performance across different hazard levels. These include, laps and anchorage of longitudinal reinforcement, spacing of transverse reinforcement, volumetric ratio of transverse reinforcement, and quantity of steel reinforcement within critical regions. One primary challenge is that conventional optimisation methods would be computationally expensive when attempting to modify such a large number of variables, especially in the case of high-rise buildings.

Previous studies have highlighted that uncertainties related to material properties, modelling assumptions, and variations in ground motion records can considerably impact the accuracy of structural performance assessments [88,89]. Consequently, there is a need for comprehensive reliability-based optimisation studies to consider the effects of these uncertainties on the optimum design solution by incorporating performance-based constraints in a probabilistic manner.

7.2. Seismic performance assessment

Extensive optimisation studies aimed to improve the structural seismic performance of RC structures through different optimisation frameworks. However, these studies mainly focused on minimizing the damage to structural elements and did not directly control damage to non-structural elements that are affected by maximum floor accelerations. However, the non-structural damage can directly contribute to the economic losses and life-cycle costs after an earthquake event, consequently affecting the effectiveness of the applied optimisation method.

7.3. Application of optimisation methodology

While the UDD optimisation methods can considerably reduce the

computational costs of complex optimisation process of non-linear RC structures, it is still a challenging task to simultaneously modify several design variables. There is also no guarantee of convergence, especially in the case of non-linear systems under dynamic loads, where seismic performance can be sensitive to small variations in design variables. This highlights the need for the development of more efficient UDD optimisation methods that can address the aforementioned issues.

The metaheuristic optimisation algorithms adopted in previous studies generally perform well only for specific design problems, and they cannot ensure that the obtained solution is the "best" design when the same method is applied but a different objective function is formulated that influences the selection of the search space. Therefore, additional research is necessary to propose a metaheuristic optimisation method capable of offering optimal solutions for a wider range of optimisation problems in common practice.

7.4. Artificial Intelligence

While artificial intelligence (AI) has been recently integrated into the optimisation process of different structural systems to achieve more efficient, practical, and reliable optimal design solutions [90], it is currently overlooked in the literature for RC frame under earthquake excitations. AI algorithms can be incorporated into existing structural optimisation methods to iteratively adjust multiple sets of design variables and balance several conflicting design objectives. Additionally, AI techniques in data-driven approaches can be employed during seismic analysis, assisting in selecting the most suitable ground motion records for linear or nonlinear time history analysis.

CRediT authorship contribution statement

Iman Hajirasouliha: Writing – review & editing, Validation, Supervision, Methodology, Conceptualization, Investigation. **Kypros Pilakoutas:** Writing – review & editing, Validation, Supervision. **Reyes Garcia:** Writing – review & editing, Validation. **Geyu Dong:** Writing – original draft, Visualization, Methodology, Investigation, Data curation, Conceptualization.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data Availability

Data will be made available on request.

Appendix A

Single-objective design optimisation:

Table A1
Research developments in “minimum structural damage optimisation”.

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

(continued on next page)

Table A1 (continued)

Researcher	Year	Structure	Optimisation methodology	Seismic analysis	Design Variables
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 frames	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 A2

Research developments in “minimum economic cost optimisation”.

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
Frageadakis 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 back propagation)	sectional dimensions, arrangement of reinforcements
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 Frageadakis	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 design optimisation:

Table A3
Research developments in “multi-objective design optimisation”.

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, reinforcements 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

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