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Residual design life-based evaluation of structural retrofitting on high-rise reinforced concrete buildings

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Highlights:

- A reversed framework is proposed to evaluate the residual design life of retrofitted buildings
- Influences of obsolete and prevailing design codes on the building residual design life are revealed
- Effects of steel bracing, RC wall addition, and exoskeleton on the building residual life are elaborated

Abstract:

The deteriorating structural performance of aging high-rise reinforced concrete (RC) buildings has brought increasing attention to the necessity of retrofitting in developed regions. In this context, the present paper introduces a novel approach by reversing the conventional service life-based structural design procedure. The proposed approach establishes a residual design life evaluation framework, enabling a comprehensive assessment of the performances of existing RC buildings both before and after retrofitting. This assessment framework takes into account various load scenarios, including wind and earthquake forces, etc., and yields an intuitive index based on design codes, termed "residual design life", which serves as valuable information for decision-makers. To exemplify the framework's application, a case study involving a high-rise RC building is presented. Initially, the influence of the adopted design codes on the residual life of the aging building is investigated. Subsequently, three commonly employed structural-level retrofit methods, namely steel bracing, RC wall addition, and exoskeleton, are examined. The findings reveal that the residual design life of the existing building decreases as the adopted design code transitions from an outdated version to the current one. However, the implementation of structural-level retrofit interventions proves highly effective in enhancing the building's residual life, particularly when failure is primarily governed by column capacities and storey drift. Nonetheless, it should be noted that certain structural-level interventions may increase the load on adjacent beams, potentially leading to a negative impact on the building's residual life when the condition of the beams becomes the predominant factor. Furthermore, the obtained residual life of the retrofitted building could be used to assist in conducting life-cycle analysis and further investigations and contribute to the understanding regarding structural performance and safety in the built environment.

Keywords: Structural retrofit, residual life, high-rise building, RC structure

1. Introduction

In numerous developed regions, such as Hong Kong and Shenzhen in China, a substantial proportion of buildings are progressively aging and nearing their intended design end of life. For instance, according to the 2022 report by the Hong Kong government's development bureau [1, 2] over 9100 buildings are now over 50 years old, and this number is increasing at a rate of 600 per year. Due to the usage of outdated design codes and the natural degradation of materials, most of these aged structures may fail to meet current design requirements and are more susceptible to environmental hazards like wind, snow, and seismic risks, compared to newer constructions. Among these older buildings, high-rise residential reinforced concrete (RC) buildings are predominant in regions with high population density, offering greater occupancy capacity with lower space and construction costs. The compromised structural performance of these aging high-rise residential RC buildings poses a significant threat to the safety of residents and their property, potentially leading to irrevocable tragedies.

In response to the structural deficiencies and to enhance the safety of RC buildings, numerous structural retrofit strategies have been developed in recent years. Various technical notes, such as ATC-40 [3] and fib bulletin 24 [4], propose several retrofitting alternatives. These retrofit interventions can be broadly categorized into local and global levels. Local retrofit interventions aim to strengthen specific structural components with minimal disruption to the overall structural response, while global-level retrofitting involves modifications to large portions of the structure, effectively improving the overall building performance. Commonly employed local retrofit methods include FRP wrapping [5], steel jacketing [6], TRM wrapping [7] and RC jacketing [8] etc. These techniques aim to prevent building failure by increasing the compressive, flexural, and torsional capacities of existing structural members. On the other hand, global-level interventions involve strategies such as steel bracing [9], exterior buttress [10], and RC wall addition [11] etc., which enhance the entire structure by adding additional lateral resistance. However, the impact of global-level structural retrofit approaches on aging buildings is more complex and requires thorough investigation.

Selecting suitable retrofitting technologies necessitates the evaluation of the structural performance of existing buildings before and after retrofitting and the estimation of the effects of different retrofitting approaches. For the component level, the structural performance improvement can be assessed by comparing the member capacity before and after retrofitting [12]. On the other hand, for the structural level, the building seismic loss is often assessed using the performance-based earthquake engineering (PBEE) paradigm developed by Cornell and Krawinkler [13] and the Pacific Earthquake Engineering Research Center (PEER) [14]. Some studies[15, 16] also adopt an annual economic loss index caused by earthquakes to evaluate structural retrofitting performances to evaluate the structural retrofitting performances. Additionally, the maximum peak ground accelerations sustained by existing buildings before and after retrofit interventions have been commonly used as an evaluation metric [17]. However, most of these previous performance indexes mainly focus on earthquake effects, and other environmental factors like wind and snow are often neglected. In some cases, building collapse may not be primarily caused by seismic action, necessitating the consideration of all possible load situations when assessing structural retrofit approaches. Moreover, the compliance of retrofitted buildings with prevailing design codes is not directly reflected in these previous performance indexes.

To comprehensively evaluate retrofitted buildings under various load situations based on current design codes, this study introduces the residual design life evaluation framework for assessing retrofitted high-rise concrete buildings. This framework estimates the building's residual life by reversing the service life-based structural design procedure commonly employed in design codes of various countries, such as code GB 50009 [18] in China, code IS 875 [19] in India, code AS/NZS 1170 [20] in Australia and New Zealand etc. In addition to reflecting the building status under the adopted codes and various design load situations, such as wind and earthquake etc., the resulting residual design life offers a more intuitive measure for stakeholders compared to other performance indexes. Furthermore, the residual design life of retrofitted buildings can be utilized for life cycle analysis of older buildings. In the subsequent sections of this study, the developed residual design life evaluation framework is demonstrated, followed by a case study on a high-rise residential RC building to showcase its application. The influence of obsoleted and prevailing codes is revealed, and three structural level retrofit technologies, namely steel bracing, RC wall addition, and exoskeleton, are discussed based on the building's residual design life.

2. Residual design life evaluation of existing concrete buildings

2.1. Service life-based structural design

The service life-based structural design entails conducting structural analysis using service life-based loads, a practice adopted in numerous countries, including China, India, Australia, among others. As a crucial aspect of this design procedure, the determination of non-permanent loads based on the design life holds significant importance. Although slight variations may exist, the service life-based non-permanent load calculations in most countries follow a similar underlying principle. To maintain clarity and generality, this study utilizes the Chinese structural codes to exemplify the estimation of service life-based loads, which will be applied for further investigations. The service life-based designs of the live load, wind load and seismic action are outlined below, while other loads, such as snow action, are not detailed here but can be determined in a similar manner as specified in the codes.

2.1.1. Live load calculation

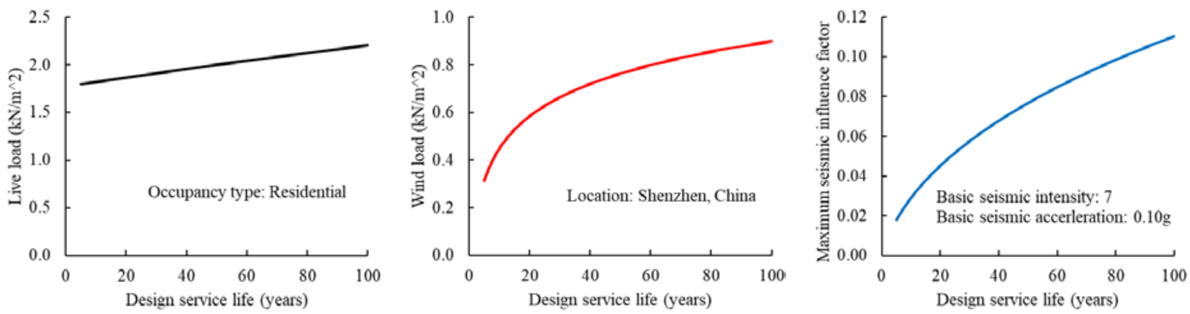
The Chinese code GB50009-2012 [18] uses the design life modification factor γ_L to calculate the live load Q (kN/m^2) at the design service life length of L, as demonstrated in Eq. (1). The factor γ_L can be determined by linearly interpolating the values presented in **Table 1**.

$$Q = \gamma_L Q_k \tag{1}$$

where Q_k is the reference value of the live load and is related to the type of occupancy or activity of the building. For residential buildings, Q_k is 2.0 kN/m^2 in the Chinese code. The typical live load for the given design working life is visualized in **Figure 1** (a).

Table 1 Design life modification factor γ_L specified in the Chinese code GB50009-2012

Design working life (years)	5	50	100
γ_L	0.9	1.0	1.1



(a) (b) (c)

Figure 1 (a)Live load, (b)Wind load and (c) Maximum seismic influence factor for the given design service life

2.1.2. Wind load calculation

The wind load W (kN/m^2) at the design service life length of L is calculated by Eq. (2) in the code GB50009-2012 [18].

$$W = \beta_z \mu_s \mu_z w_L \tag{2}$$

where β_z is the gust factor at the height of z, μ_s is the shape coefficient, μ_z is the height coefficient, and w_L is the basic wind pressure (kN/m^2) at the service life length of L on the given site and could be calculated by the

$$w_L = w_{10} + (w_{100} - w_{10})(\ln(L) / \ln 10 - 1) \tag{3}$$

where w_{10} and w_{100} are the basic wind pressure at the service life length of 10 years and 100 years, respectively, on the specified location. w_{10} and w_{100} could be found in the code. **Figure 1** (b) illustrated the wind load in Shenzhen China for the design life ranging from 5 to 100 years.

2.1.3. Seismic action calculation

For the given design life L, the design seismic intensity I_L on the given site could be calculated by the following equation [21]:

$$I_L = I_{up} - \left(I_{up} - I_\varepsilon\right)\left[-\ln\left(1 - P(I_L)\right)\right]^{\frac{1}{k}} \tag{4}$$

$$I_\varepsilon = I_b - 1.55 \tag{5}$$

$$P(I_L) = 1 - e^{-50/L} \tag{6}$$

where I_{up} is the upper limit of the seismic intensity and equal to 12 in China; I_ε is the mode value of the seismic intensity distribution on the given site; I_b is the basic seismic intensity on the given site and could be found in the Chinese code GB50011 [22], $P(I_L)$ is the exceeding probability of the design seismic intensity I_L within 50 years, k is the distribution shape factor of the seismic intensity on the given site. The value of k can be found in **Table 2** based on the basic seismic intensity and the basic seismic acceleration specified by the code GB50011 [22] for the given site. For example, the basic seismic intensity and the basic seismic acceleration of Shenzhen, China are 7 and 0.10g, respectively, so the corresponding distribution shape factor k is 8.3339.

Table 2 Shape factor k of the seismic intensity distribution on the given site

Basic seismic intensity (Basic seismic acceleration)	6 (0.05g)	7 (0.10g)	7 (0.15g)	8 (0.20g)	8 (0.30g)	9 (0.40g)
k	9.7932	8.3339	7.4788	6.8713	6.0132	5.4028

With the help of the design seismic intensity I_L, the maximum seismic influence factor α_{max} and the peak ground acceleration (PGA) (m/s^2) for the given design life could be estimated by the Eq.(7) and Eq.(8). By the means of the maximum seismic influence factor, the seismic action on structures could be calculated by the base shear method or response spectrum method following the design code. **Figure 1** (c) presented the maximum seismic influence factor α_{max} with respect to the design service life in Shenzhen China.

$$\alpha_{max} = \frac{\text{PGA}}{g}\beta_{max} \tag{7}$$

$$\text{PGA} = 10^{(I_L \cdot \log 2 - 2.1072)} \tag{8}$$

where g and β_{max} are the acceleration of gravity and the amplification factor, respectively. g is equal to 10 m/s^2, while β_{max} is equal to 2.25.

2.2. Residual design life evaluation framework

In this section, an iterative framework is proposed to evaluate the residual design life of existing buildings by reversing the service life design described in the last section, as depicted in **Figure 2**. The detailed steps of the framework are elucidated below.

1. **Measurement of Building Dimensions and Material Properties:** The dimensions and material properties of the existing building are ascertained. Advanced measuring apparatus, such as robotic total stations, aid in identifying the dimensions of structural components through site investigation. Non-destructive testing (NDT) methods, such as electromagnetic testing, ultrasonic pulse velocity test, and rebound method, are utilized to determine material properties like concrete and steel strength, as well as reinforcement contents. The material properties measured at this stage are indicative of the current material condition of the old buildings and encompass the impact of material degradation resulting from factors such as material durability reduction and environmental exposure during its prior service life, including phenomena such as strength degradation and section reduction. When employing the measured material characteristic strength for the residual life assessment, it is divided by a design factor of 1.4. This approach serves the dual purpose of ensuring a

conservative estimation and guaranteeing adequate durability throughout the remaining service life.

2. **Collection of Building Information:** Relevant information about the building location, type, and other features is gathered to facilitate the determination of design loads. Various load situations, including wind load, snow load, earthquake, and blast load, are simultaneously considered in this framework. For illustration purposes, three common loads, namely wind load, seismic action, and live load, are studied here. Other load types can be included similarly, if necessary. The initial design life L_i is set to 100, and the design value for each load type could be worked out using the service life-based load design procedure specified by the codes, as illustrated in Section 2.1.

3. **Development of Numerical Structural Model:** With the identified material properties and structural dimensions, a numerical structural model is constructed using software like ETABS, Staad Pro, or SAP2000. Different load effects on the existing structure are comprehensively considered using load combinations specified in the design codes. **Table 3** presents the load combinations specified by both the prevailing and obsolete Chinese codes for high-rise concrete buildings. Notably, according to code JGJ3-2010 [23], wind load and seismic action need to be considered simultaneously in load combinations only for buildings with a height over 60 m. As a result, wind load and earthquake load are not coupled in the combinations exhibited in **Table 3**. The design codes of other countries can be used to determine possible load combinations similarly, although the Chinese code is utilized here for illustration purposes.

4. **Evaluation of Structural Responses and Checking of Criteria:** The structural responses, such as internal forces on structural components (e.g., beams, columns) and building drift ratios, are evaluated using the developed structural model and various load combinations. Specific structural criteria, such as component capacity limits and drift ratio limits, are

checked. If these criteria are not met, the design life L_i is updated using the equation shown in **Figure 2**. With the design loads modified by the updated design life L_i, the structural responses are recalculated, and the criteria are rechecked. This updating and rechecking process continues iteratively until a converged design life L_i, which satisfies all required criteria, is obtained. This converged design life represents the residual life of the building. When checking only one aspect's criterion, the obtained residual life corresponds to the residual life of that specific aspect. For instance, when only the beam capacity is checked, the resulting residual life would be the residual life of the building's beam system. The envelope of all possible criteria represents the expected residual life of the entire RC building.

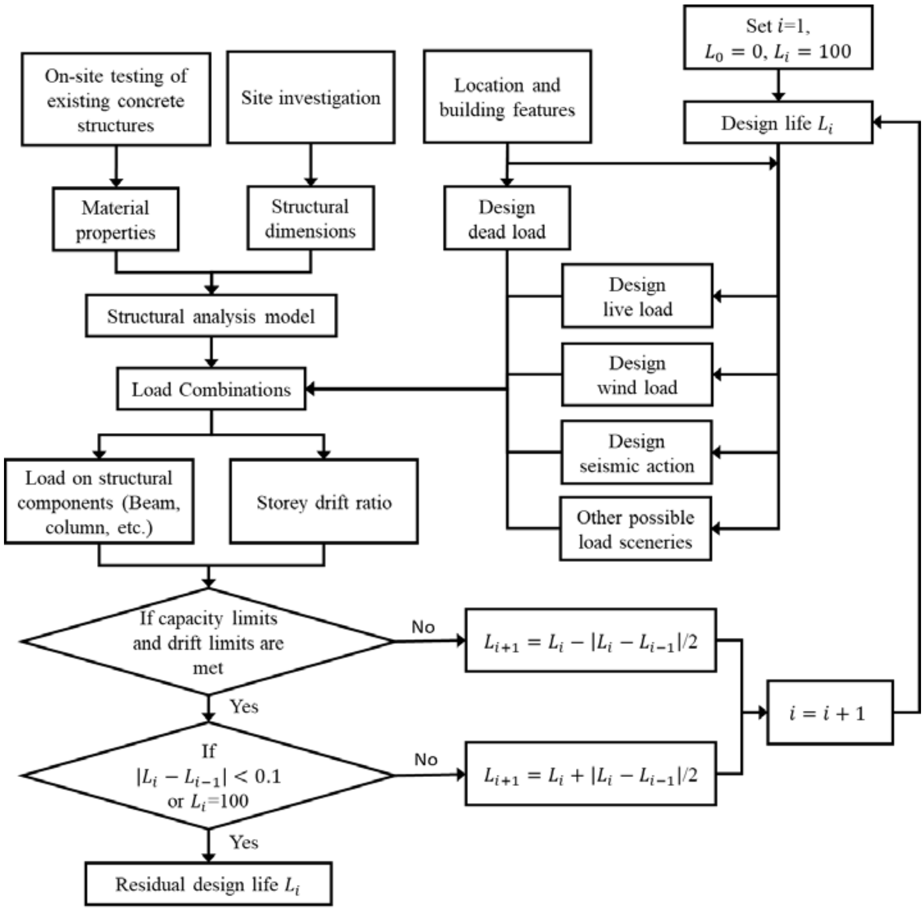


Figure 2 Residual design life evaluation framework for the existing building

Table 3 Load combinations given by both obsolete and current Chinese codes for the high-rise buildings with a height lower than 60 m

Combination type	No.	Load Combination	
		Obsolete code in China Before 2019 (GB50153-2008 [24])	Current code in China After 2019 (GB 50068-2018 [25])
Live load	1	$1.35DL + 1.4 \times 0.7LL$	
	2	$1.2DL + 1.4LL$	$1.3DL + 1.5LL$
	3	$1.0DL + 1.4LL$	$1.0DL + 1.5LL$
Wind load	4	$1.2DL \pm 1.4WL$	$1.3DL \pm 1.5WL$
	5	$1.0DL \pm 1.4WL$	$1.0DL \pm 1.5WL$
	6	$1.2DL + 1.4LL \pm 1.4 \times 0.6WL$	$1.3DL + 1.5LL \pm 1.5 \times 0.6WL$
	7	$1.0DL + 1.4LL \pm 1.4 \times 0.6WL$	$1.0DL + 1.5LL \pm 1.5 \times 0.6WL$
	8	$1.2DL \pm 1.4WL + 1.4 \times 0.7LL$	$1.3DL \pm 1.5WL + 1.5 \times 0.7LL$
	9	$1.0DL \pm 1.4WL + 1.4 \times 0.7LL$	$1.0DL \pm 1.5WL + 1.5 \times 0.7LL$
Seismic action	10	$1.2(DL + 0.5LL) \pm 1.3Eq$	$1.3(DL + 0.5LL) \pm 1.4Eq$
	11	$1.0(DL + 0.5LL) \pm 1.3Eq$	$1.0(DL + 0.5LL) \pm 1.4Eq$

Note:
DL: Dead Load; LL: Live Load; WL: Wind Load; Eq: Earthquake Action
Additional code considered for load combinations: JGJ3-2010 [23]

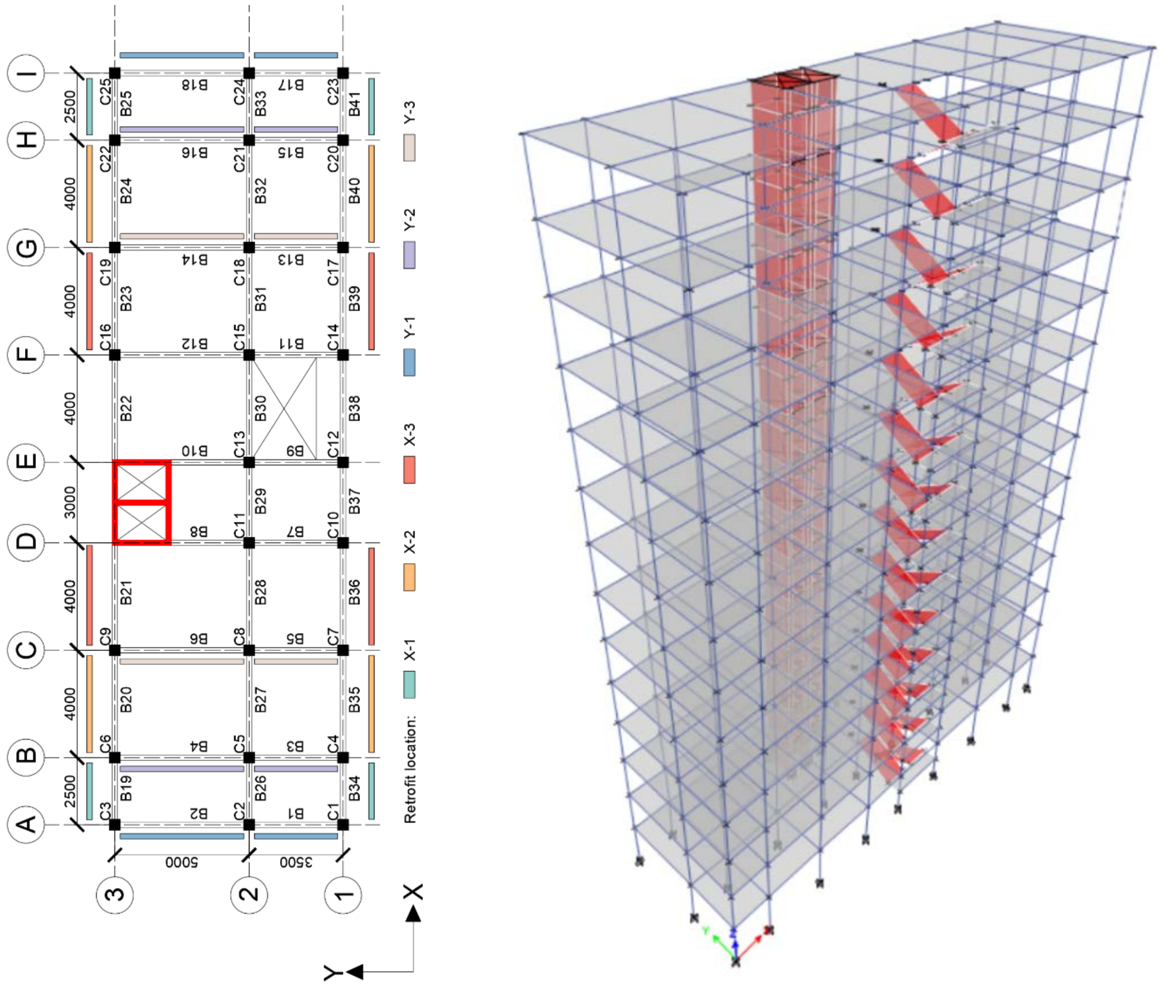
3. Case study on high-rise residential RC building

3.1. Model description

As a case study to demonstrate the residual design life evaluation framework for a retrofitted building, a typical G+13 storeys high-rise residential RC building situated in Shenzhen, China, is selected. The building's storey height is 3 m, and the storey plan is depicted in **Figure 3** (a). The Y-axis represents the weak axis, while the X-axis signifies the strong axis. For this particular building, the sizes of the beams and columns in the existing structure are 200 mm × 400 mm and 400 mm × 400 mm, respectively, with the configuration shown in **Figure 4**. The detected material characteristics of this existing building are assumed to follow the values detailed in **Table 4**.

The numerical structural analysis model developed by the software ETABs is presented in **Figure 3** (b). For this conventional residential building, linear elastic analysis is conducted using ETABS to simulate structural response and check the failure criterion both before and after retrofitting in the residual design life evaluation framework. The criterion for structural failure is established as the point at which any structural component reaches its capacity and enters the plastic phase or when the drift ratio reaches its limit, signifying the conclusion of the structure's service life. The formation of plastic hinges after entering the plastic stage is implicitly considered and avoided by ensuring that all elements remain below their capacity limits throughout this study. Consequently, the analysis herein does not account for post-failure collapse resulting from the formation of plastic hinges, as all structural elements are controlled to remain within the elastic stage. This elastic analysis approach, grounded in engineering practice, is conservative in nature. It should be noted that the consideration of post-failure collapse is beyond the scope of this study. However, the post-failure collapse analysis and plastic analysis could also be used for checking the failure state of the structure and will be integrated into our evaluation framework in forthcoming studies.

Following the guidelines outlined in code JGJ3-2010 [23], the prescribed drift limit for this building has been established at 1/800 (equivalent to 0.125%). In our evaluation, the capacities of the beams are assessed based on considerations of both bending moment and shear load, whereas the capacities of the columns are appraised in terms of axial load, bending moment, and shear load. It is noteworthy that the assessment of the bending moment in the columns is not conducted in a direct manner; rather, it is transposed into an eccentric distance of the axial load and is concurrently verified with the axial load. For a more comprehensive understanding of the evaluation process, the detailed equations used for assessing the capacities of the beam and column elements are elucidated in Appendix. Capacity calculations. Based on our evaluation, it is evident that all beams within the existing building fall into the category of under-reinforced beams, satisfying the necessary ductility requirements for concrete beams. Furthermore, the ductility of the existing column elements has been ensured by appropriately configuring the stirrup reinforcements and adhering to detailing specifications. Since our study exclusively employs linear elastic analysis, the influence of the ductility of the reinforced concrete frame elements on the results is deemed negligible. Considering the importance of the ductility, we plan to explore the incorporation of the ductility analysis into the residual design life evaluation framework in future studies.



(a) Plan View (b) 3D view

Figure 3 Typical G+13 storeys high-rise residential building in Shenzhen China: (a) Storey plan view and (b) Numerical structural analysis model.

Table 4 Features of the existing building.

Building features	Values
Concrete characteristic compressive strength f_{ck} (MPa)	19.6
Concrete characteristic tensile strength f_{tk} (MPa)	1.89
Failure strain of concrete ε_{cu}	0.0033
Steel yield strength f_y (MPa)	345
Elastic modulus of concrete E_c (GPa)	30
Elastic modulus of steel reinforcement E_s (GPa)	200

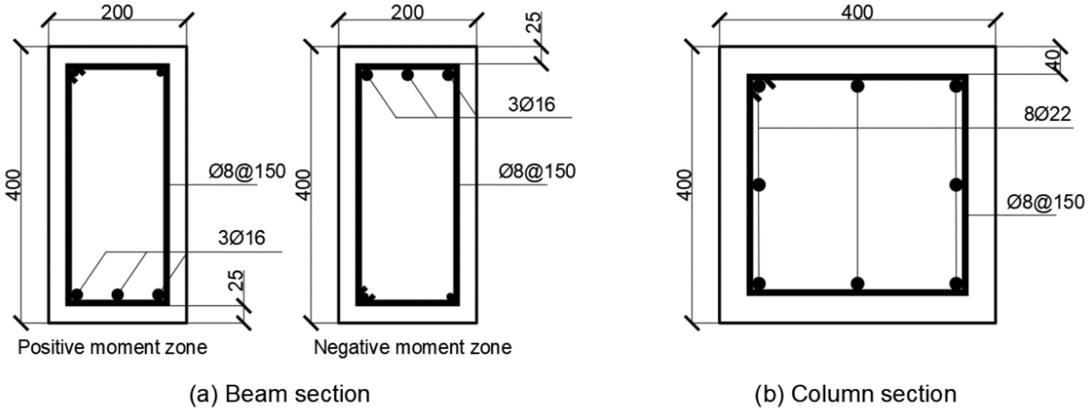


Figure 4 Section configurations of beams and columns

3.2. Structural retrofit technologies

In this study, three global-level (i.e., structure-level) retrofit interventions recommended by the fib bulletin 24 [4] are adopted. These retrofit strategies, namely steel bracing, RC wall addition, and exoskeleton, are used to enhance the structural performance of the existing building. The following sections provide detailed descriptions of each retrofit technology, and **Figure 5** illustrates their schematic diagrams. The chosen retrofit locations are visually represented in **Figure 3** (a). These locations were carefully selected to encompass most of the structural bays within the case structure and include most intuitively conceivable retrofit positions. It is important to note that, in our approach, we avoided retrofitting only a few floors in the middle of the building to prevent potentially worsening the axial performance of the existing columns. Therefore, all retrofit interventions begin from the ground level and gradually encompass the entire building, ensuring that no additional load is imposed on the existing structure. However, it is crucial to emphasize that the primary purpose of the case building and the retrofit strategies applied in this study is to illustrate the proposed residual design life evaluation framework. Consequently, the precise selection of optimal retrofit locations falls beyond the scope of this particular research endeavour and will be a subject of further investigation in future studies.

Given that this study focuses on structural or global-level retrofitting, the strength and ductility of the existing elements in the RC frame remain unchanged before and after retrofitting. The ductility of the newly added retrofitting elements, such as steel braces, steel exoskeleton, and RC walls, is ensured by adhering to the detailing requirements specified in the design code. In addition, the design principles of "strong column weak beam" and "strong shear capacity weak bending capacity" are enforced by retrofitting to ensure the ductility of the retrofitted structure.

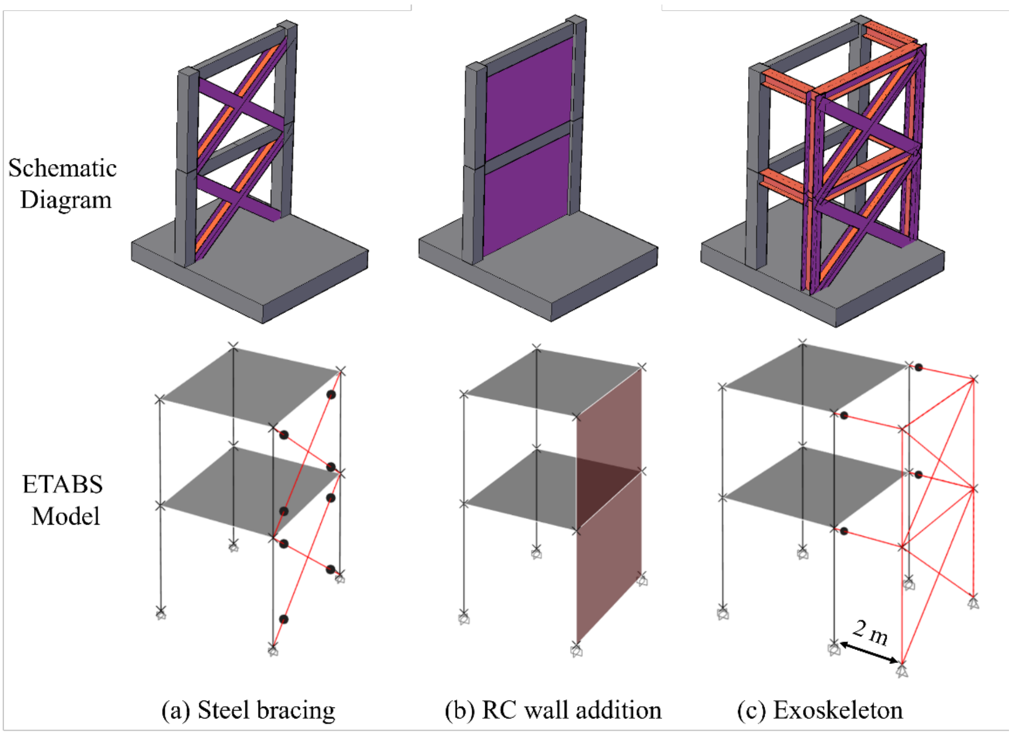


Figure 5 Schematic diagrams and ETABS models of three structural level retrofit technologies (The grey one is the existing old building, and the colour is the retrofit strategies, the black dot indicate the pin connection)

3.2.1. Steel Bracing

The steel bracing retrofit involves the addition of steel members in the bays of the existing concrete frame to enhance the overall capacity and stiffness of the building structure. By providing steel braces, the deformation of the concrete frame is effectively restrained, and the load-transferring path is modified to enhance the building's lateral stability and resistance. The typical cross steel bracing form, as depicted in **Figure 5** (a), is adopted for this retrofit strategy in this study. It is modelled through the use of brace elements within the ETABS software, as shown in **Figure 5** (a). The steel braces possess a cross-section dimension of HW350x350x10x16 and belong to the Q345 strength grade. They are connected to the existing RC structures via pin connections, which is modelled by releasing the in-plane end moments of the steel braces in ETABS. The length of the steel braces is equivalent to the diagonal measurement of the selected bay. The retrofitting work involves six different locations or frame bays, namely X-1, X-2, X-3, X-4, X-5, and X-6, which are labelled in **Figure 3** (a) for reference.

3.2.2. RC wall addition

Similar to the steel bracing approach, the RC wall addition method involves the installation of additional shear walls to occupy specified bays within the concrete frames, as depicted in **Figure 5** (b). These newly introduced RC walls serve the purpose of providing ample support to the designated bays, effectively carrying axial loads, and mitigating the lateral deformation of the building. The RC wall retrofit intervention is modelled using wall elements in ETABS, as illustrated in **Figure 5** (b). These RC walls have a thickness of 200 mm and a characteristic compressive strength of 22.5 MPa. The added RC wall is firmly attached to the existing column without any release of moments at its edges. The dimensions of the RC wall match the opening of the selected bay, ensuring complete coverage. Just like with the steel bracing, six retrofitting positions are considered for the RC wall addition, as illustrated in **Figure 3** (a).

3.2.3. Exoskeleton

The exoskeleton is an external structural system utilized to provide additional lateral resistance to the existing building. Compared to the other two retrofit approaches, the exoskeleton involves less disruption to the occupants within the building, but it necessitates clear space around the structure for implementation. In this study, the steel exoskeleton, as depicted in **Figure 5** (c), is employed to retrofit the RC building. Its beams, columns, and braces are represented using beam, column, and brace elements in ETABS, respectively. All elements within the exoskeleton share a cross-sectional profile of HW350x350x10x16 and belong to the Q345 strength grade. The exoskeleton is positioned 2 m away from the selected bay and is connected to the bay through horizontal beams. The connection between the exoskeleton and the existing building employs pin connections, with in-plane end moments released in the ETABS model. Due to the need for placement outside the building, only four retrofit locations are considered, namely X-1, X-2, X-3 and Y-1. The exoskeleton is strategically positioned outside the specified frame bays as shown in **Figure 5** (c).

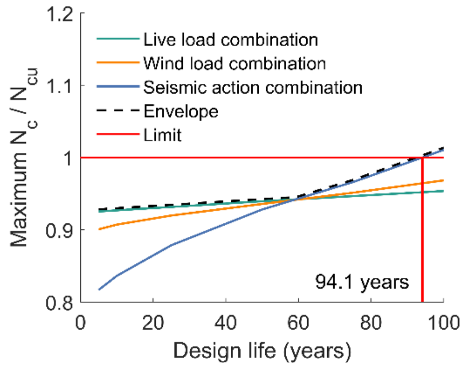
3.3. Results and discussions

3.3.1. Performance of existing buildings

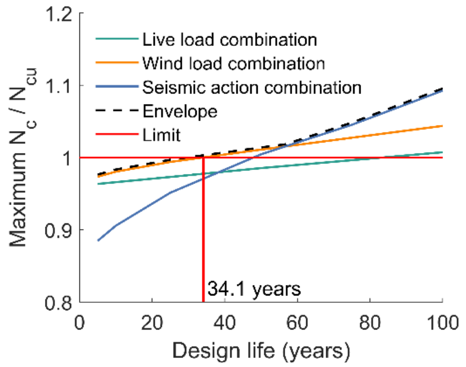
In accordance with both the outdated and current building codes, we have computed the maximum load-to-capacity ratios for structural elements, including beams and columns, as well as the maximum ratios of the drift ratio to its limit, at various design life stages, as depicted in **Figure 6**. In the case of columns, we have separately presented the axial load N_c and shear load V_c. The bending moment is indirectly considered as it is concurrently assessed through its translation to eccentric distance of the axial load. It has already been included within the eccentric axial capacity N_{cu}, as demonstrated in Appendix. Capacity calculations. For the beams, both bending moment M_b and shear load V_b are displayed.

When applying the outdated building code, we observe that abrupt shear failure is not anticipated for this building. Shear loads are not the critical factors for either columns or beams, aligning with the overarching design principle of "strong shear capacity and weak bending capacity." The failure modes for the existing column and beam systems are primarily influenced by axial loads and bending moments, respectively. The seismic load combination and wind load combination constitute the critical scenarios for existing columns and beams, respectively. As the inner column, column C18 bears a higher axial load than others. Due to the presence of the stairwell, column C18 experiences less constraint compared to its counterpart, column C8.

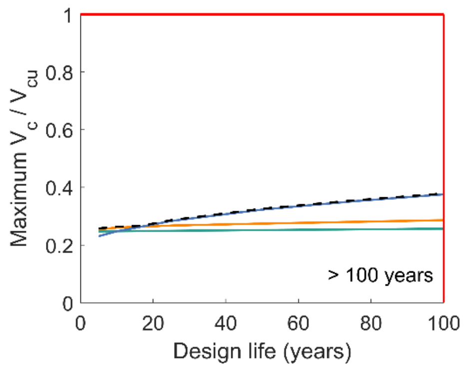
Consequently, column C18 governs the failure of the column system. Furthermore, beams oriented along the Y direction bear a greater load compared to those in the X direction. Beam B10 emerges as the critical component in the beam system, experiencing the most significant deformation constraints from the shear wall and frames along axes 1 and 2, consequently resulting in the highest bending moment at its ends. As depicted in **Figure 6** (a) and (e), the existing beams are capable of withstanding loads corresponding to a design life of 65.6 years, whereas the residual lifespan of the existing columns exceeds 94.1 years. This indicates that beams represent the critical structural elements in the existing building and would likely fail before the columns, aligning with the fundamental design principle of "strong column and weak beam.". In terms of storey drift, as shown in **Figure 6** (i), it increases with an extended design life, with seismic action exerting a more significant influence than wind load when the design life surpasses 35 years. This underscores the dominance of seismic action in governing storey drift. Considering drift as a factor, the residual design life of the building is estimated to be 44.9 years. Thus, it can be concluded that the residual design life of the building is primarily controlled by storey drift induced by seismic action and amounts to 44.9 years. Notably, this residual design life, as assessed by the outdated standards, closely aligns with the desired service life of 50 years.



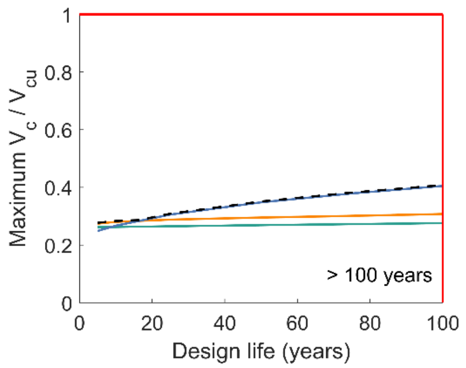
(a) Maximum ratio of axial load to its capacity for columns (obsolete code)



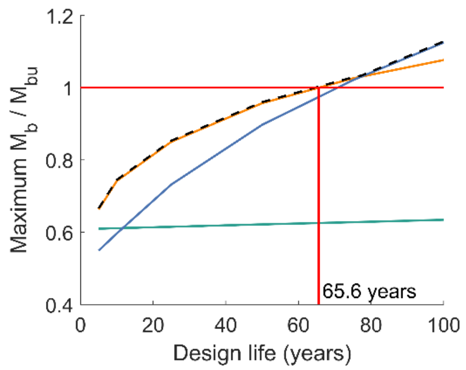
(b) Maximum ratio of axial load to its capacity for columns (current code)



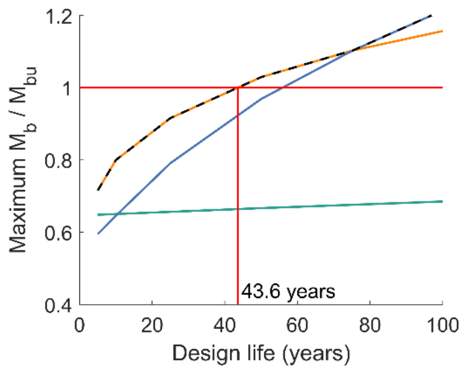
(c) Maximum ratio of shear load to its capacity for columns (obsolete code)



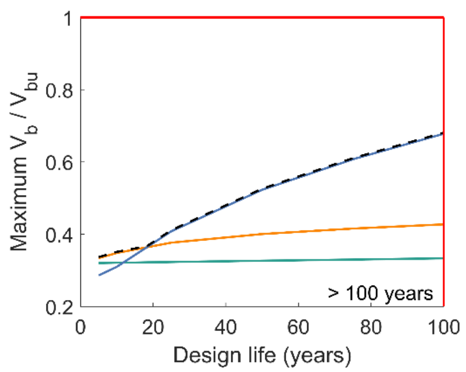
(d) Maximum ratio of shear load to its capacity for columns (current code)



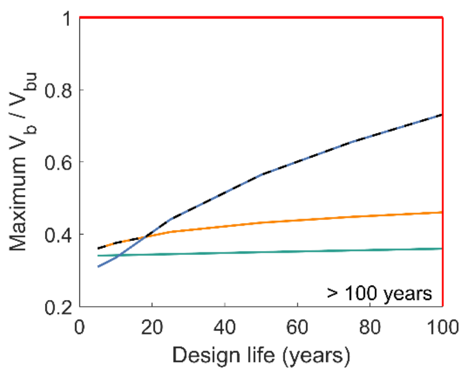
(e) Maximum ratio of bending moment to its capacity for beams (obsolete code)



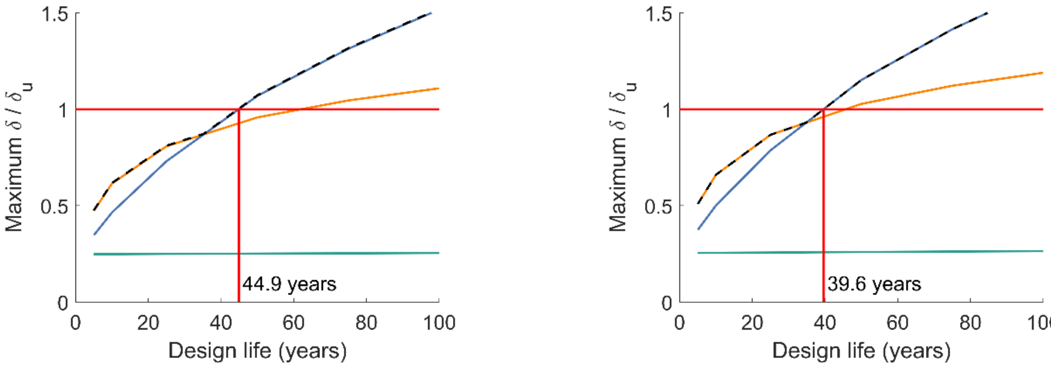
(f) Maximum ratio of bending moment to its capacity for beams (current code)



(g) Maximum ratio of shear force to its capacity for beams (obsolete code)



(h) Maximum ratio of shear force to its capacity for beams (current code)



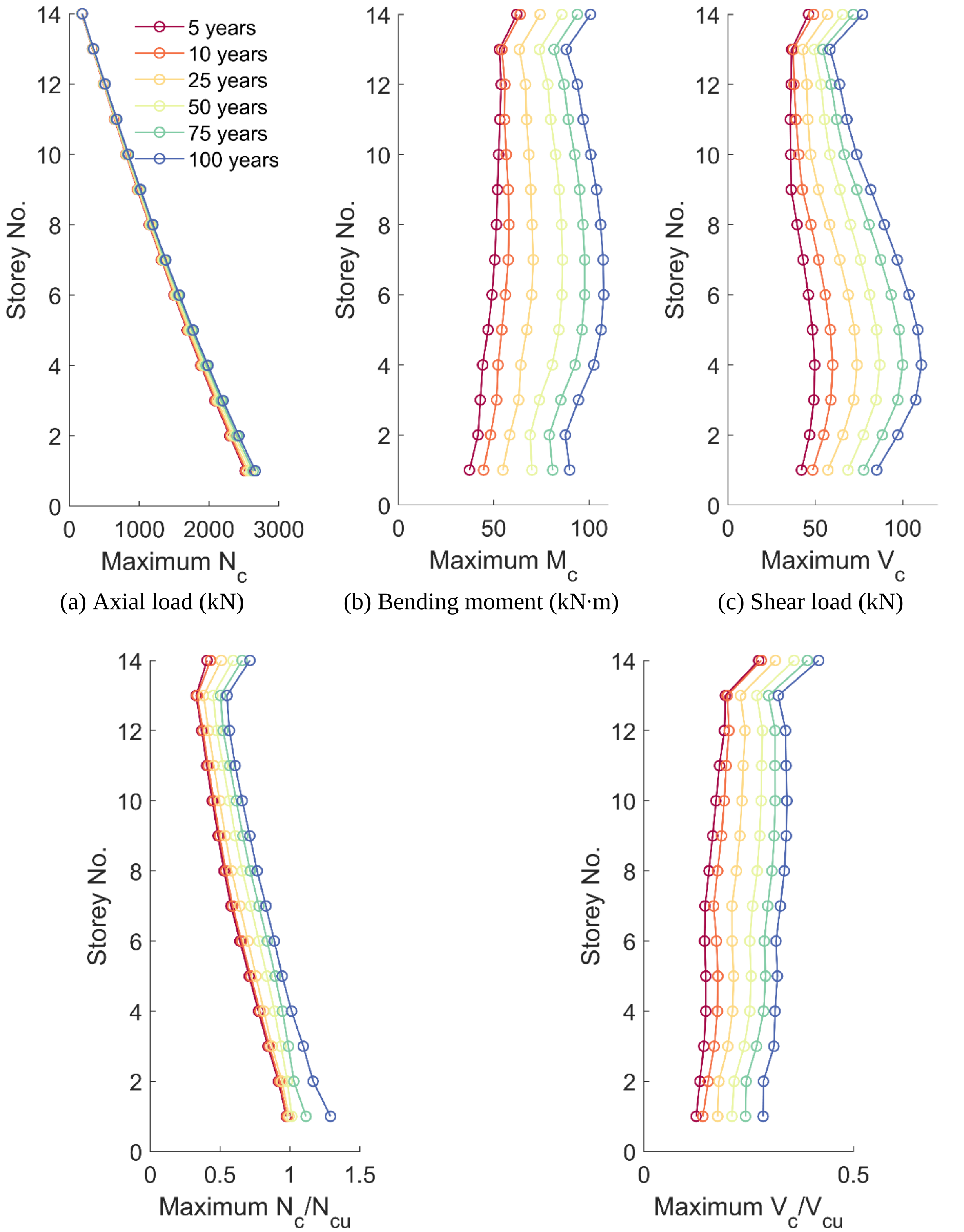
(i) Maximum ratio of drift ratio to its limit (obsolete code)

(j) Maximum ratio of drift ratio to its limit (current code)

Figure 6 Maximum (load to capacity ratio) of structure by considering different load combinations given by the obsolete code (GB50153-2008) and the current code (GB50068-2018)

With the transition from the obsolete version of the design code to the current edition, the existing beams and columns face heightened design load requirements, consequently resulting in a reduction in the residual design life of these structural elements. Notably, shear load remains a non-dominating factor for both columns and beams, reaffirming the fundamental design principle of "strong shear capacity and weak bending capacity". Beams continue to be primarily governed by bending moments under the wind load conditions. In contrast, columns are no longer predominantly affected by seismic combinations, as observed in the obsolete code, but are now controlled by axial loads under wind load combination. It's noteworthy that the critical beams and columns within the frame system remain consistent with those identified in the previous code edition. However, the residual design life of the column system experiences a notably higher reduction rate compared to that of the beam systems due to increased axial loads and bending moment, diminishing dramatically from 94.1 years to 34.1 years. In contrast, the residual design life of the beams decreases from 65.6 years to 43.6 years. This significant reduction in the column system's residual life leads to a violation of the "strong column and weak beam" design principle, underscoring the imperative need for retrofitting existing buildings following the adoption of the new code. Additionally, the updated design code results in a substantial increase in storey drift. Seismic load cases continue to govern the residual design life based on storey drift, which is determined to be 39.6 years. Considering both beam and column capacity along with storey drift, the building's overall residual design life concludes at 34.1 years. Intriguingly, unlike the previous design code, the residual design life is now primarily influenced by column capacity rather than storey drift. Furthermore, it's essential to highlight that for this specific building, the residual design life determined using the current standard is 24.1% lower than that calculated under the obsolete standard. This reduction fails to meet the safety requirement of a 50-year design working life. Consequently, retrofitting the building is imperative to extend its residual life and ensure the safety of its occupants.

To conduct a thorough analysis of the governing components, which in this case are the columns within the entire building structure, we have visualized the maximum axial load, bending moment, and shear load of the column systems at various storeys in **Figure 7** (a-c). The axial load exhibits a gradual decrease with an increase in the number of floors, with the maximum axial load occurring on the lower floors. This phenomenon arises because the lower storeys bear the cumulative loads transmitted from the floors above. As the axial load is primarily a consequence of the dead load and live load, it experiences minimal influence from variations in the design life. In contrast, both the bending moment and shear load exhibit an initial increase followed by a decrease as one moves higher in the building. The peak values for bending moment and shear load manifest at intermediate heights within the structure. These two factors, bending moment and shear load, predominantly stem from wind and seismic actions, making them highly susceptible to changes in the design life.



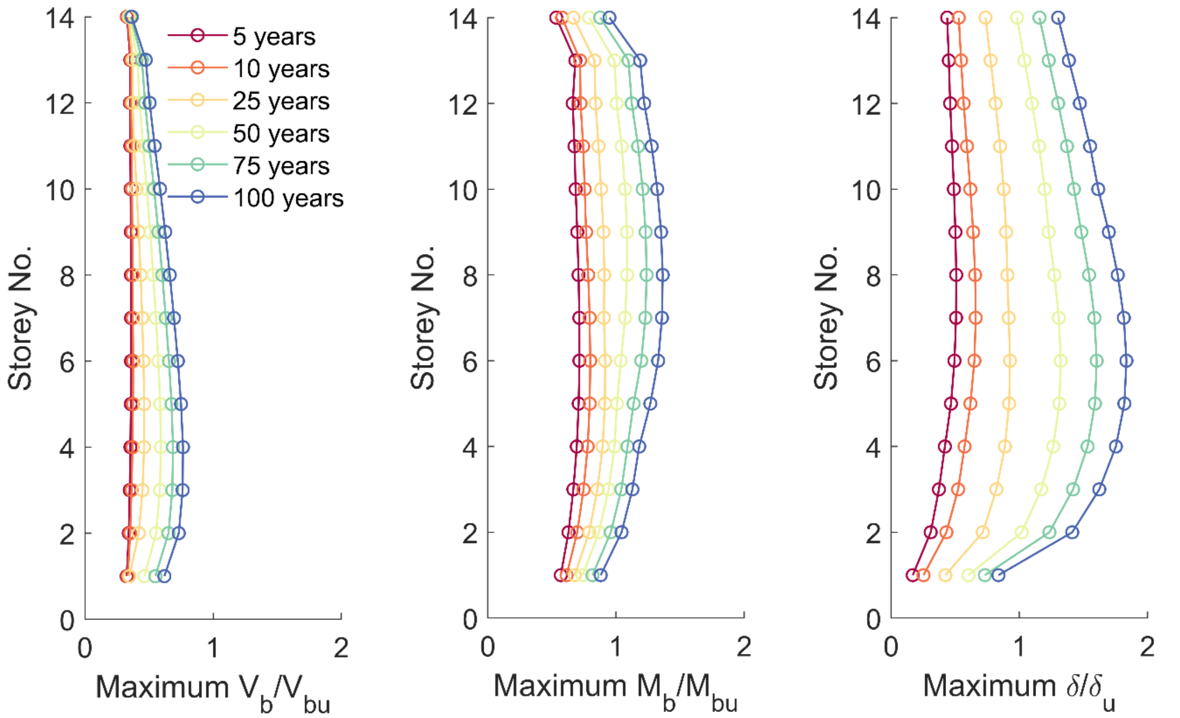
(a) Axial load (kN) (b) Bending moment (kN·m) (c) Shear load (kN)

(d) Maximum ratio of axial load to its capacity (e) Maximum ratio of shear load to its capacity

Figure 7 Maximum loads on the columns, and maximum ratios of the loads to the corresponding capacities at different storeys and different design years based on the current code.

The maximum ratio of axial load N_c to capacity N_{cu}, which considers both the effect of the bending moment and the axial load, and the maximum ratio of shear load V_c to capacity V_{cu}, which considers both the effect of the shear load and the axial load, are illustrated in **Figure 7** (d-e). Different from the axial load N_c shown in **Figure 7** (a), the ratio N_c/N_{cu} is more sensitive variations in the design life, owing to the significant influence of bending moments on the axial load capacity N_{cu}, as illustrated in Eqs. (13)-(17). Due to the benefits of axial compression on shear capacity V_{cu}, the distribution of the ratio V_c/V_{cu} across different storey heights is more uniform than that of shear load V_c. As the dominant factor governing the column system, the ratio N_c/N_{cu} also plays a pivotal role in determining the behaviour of the entire structure, since the structure failure is primarily governed by the column system after adopting the current code. The ratio N_c/N_{cu} exhibits larger values on the lower storeys, underscoring why the retrofitting process conducted in this study initiates from the ground level rather than the mid-height of the building.

On the other hands, the maximum ratios of the bending moment M_b to its capacity M_{bu} and the shear load V_b to its capacity V_{bu} of the beam systems are also presented in **Figure 8**, alongside the maximum ratio of the drift ratio δ to its limit δ_u. Since the bending and shear capacities of the beams remain constant throughout the entire structure, both M_b and V_b exhibit the same trends as the ratios M_b/M_{bu} and V_b/V_{bu}. Therefore, we have not presented them separately in this context. These three indexes, namely the maximum M_b/M_{bu}, maximum V_b/V_{bu} and maximum δ/δ_u, display a consistent pattern of ascending and descending trends with an increase in storey height, culminating at their peaks around the middle height of the building. Additionally, they exhibit a high degree of sensitivity to variations in the design life, primarily because they are profoundly influenced by lateral forces, namely wind loads and seismic actions. In terms of sensitivity levels, these indices can be ranked from high to low sensitivity as follows: drift ratio, bending moment, and shear load. At longer design life spans, the drift ratio becomes more critical, whereas at shorter design life spans, the bending moment takes precedence.



(a) Maximum ratio of shear load to its capacity for beams

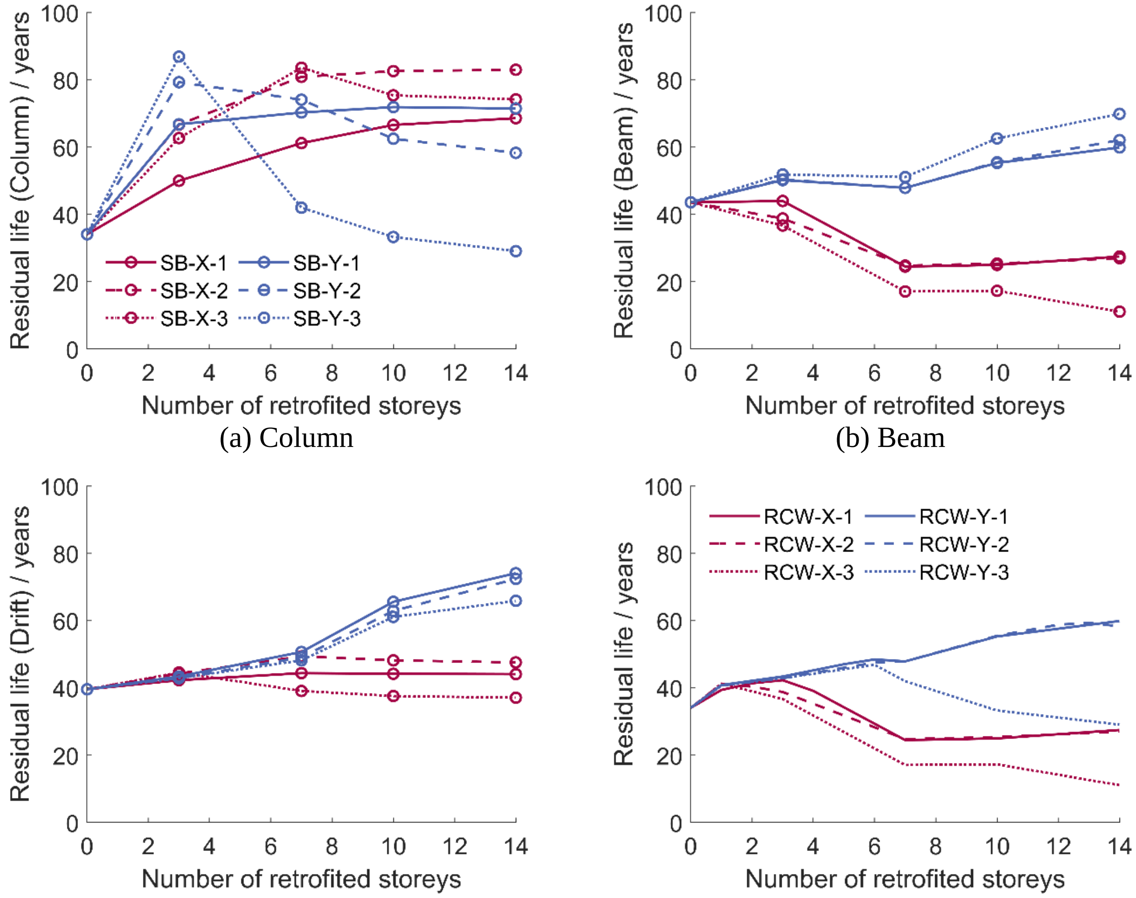
(b) Maximum ratio of bending moment to its capacity for beams

(c) Maximum ratio of drift ratio to its capacity for beams

Figure 8 Maximum ratios of the shear load on beams, bending moment on beams and drift ratio to the corresponding capacities at different storeys and different design years based on the current code.

3.3.2. Effect of Steel Bracing

After employing the steel bracing retrofitting, **Figure 9** exhibits the residual design lives considering the beam system, column system, and drift ratio, as well as the envelope of the three. The residual design life of the beam systems has taken the bending moment, and shear load into account, while the axial load, bending moment and shear load on the column have also been involved in the residual life of the column system. The number of retrofitted storeys serves as the x-axis for **Figure 9**, as well as **Figure 10** and **Figure 11** in the rest of this study. For clarity, when the number of retrofitted storeys is 7, it implies that the retrofitting process has covered the first 7 floors of the structure, commencing from the ground level. Likewise, when the number of retrofitted storeys is 14, it indicates that the entire structure has undergone retrofitting.



(a) Column (b) Beam

(c) Drift (d) Envelop

Figure 9 Residual life of the building retrofitted by the steel brace (SB) considering (a) beam performance, (b) column performance, (c) storey drift, and (d) envelop performance

When the strong axis of the existing building is reinforced with a steel brace at retrofit locations X-1, X-2, and X-3, the residual life of the column system could be effectively increased, as shown in **Figure 9** (a). This is because the inclusion of the X-axis steel brace facilitates the transfer of load from the critical column to the ground and the adjacent columns along the X-axis, ultimately enhancing the axial load distribution within the column system. With the retrofit location moving from the outer side to the inner side, namely from X-1 to X-2, the effectiveness of the steel bracing is enhanced. However, when the steel bracing is placed on location X-3, the highly strengthened building stiffness makes the control case change from wind load to the seismic action and induces the additional bending moment on the column, which results in a lower residual life than location X-2. In contrast, the residual life of the beams progressively decreases with an increasing number of retrofitted storeys, as depicted in **Figure**

9 (b). For retrofit locations X-1, X-2, and X-3, the steel brace gradually shifts towards the centre of the building, resulting in a transfer of critical beam from B10 to B16, B14, and B14, respectively. Following retrofitting, the beam in closest proximity to the added steel brace becomes the critical beam due to the structural frame's enhanced deformation constraint along the weak axis (i.e., the Y-axis), ultimately leading to an increase in load for these beams. Since the building's storey drift is predominantly controlled by the stiffness along the weak axis, the addition of the steel bracing on the strong axis has a minor effect on the storey drift, as evidenced in **Figure 9** (c).

In **Figure 9** (a), it can be observed that the utilization of steel bracing along the weak axis at retrofit locations Y-1, Y-2, and Y-3 effectively improves the residual design life of the column system when only the first three storeys are retrofitted. However, by placing steel braces along the Y axis, the load is transferred from the inner columns to the edge columns, resulting in an increased load on the edge columns. Retrofitting at locations Y-2 and Y-3 causes axial load redistribution along the Y axis, leading to an amplified axial load on edge columns C19 and C22 respectively. Therefore, these edge columns become the critical components, resulting in the reduction of the residual life of the column system when the number of retrofitted floors is high, as evidenced in **Figure 9** (a). **Figure 9** (b) demonstrates an increasing trend in the residual life of the beams with each retrofitted storey. Nonetheless, Beam B10 remains critical in the beam system. The improvement in beam life can be attributed to the fact that the added steel braces partly transfer the end load on the beams along the weak axis to the columns. Furthermore, the reduced storey drift diminishes the deformation-induced load on the beams during wind and seismic actions, ultimately leading to an increase in beam residual life. In **Figure 9** (c), it can be observed that the utilization of steel bracing along the weak axis at retrofit locations Y-1, Y-2, and Y-3 effectively reduces the drift of the entire building. It is important to note that the frames on the sides (axes A and I) are weaker compared to the frames in the middle (axes C and G). As a result, the steel bracing at location Y-1 proves to be more efficient than the bracing at location Y-3 in reducing building deformation and prolonging the residual life caused by storey drift.

Figure 9 (d) presents the overall residual life of the retrofitted building, taking into account the beam, column, and storey drift. The introductions of steel braces at both locations Y-1 and Y-2 present a similar retrofitting performance and are more effective than the others, resulting in a more than 70% increase in the building's residual life. Notably, each retrofitted storey contributed to a notable 5% enhancement in the residual life.

3.3.3. Effect of RC wall addition

In comparison to steel bracing, retrofitting with RC shear walls proves to be notably more effective in improving the residual life of the column **Figure 10** (a). Regardless of the retrofit location, the residual design life of the column system could be enhanced more than twice and reach over 80 years after the whole storeys are retrofitted. This is because the RC wall could directly carry the axial load and transfer it to the ground, unlike the steel bracing which mainly transfers the axial load to the other components. The benefit of the RC wall in reducing the building storey drift is also remarkable, as demonstrated in **Figure 10** (c). Even when the RC wall is positioned along the strong axis, specifically at locations X-1, X-2, or X-3, it still contributes to a considerable mitigation of lateral deformation in the building. However, the most significant reduction is achieved when the RC wall is added to the core of the building, rather than at its periphery.

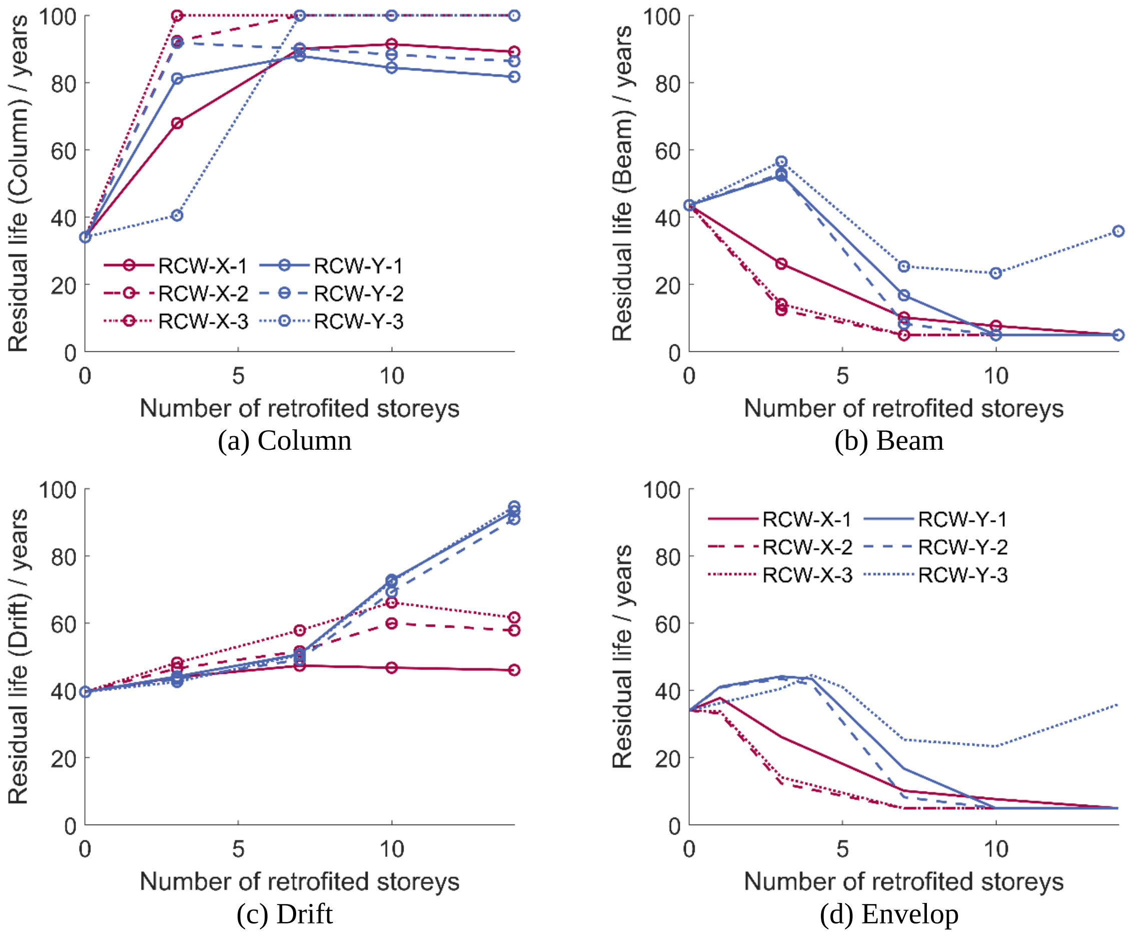


Figure 10 Residual life of the building retrofitted by the RC wall (RCW) considering (a) beam performance, (b) column performance, (c) storey drift, and (d) envelop performance

On the one hand, the addition of RC walls significantly increases the bending load on beams adjacent to the walls while reducing the load on beams that are not adjacent to them. This effect arises because the RC walls enhance the lateral deformation-induced forces on the adjacent beams. When the RC wall is positioned along the X-axis, the load on the beams along the weak axis (Y-axis), which are adjacent to the wall, experiences a substantial increase. Given that beams along the weak axis already carry larger loads compared to those along the strong axis, the augmented load on the beams along the weak axis can lead to a considerable drop in the residual life of the beam system, as indicated in **Figure 10** (b). Locations X-2 and X-3 have more adjacent beams than location X-1, suggesting that the reduction in beam residual life caused by the RC wall at location X-1 is milder than the effects observed at the other two locations. Conversely, when the RC wall is placed along the Y-axis, the bending loads on the beams along the Y-axis are mitigated, but the conditions of the beams along the X-axis are worsened. Given that the beams along the Y-axis govern the failure of the beam system, the residual life of these beams initially increases with the number of retrofitted storeys. However, as the number of retrofitted storeys continues to rise, the beams along the X-axis, adjacent to the added RC wall, start to dominate the failure status of the beam system, leading to a decrease in beam residual life.

Considering these three aspects collectively, the retrofitting performance of adding RC walls, evaluated by the building's residual life, is demonstrated in **Figure 10** (d). Among the six retrofitting locations, locating the RC wall at Y-3 exhibits the most outstanding performance. The overall residual life of the building retrofitted with the RC wall at location Y-3 initially increases and then decreases with the number of retrofitted storeys. During the rising stage, storey drift and column behaviours governs the overall building performance, while beam conditions become the determining factor during the decreasing stage. Optimal retrofitting is achieved when the RC wall is added to the first 4 storeys of the building, resulting in a 30.8% increase in residual life. Considering that the RC wall induces additional loads on the beams adjacent to it, it is recommended to implement component-level retrofitting to strengthen the beams around the RC wall when adopting the RC wall addition retrofitting. This approach ensures the full utilization of the RC wall and further extends the building's residual life.

3.3.4. Effect of exoskeleton

The utilization of the exoskeleton could partially carry the axial load and improve the residual design life of the column system, as shown in **Figure 11** (a). With the exoskeleton moving from the corner to the centre, namely from location X-1 to X-3, the exoskeleton could more effectively reduce the axial load on the critical column and elongate the residual life. Since both the locations X-2 and X-3 have the same distance to the inner critical column C18, these two retrofit locations exhibit a similar effect on reducing the axial load on the critical members and enhancing the residual life. The location Y-1 presented more benefit on the column system when the number of retrofitted storeys is low, since it could enhance the columns on three strong axes simultaneously and has less distortion on the geometry. However, when the whole structure is retrofitted, the Y-1 is not as effective as the locations X-2 and X-3, as X-2 and X-3 are closer to the critical columns and could share more loads from the critical column.

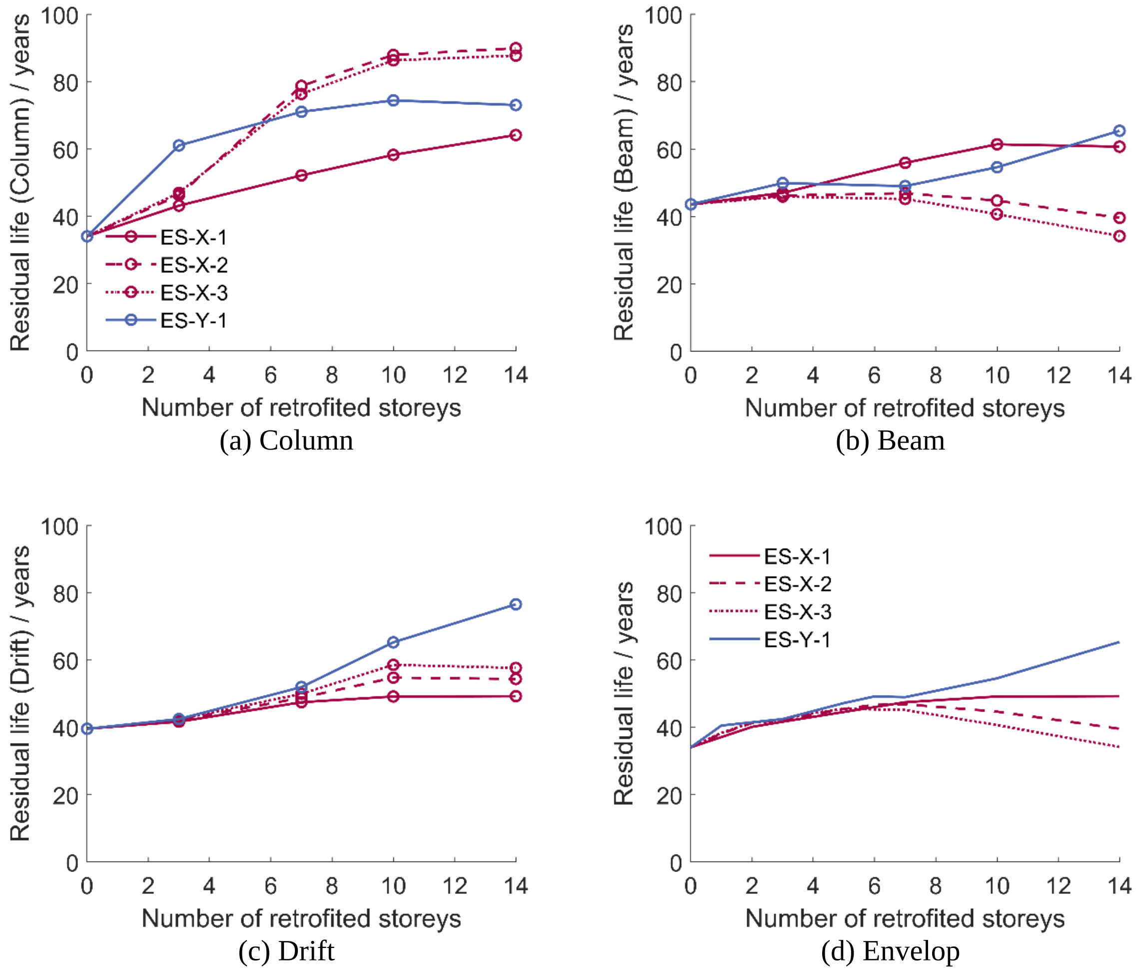


Figure 11 Residual life of the building retrofitted by the steel exoskeleton (ES) considering (a) beam performance, (b) column performance, (c) storey drift, and (d) envelop performance

Similar to the RC wall, the addition of the exoskeleton enlarges the bending loads on beams adjacent to and along the exoskeleton while reducing the loads on other beams. For instance, the exoskeletons at location Y1 increase the bending loads on beams B16, B25, B26, B33, B34, and B41 but decrease the loads on other beams. Notably, beams positioned at the core of the building bear higher loads compared to those at the edge. The exoskeletons at edge locations, such as Y-1 or X-1, facilitate a more uniform distribution of bending loads on the beam system by raising the loads on beams at the edge and reducing the loads on beams at the core. Consequently, **Figure 11** (b) demonstrates an improvement in the residual life of the beam systems when exoskeletons are added at locations Y-1 and X-1. However, exoskeletons placed at locations X-2 and X-3 aggravate the bending loads on beams at the core, consequently worsening the residual life of the beam system, as observed in **Figure 11** (b).

The steel exoskeleton also exerts a positive effect on the residual life concerning storey drift, as illustrated in **Figure 11** (c). Comparing the location of the exoskeleton along the weak axis (i.e., locations X-1, X-2, and X-3), employing the exoskeleton to reinforce frames along the strong axis (i.e., location Y-1) proves to be more effective in mitigating building lateral deformation. This is because adding the exoskeleton to frames along the weak axis would lead to some distortion in the building's geometry, except in cases where all frames along the weak axis are enhanced by the exoskeleton. However, enhancing all frames along the weak axis would incur higher costs and is not an economically viable solution. The distortion in geometry results in an amplification of deformation under lateral loading, counteracting the benefits provided by the exoskeleton. As the exoskeletons along the weak axis are moved from the building's edge towards the core, this distortion effect weakens. Consequently, adding the exoskeleton at location X-3 demonstrates better performance than at locations X-1 and X-2.

The residual life envelope curves considering three performance indexes are illustrated in **Figure 11** (d). The comprehensive residual life improvement achieved with the exoskeleton at position Y-1 surpasses the other three strategies. Retrofitting the entire building with the exoskeleton in the optimal position results in a remarkable 91.8% increase in the building's residual life, reaching 65.4 years. The steel exoskeleton exhibits a more attractive capability than the other two retrofit technologies, with fewer side effects. However, it should be noted that the utilization of the exoskeleton requires additional space around the existing building.

4. Conclusion

In order to comprehensively assess retrofitted old RC buildings under various loading conditions and provide a design code-based intuitive index, this study introduces a residual design life evaluation paradigm by reversing the service life-based structural design procedure. The developed paradigm is demonstrated through a case study on a high-rise residential RC building. The results indicate that adopting the prevailing design code may significantly reduce the expected residual design life of existing buildings originally designed by an outdated code. Retrofitting becomes necessary to bring these old buildings in line with current performance requirements. Three typical structural-level retrofitting approaches, namely steel bracing, RC wall addition, and exoskeleton, are studied to enhance the residual design life of the existing buildings. It is observed that these three strategies can considerably decrease storey drift and significantly extend the residual life of the old building with proper planning. However, it is noted that the added retrofitting measures would increase the loads on adjacent beams, potentially affecting the residual life negatively. Among the three retrofitting approaches, the exoskeleton has a slightly lesser adverse influence on the existing components in the old building but requires more space around the building for installation. For a more comprehensive retrofitting, it is recommended to combine component-level retrofit strategies with structural-level ones to mitigate the adverse effects induced by structural-level retrofitting, enhance the durability of the structural components, and further extend the building's residual life. Meanwhile, the residual design life estimated by the framework could provide a more meaningful life span metric for the future life cycle analysis of the retrofitted buildings as well.

5. Appendix. Capacity calculations

In accordance with the specifications outlined in Chinese code GB 50010 [26], the capacities of beams are evaluated from the perspective of both bending moment and shear load. The bending and shear capacities of the beams are determined using the following equations:

$$M_{bu} = f_y A_{bs}(h_0 - \frac{f_y A_{bs}}{2 f_c b}) \tag{9}$$

$$V_{bu} = 0.7 f_t b h_0 + f_y \frac{A_{bsv}}{s} h_0 \tag{10}$$

where M_{bu} is the bending moment capacity of the beam for both positive moment and negative moment zone, V_{bu} is the shear capacity of the beam, A_{bs} is the area of tensile steel reinforcement in the beam, A_{bsv} is the stirrup area of the beam. h_0 stands for the effective height of the section, b is the section width, s is the spacing of the stirrup, f_c is equal to $f_{ck}/1.4$, f_t is equal to $f_{tk}/1.4$.

The evaluation of column capacities considers bending moment, shear load, and axial load. Bending moments on the columns are indirectly assessed by transferring them to eccentric distances relative to axial loads, and they are factored into the eccentric axial capacity of the column. For columns under compression, the eccentric axial capacity is calculated as follows:

$$\frac{1}{N_{cu}} = \frac{1}{N_{cux}} + \frac{1}{N_{cuy}} - \frac{1}{N_{cu0}} \tag{11}$$

where N_{cu} is the eccentric axial capacity of the column, N_{cux} and N_{cuy} are the axial capacities considering the moment or eccentricity in the X and Y directions, respectively, N_{cu0} is the axial capacity without considering the eccentric distance or moment and is calculated as:

$$N_{cu0} = f_c A + f_y A_{cs} \tag{12}$$

where A is the section area of the column, A_{cs} is the area of all reinforcement in the column.

N_{cux} can be obtained by solving the following equations and N_{cuy} could be evaluated similarly:

$$N_{cux} = 0.8 f_c bx + f_y A_{cs1} + \sigma_{s2} A_{cs2} + \sigma_{s3} A_{cs3} \tag{13}$$

$$N_{cux} e_x = 0.8 f_c bx \left(h_0 - \frac{0.8x}{2} \right) + f_y A_{cs1} (h_0 - a_s) + \sigma_{s2} A_{cs2} \left(h_0 - \frac{h}{2} \right) \tag{14}$$

$$e_x = e_0 + M_{cx}/N_c + h/2 - a_s \tag{15}$$

$$\sigma_{s2} = E_s \varepsilon_{cu} (x - h/2)/x \tag{16}$$

$$\sigma_{s3} = -E_s \varepsilon_{cu} (h_0 - x)/x \tag{17}$$

where x is the height of the compression region, A_{cs1}, A_{cs2}, A_{cs3} are the area of the steel reinforcement in the first, second and third rows from the compression side to the opposite side. σ_{s2} and σ_{s3} are the stresses of the steel reinforcement in the second and third rows, respectively, both below f_y. a_s is the centre distance of the reinforcement steel to the near edge of the section, e_0 is the initial eccentric distance (i.e., 20 mm, as specified by code GB 50010 [26]), M_{cx} and M_{cy} is the bending moment about X and Y axes on the column, and N_c is the axial load on the column.

For columns under tension, the eccentric axial capacity is calculated as follows:

$$\frac{1}{N_{cu}} = \frac{1}{N_{cu0}} + \sqrt{\left(\frac{M_{cx}/N_c + e_0}{M_{cux}} \right)^2 + \left(\frac{M_{cy}/N_c + e_0}{M_{cuy}} \right)^2} \tag{18}$$

$$N_{cu0} = f_y A_{cs} \tag{19}$$

$$M_{cux} = M_{cuy} = f_y A_{cs1} (h_0 - a_s) \tag{20}$$

The shearing capacities of the column along two directions are calculated using the following equations:

$$V_{cux} = V_{cux0} / \sqrt{1 + (\frac{V_{cux0}}{V_{cuy0}} \cdot \frac{V_{cy}}{V_{cx}})^2} \tag{21}$$

$$V_{cuy} = V_{cuy0} / \sqrt{1 + (\frac{V_{cuy0}}{V_{cux0}} \cdot \frac{V_{cx}}{V_{cy}})^2} \tag{22}$$

$$V_{cux0} = \frac{1.75}{M_{cx}/(V_{cx}h_0)+1} \cdot f_t b h_0 + f_y \frac{A_{csv}}{s} h_0 + \varphi N_c \tag{23}$$

$$V_{cuy0} = \frac{1.75}{M_{cy}/(V_{cy}b_0)+1} \cdot f_t b_0 h + f_y \frac{A_{csv}}{s} b_0 + \varphi N_c \tag{24}$$

where V_{cux} and V_{cuy} are the shear capacities along the X and Y directions, respectively. V_{cx} and V_{cy} are the shear loads along the X and Y directions, respectively. b_0 is the effective width. A_{csv} is the stirrup area of the column, φ is 0.07 for compression and -0.02 for tension.

Compliance with ethical standards

Conflict of interest: The authors declare they have no conflicts of interest.

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CRediT authorship contribution statement

Xingquan Mao: Supervision, Funding acquisition.

Baixi Chen: Conceptualization, Methodology, Investigation, Visualization, Formal analysis, Writing – original draft, Writing – review & editing.

Pak-wai Chan: Supervision, Funding acquisition.

You Dong: Supervision, Funding acquisition, Writing – review & editing.

Data availability

The data forming the basis of this study is available from the corresponding authors upon reasonable request.

Declaration of Generative AI and AI-assisted technologies in the writing process

During the preparation of this work, the authors used ChatGPT in order to improve the readability and language of this paper. After using this tool, the authors reviewed and edited the content as needed and take full responsibility for the content of the publication.

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