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Study on safety assessment method considering the deterministic characteristics of existing masonry structures

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Abstract. This paper presents a novel method to safety assessment, considering the deterministic characteristics of existing structures. Initially, the concept of disposable resistance is introduced, stemming from the deterministic attributes of the existing structure, leading to the formulation of the limit state expression. Subsequently, a rational value for the ratio of the permanent action effect to resistance is derived, serving as the foundation for optimizing the partial factors within the limit state expression. The rating principles for the safety assessment of existing structural members are established. Finally, a systematic study of safety assessment methods for existing masonry structures is conducted. The findings indicate that in evaluating the safety of existing masonry structures: (1) the resistance partial factor (γ_R) is set at 2.1; (2) the permanent action partial factor (γ_G) is assigned a value of 1.0; (3) for variable action factors (γ_Q), values of 1.3, 1.4, and 1.5 are applied, respectively, for subsequent working years of 30a, 40a, and 50a. The assessment method proposed in this paper is more in line with the reality of existing structures and can effectively avoid over-strengthening of structures.

Keywords: deterministic characteristics; existing structures; limit state expression partial factors; rating principle; structural safety assessment

1. Introduction

As society advances, China's construction sector is gradually transitioning from urban construction to urban renewal and the retrofitting of existing structures. Within this urban renewal process, the precise and rational evaluation of the safety of these structures has become a crucial

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component (Wu *et al.* 2023). However, upon employing the limit state design expressions and partial factors specified in current codes for assessing these existing structures, numerous cases reveal that many fail to comply with the current standards. This necessitates the implementation of strengthening and upgrading measures.

Despite this, practical projects have highlighted that evaluations based on current design codes often misalign with actual structural conditions, presenting significant obstacles to urban renewal efforts. The primary cause of this challenge lies in the decreasing applicability of the limit state design approach for assessing existing structures (Ellingwood 1996, Stepinac *et al*. 2020). Specifically, the limit state design expression and partial factors within the design code tend to underestimate the safety and load-bearing capacity of structural members, resulting in a conservative safety assessment and leading to unreasonable or excessive reinforcement requirements (Steenbergen *et al*. 2015, Sykor *et al.* 2017).

In contrast to the design of new structures, the existing structure is an objectively present entity with numerous predefined and quantifiable characteristics. For instance, the probabilistic characteristics of resistances and loads can be statistically gauged on-site. Additionally, the material attributes of the existing structure can be ascertained through an on-site inspection (Küttenbaum *et al*. 2021), while the historical loads can be verified through investigation. These data points enable the updating of the probabilistic resistance model via Bayesian theory (Faroz *et al*. 2016, Hackl *et al*. 2016, Croce *et al*. 2019). The probabilistic model pertaining to variable loads is subject to change as the subsequent years of operation progress, and it can be adjusted in accordance with the "principle of equal exceeding probability" (Gu *et al*. 2004, Huang *et al*. 2008). Furthermore, the limit state expressions and their partial factors for the assessment of existing structures require adaptation to accurately reflect the current conditions of such structures.

Given the inherent uncertainties of field-based material property testing for existing structures, pertinent domestic and international standards for identification and evaluation recommend the adaptation of resistance partial factors in accordance with on-site inspection (ISO 13822. 2010, GB 50292-2015. 2015, BSEN1998-3. 2005). Depending on the extent of completeness of the pre-existing structural data, the ASCE 41-17 incorporates the cognitive coefficient (K) to modify the load-bearing capacity of existing structural members. Similarly, the BS EN 1998-3 introduces the confidence factor (CF) to adjust the standard resistance value. It is evident that these two standards primarily emphasize the modification of the structural member resistance model, while overlooking the necessity for adjustment of the action partial factors.

From the above studies, it can be seen that most of the current methodologies regarding the assessment of existing structures still adopt the uncertainty characteristics of new building design, while ignoring some deterministic factors that actually exist in existing structures (e.g., the geometric characteristics of the structural members are deterministic and measurable, the gravity loads are constant, etc.). These assessment methods can lead to excessively conservative results, resulting in over-strengthening of existing structures and significant cost increases. Therefore, it is necessary to update the assessment method of existing structures considering the deterministic factors in existing structures. The updated assessment method will enable the assessment results of existing structures to be more in line with their actual conditions, avoiding over-strengthening and making existing structures sustainable.

In light of the aforementioned challenges in evaluating existing structures, this paper introduces a safety assessment methodology that considers the deterministic characteristics of existing structures. Subsequently, the safety evaluation of existing masonry structures is examined and investigated based on this novel assessment approach. The research outcomes are anticipated to augment the existing theory of structural safety assessments and enhance the method's applicability to engineering structures, thereby fostering the stable progression of urban renewal initiatives.

2. Theory and methodology

2.1 Deterministic characterization of existing structures

The resistance uncertainty of structural members is categorized into material properties uncertainty, geometrical characteristics uncertainty, and computational model uncertainty, represented by random variables Ω_f, Ω_a, and Ω_p, respectively (ISO 2394:2015, 2015, GB 50068-2001. 2001). The reduced uncertainty in the existing structures compared to the newly built structures suggests that there are many deterministic characteristics in the existing structures. However, there is no systematic description of these deterministic characteristics.

The existing structures inherently possess objective existence, and their structural resistances remain deterministic and constant. Despite the necessity for engineers to rely on a constrained number of samples to evaluate the mechanical properties of the members during structural assessment, the resulting uncertainty does not alter the inherent deterministic nature of the resistances exhibited by the existing structure and its components.

Adjusting the resistance and permanent action of an existing structure or member poses significant challenges, and enhancing its resistance often incurs substantial costs. Retrofitting existing structures has the potential to inflict damage, introduce additional uncertainties, and possibly reduce structural reliability. When the reliability of the existing structure falls short of the required standards, a safe, economical, and effective approach is to reduce the variable action on the structures.

Pursuant to the aforementioned, the deterministic characteristics of existing structures are described as follows:

(1) The resistance of the existing structural members is treated as deterministic. The resistance of structural members can only consider the computational modeling uncertainty.

(2) The variability of the permanent action effect is minimal and can thus be deemed constant.

(3) Adjustments to the variable action in the existing structure are easy and feasible; the structure's original load-bearing capacity should be fully exploited, and its safety and reliability can be achieved by modulating the variable action effect on the structure.

(4) The probability model of variable action necessitates adjustment in accordance with the varying subsequent working years.

2.2 Limit state expression for bearing capacity

The objective of a structural assessment is to aid the owner or engineer in judging the ability of a structure to fulfill its designated function during its service life. The process of evaluating an existing structure differs significantly from the methodology of designing a new structure. The principal criterion for the structural safety evaluation is to ensure that the resistance (R) surpasses or equals the anticipated action effect (S), which can be mathematically expressed as

$$R \geq S \tag{1}$$

The action effects S can be categorized into two distinct groups: permanent action effects S_G and variable action effects S_Q. In the context of safety assessments for existing structures, the actual disposable resistance of the existing member, denoted as DR, is specifically defined. $DR = R - S_G$, then Eq. (1) is expressed as Eq. (2).

$$DR \geq S_Q \tag{2}$$

In consideration of a permanent load and a variable load, Eq. (2) is expressed in terms of partial factors, as delineated in Eq. (3).

$$\frac{R_k}{\gamma_R} - \gamma_G S_{Gk} \geq \gamma_Q S_{Qk} \tag{3}$$

The physical significance of Eq. (3) and its parameters is related to the probabilistic modeling of the resistance and action of the existing structure. Specifically, the left-hand side of the inequality denotes the estimated disposable resistance capacity of the structural member, while the right-hand side represents the anticipated variable action effect. The symbols R_k and S_{Gk} represent the standard values of resistance and permanent load effect, respectively, taking into account the deterministic characteristics of the existing structure. Furthermore, S_{Qk} denotes the standard value of the variable action effect for various subsequent working years. The factors γ_R, γ_G, and γ_Q are the resistance partial factor, permanent load partial factor, and variable load partial factor, respectively. To account for varying structural safety levels, the structural importance factor γ_0 is introduced, leading to the transformation of Eq. (3) into the format of Eq. (4).

$$\frac{R_k}{\gamma_R} - \gamma_G S_{Gk} \geq \gamma_0 \gamma_Q S_{Qk} \tag{4}$$

2.3 Safety assessment rating principles

Pursuant to the GB50153-2008 (2008) "Unified standard for reliability design of engineering structures" , the target reliability indices are distinctly defined for various structural safety classes. Specifically, the index for structural safety class II stands at β_T, while for class I, it is incremented by 0.5 to $\beta_T + 0.5$, and for class III, it is decreased by 0.5 to β_T-0.5. Additionally, the "Standard for appraisal of reliability of civil buildings" GB 50292-2015 elaborates on the principles of structural safety assessment ratings, mapping them precisely to the reliability indices as tabulated in Table 1.

Within these assessment frameworks, the a_u level, characterized by $\beta \geq \beta_T$ (β is the reliability index of the structural member), indicates a satisfactory level of reliability, necessitating no immediate remedial action. However, as reliability degrades, reaching the b_u level ($\beta_T > \beta \geq \beta_T$ - 0.25), though not requiring immediate action, necessitates vigilant monitoring. The c_u level, signified by β_T -0.25> $\beta \geq \beta_T$ -0.5 , necessitates the implementation of remedial measures to enhance structural safety. Finally, the d_u level, where $\beta < \beta_T$ -0.5, represents a critical state, demanding prompt or immediate remedial action to ensure structural safety.

The general format of the limit state function g is

Table 1 Structural safety assessment rating principles

Rating principles	Reliability index	Means of implementation	Rating factor
a_u	$\beta \geq \beta_T$	do not have to take measures	$RF \geq 1.0$
b_u	$\beta_T > \beta \geq \beta_T$ -0.25	can not take measures	$1 > RF \geq a$
c_u	β_T -0.25> $\beta \geq \beta_T$ -0.5	should take measures	$a > RF \geq b$
d_u	$\beta < \beta_T$ -0.5	must take measures promptly or immediately	$RF < b$

$$g = R - S \leq 0 \tag{5}$$

The failure probability (P_f) of the limit state function can be calculated by Monte Carlo methods or by first-order moments method. The reliability index β can be considered as a function of the probability of failure (P_f):

$$\beta = \Phi^{-1}(P_f) \tag{6}$$

where Φ^{-1} = inverse standard normal distribution function.

In accordance with the well-established "Load Factor Classification" methodology (MBE-1. 2018), the assessment of structural member safety is precisely formulated by means of Equation (7). As shown in Table 1, when the rating factor (*RF*) is greater than or equal to 1.0, it is categorized as the a_u level, indicating a robust safety margin. If the *RF* falls within the range of 1 to *a* (exclusive of 1 and inclusive of *a*), it is deemed to be at the b_u level, suggesting *a* moderate safety status, can not take measures to enhance the structural safety. Further, when the *RF* lies between *a* and *b* (inclusive of *a* and exclusive of *b*), it corresponds to the c_u level, representing a lower yet acceptable safety level. Finally, if the *RF* is less than *b*, it is classified as the d_u level, indicating a need for urgent attention as the safety margin is inadequate. The specific numerical values for *a* and *b* are detailed in Table 1, which serves as a reference for determining the safety assessment level.

$$RF = \frac{R_k / \gamma_R - \gamma_G S_{Gk}}{\gamma_0 \gamma_Q S_{Qk}} \tag{7}$$

2.4 Method of determining the partial factors

Fig. 1 shows the process of determining the partial factors in the limit state expression, which includes the following main steps:

(1) Based on the types of structural members, it is imperative to select the appropriate ratio ρ_G, which represents the ratio of the standard value of permanent action effects (S_{Gk}) to the standard value of resistance (R_k).

(2) γ_0 can be taken as 1.0 for structural safety class II.

(3) For selected structural members, calculate the disposable resistance (DR_{kij}) of the members under an arbitrary set of resistance partial factors (γ_R) and permanent partial factors (γ_G) ; ($DR_{kij} = R_{kij} / \gamma_R - \gamma_G S_{Gkij}$, R_{kij} denotes the standard value of resistance at *j*th ρ_G for the *i*th type of member; S_{Gkij} denotes the standard value of permanent action effect at *j*th ρ_G for the *i*th type of member).

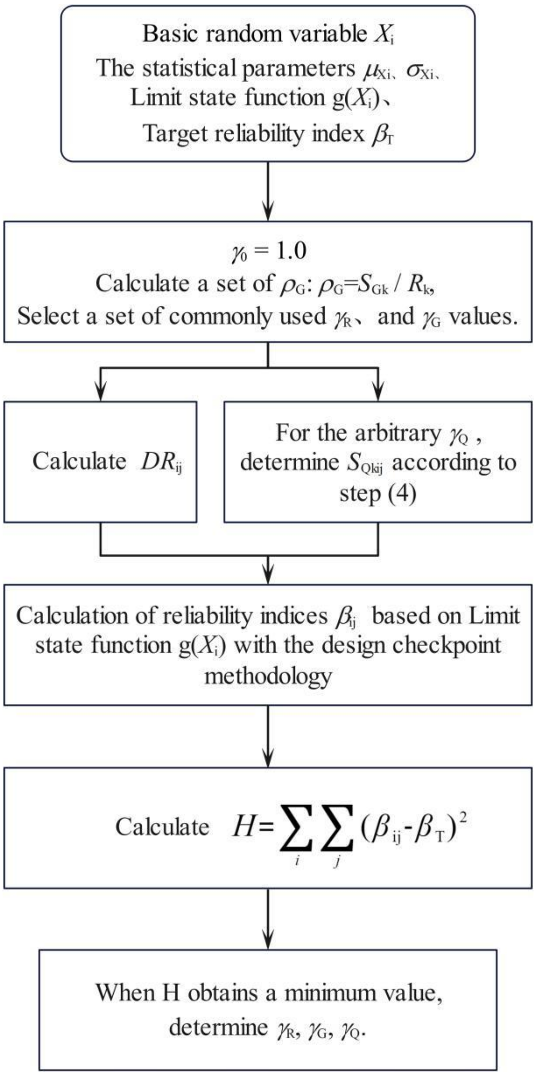


Fig. 1 The process of determining the partial factors

(4) For selected structural members, determine the standard value of variable action (S_{Qkij}) for any variable action partial factors (γ_Q); ($S_{Qkij} = DR_{kij} / \gamma_Q$)

(5) For selected structural members, calculate the reliability index (β_{ij}) at ρ_G; (β_{ij} denotes the reliability index at the *j*th value of ρ_G for the *i*th member type)

(6) For the selected structural member, select the partial factors that make the reliability index (β_{ij}) closest to the target reliability index (β_T) (i.e., it is satisfied that $H = \sum_i \sum_j (\beta_{ij} - \beta_T)^2$ obtains the minimum value) ;

(7) For structures with safety class I and III, follow the steps above to optimize the structural importance factor (γ_0).

3. Safety assessment of existing masonry structure

3.1 ρ_G of existing masonry structures

For reliability calibration of the current masonry design code, the ratio of the standard value of variable loads to the standard value of permanent loads ρ is generally taken as 0.1, 0.25, and 0.5, with $\rho \leq 0.375$ for permanent load control and $\rho > 0.375$ for variable load control (GB 50003-2001. 2001). The limit state design expression for a masonry structure when considering one variable load and one permanent load is shown in Eq. (8).

$$\begin{cases} \dfrac{R_{\mathrm{k}}}{\gamma_{\mathrm{R}}} = 1.2S_{\mathrm{Gk}} + 1.4S_{\mathrm{Qk}} & \rho > 0.375 \\ \dfrac{R_{\mathrm{k}}}{\gamma_{\mathrm{R}}} = 1.35S_{\mathrm{Gk}} + 1.0S_{\mathrm{Qk}} & \rho \leq 0.375 \end{cases} \tag{8}$$

Where γ_{R} = 2.1 (under compression) , γ_{R} = 2.25 (under shear) for structural safety class II.

Based on Eq. (8), the ratio ρ_G, representing the ratio of the standard value of permanent action effect to the standard value of resistance, is derived as 0.23, 0.28, 0.31 (for shear members) and 0.25, 0.30, 0.33 (for compression members). Notably, the values of ρ_G for compression and shear members are largely analogous. In the present section, the values ρ_G = 0.25, 0.30, and 0.33 were employed while conducting a reliability analysis of existing masonry structures.

3.2 Partial factors and rating principle for safety assessment of existing masonry structures

3.2.1 Probabilistic model of resistance and loads

The resistances' uncertainty characteristics during structural design are exhibited in Table 2. Non-destructive testing (NDT) serves as the primary means for inspecting existing structures, enabling the acquisition of structural members' material properties and geometric characteristics through onsite tests. Pursuant to the elaboration of the deterministic characteristics of existing structures in section 2.1, the sole consideration in conducting the reliability analysis of these structures is the computational model uncertainty, denoted as Ω_P. And, the resistance probability model adheres to a log-normal distribution (GBJ 50068-2018. 2018).

The statistical characteristics of the variable and permanent loads during structural design for different subsequent working years are shown in Table 3. Since the permanent load variability of the structural design is small and relatively constant over subsequent working years, it can be considered deterministic in the existing structure assessment, i.e., κ_{SG} = 1.0, δ_{SG} = 0.0. According to the principle of "equal exceeding probability", the probability distribution model of variable loads in the subsequent working years is shown in Eq. (9)

$$F_{\mathrm{Q}_{T'}}(x) = [F_{\mathrm{Q}_i}(x)]^{\frac{T'}{T}\mathrm{m}} \tag{9}$$

Where m is the average number of occurrences of load maxima, T' denotes the subsequent working years, T is the design working years, $F_{Qi}(x)$ is the probability distribution model for any time point following an Gumbel distribution (GBJ 50068-2018. 2018).

According to Eq. (9) and GB 50009-2012 Code for structural loading of buildings (GB 50009-

Table 2 Statistical parameters of resistance uncertainty for the design of brick masonry members (GBJ 50068-2018. 2018)

	types	κ	δ		types	κ	δ
Ω_f	Axial compression	1.00	0.174	Ω_P	Axial compression	1.0922	0.2059
	Eccentric compression	1.00	0.174		Eccentric compression	1.1814	0.2159
	Shear	1.00	0.240		Shear	1.017	0.1260
Ω_a	Axial compression	1.00	0.023	R	Axial compression	1.0922	0.2705
	Eccentric compression	1.00	0.023		Eccentric compression	1.1814	0.2811
	Shear	1.00	0.036		Shear	1.0170	0.2734

Notes : $\kappa = \mu_X / X_k$, μ_X is the mean of the variable X , X_k is the standard value of variable X ; δ_x a is the coefficient of variation of the variable X。

Table 3 Load statistical parameters with different subsequent working years

Subsequent working years	S_G		S_Q					
	Dead loads		Residential live loads		Office live loads		Wind	
	κ	δ	κ	δ	κ	δ	κ	δ
30a	1.06	0.07	0.594	0.252	0.475	0.318	0.822	0.210
40a			0.622	0.241	0.502	0.300	0.928	0.201
50a			0.644	0.233	0.524	0.288	1.000	0.194

Notes : $\kappa = \mu_X / X_k$, μ_X is the mean of the variable X , X_k is the standard value of variable X ; δ_x a is the coefficient of variation of the variable X

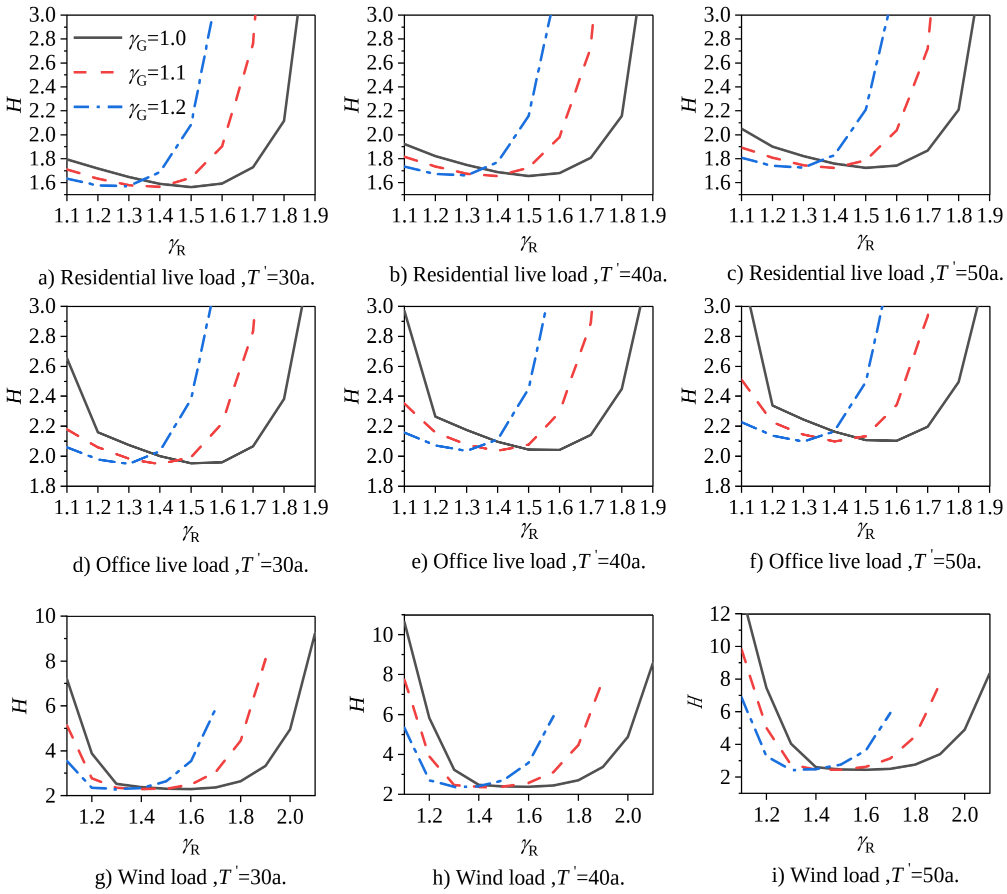
2012. 2012), the probability distribution characteristics of variable loads with subsequent working life of 30, 40 and 50 years are obtained, as shown in Table 3. And, the variable loads probability distribution model follows a Gumbel distribution.

3.2.2 Partial factors and rating principle for the existing masonry structures assessment

This paper adopts the values of 1.0, 1.1, and 1.2 for the permanent action partial factors (γ_G), and 1.1, 1.2, 1.3, 1.4, 1.5, 1.6, 1.7, 1.8, 1.9, 2.0, and 2.1 for the resistance partial factors (γ_R). The combination of these partial factors yields 33 distinct scenarios. The target reliability index β_T is set at 3.7, as referenced in GB 50068-2001.

In terms of variable loads, three distinct types are examined in this paper: office live load, residential live load, and wind load. Additionally, three-member types are considered: axial compression members, eccentric compression members, and shear members, as detailed in Tables 2 and 3. Recognizing that the resistance of a structural member may deteriorate over time, this paper postulates that once the masonry structure reaches its subsequent working life, its resistance will be reduced to 70% of its initial resistance, ensuring a higher level of safety for the partial factors.

Based on the process outlined in section 2.4 for determining partial factors, the $\gamma_R - \gamma_G - H$ curves of existing masonry structural members across various subsequent working years are



a) Residential live load ,T'=30a. b) Residential live load ,T'=40a. c) Residential live load ,T'=50a.

d) Office live load ,T'=30a. e) Office live load ,T'=40a. f) Office live load ,T'=50a.

g) Wind load ,T'=30a. h) Wind load ,T'=40a. i) Wind load ,T'=50a.

Fig. 2 $\gamma_R-\gamma_G-H$ curves of the existing masonry structural members

depicted in Fig. 2. It is evident that for varying loads, there exists a specific set of γ_R and γ_G values that minimize H. Given that the variability of the permanent action in the existing structure is not addressed in this study, it is advisable to consider the combination of partial factors at γ_G = 1.0. Under this scenario, Table 4 presents the optimal combination of partial factors for the safety assessment of existing masonry members subject to diverse primary variable loading conditions.

Adopting partial factors tailored specifically for the most unfavorable working conditions ensures both safety and practicality. Consequently, it is advisable to consider partial factors applicable under wind load conditions. This paper proposes the adoption of the following partial factors for the subsequent working years of 30a, 40a, and 50a:

(1) the resistance partial factor γ_R is taken as 1.6;

(2) the permanent action partial factor γ_G is taken as 1.0;

(3) and for the variable action coefficients, γ_Q are taken as 1.3、1.4 and 1.5, respectively, for the subsequent working years of 30a, 40a and 50a .

Table 4 The optimal combination of partial factors under different dominant variable loading conditions

dominant variable load	Subsequent working years	γ_R	γ_G	γ_Q	H
Residential live loads	30a	1.50	1.00	1.02	1.56
	40a	1.50	1.00	1.05	1.66
	50a	1.50	1.00	1.08	1.72
Office live loads	30a	1.50	1.00	1.15	1.95
	40a	1.50	1.00	1.18	2.04
	50a	1.50	1.00	1.21	2.11
Wind	30a	1.60	1.00	1.25	2.29
	40a	1.60	1.00	1.39	2.37
	50a	1.60	1.00	1.48	2.44

Table 5 The importance factors and boundary values

		Target reliability index	dominant variable load	Subsequent working years			Mean value
				50a	40a	30a	
importance factors γ_0	structural safety level I	β_T+0.5	Residential live loads	1.08	1.04	1.10	1.2
			Office live loads	1.52	1.14	1.20	
			Wind	1.15	1.54	1.52	
	structural safety level II	β_T	Residential live loads	0.79	0.83	0.88	1.0
			Office live loads	0.87	0.92	0.97	
			Wind	1.23	1.25	1.22	
	structural safety level III	β_T-0.5	Residential live loads	0.64	0.67	0.70	0.8
			Office live loads	0.70	0.74	0.78	
			Wind	1.01	1.02	0.99	
boundary value	a	β_T-0.25	Residential live loads	0.71	0.75	0.79	0.9
			Office live loads	0.78	0.83	0.87	
			Wind	1.12	1.13	1.10	
	b	β_T-0.5	Residential live loads	0.64	0.67	0.70	0.8
			Office live loads	0.70	0.74	0.78	
			Wind	1.01	1.02	0.99	

The target reliability index, considering the aforementioned partial factors and probabilistic models of resistance and action outlined in section 3.2.1, stands at approximately 4.2, denoted as $\beta_T = 4.2$. Consequently, for the determination of the importance factor at the structural safety level I, the target reliability index can be adopted as $[\beta] = \beta_T + 0.5 = 4.7$. Similarly, for the structural safety level III, the target reliability index is $[\beta] = \beta_T - 0.5 = 3.7$. For calculating the boundary value a, the target reliability index is set at $[\beta]=\beta_T - 0.25 = 3.95$, and for boundary value b, it is $[\beta] = \beta_c - 0.5 = 3.7$. Following the process detailed in section 2.4 for determining partial factors, the importance factors and boundary values can be derived, as presented in Table 5.

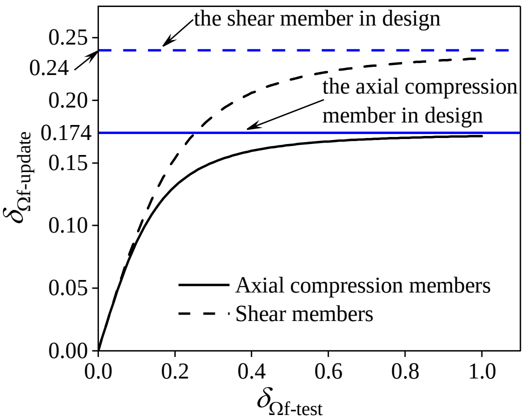


Fig. 3 $\delta_{\Omega\text{f-update}}$ — $\delta_{\Omega\text{f-test}}$ curve

The important factors can be averaged across various loading conditions and subsequent working years to accurately reflect the overall variability in structural reliability across different safety levels. When conducting a safety assessment for individual members, the boundary value should be set as the average in the most adverse operating conditions to ensure a safer outcome. Consequently, γ_0 can be assigned a value of 1.2 for safety level I, 1.0 for safety level II, and 0.8 for safety level III. Regarding the principles for safety assessment ratings, the rating factor (RF) ≥ 1.0 corresponds to the a_u grade, 1 > RF ≥ 0.9 corresponds to the bu grade, 0.9 > RF ≥ 0.8 corresponds to the c_u grade, and RF < 0.8 corresponds to the d_u grade.

3.2.3 Resistance partial factors considering uncertainty in on-site testing

Considering the unavoidable uncertainty due to on-site inspection, the resistance partial factors can be further optimized. Based on the on-site inspection, the uncertainty characteristics of material properties can be updated with the Bayesian Eq. (10).

$$\pi(\theta \mid x) = \frac{\pi(\theta)p(x \mid \theta)}{\int_{-\infty}^{+\infty} \pi(\theta)p(x \mid \theta)d\theta} \tag{10}$$

where $\pi(\theta)$ is the a priori probability density function; $p(x|\theta)$ is the conditional probability density function commonly referred to as the likelihood function.

Fig. 3 illustrates the variation rule of $\delta_{\Omega\text{f-update}}$ relative to

$\delta_{\Omega\text{f-test}}$, where $\delta_{\Omega\text{f-update}}$ represents the updated variation coefficient of material property uncertainty, whereas $\delta_{\Omega\text{f-test}}$ denotes the variation coefficient for material property uncertainty obtained during on-site inspections. Observably, the variation coefficients revised via Bayesian theory consistently undercut those based on the original probability distribution, implying a reduction in the material property uncertainty of existing masonry structures after Bayesian updating. Specifically, the $\delta_{\Omega\text{f}}$ value for new structure design serves as the ceiling for the $\delta_{\Omega\text{f-update}}$ value during the assessment of existing structures. Consequently, when executing the partial factor design for evaluating existing structures, the ceiling of the resistance partial factor is attained when accounting for both the computational model and material property uncertainty.

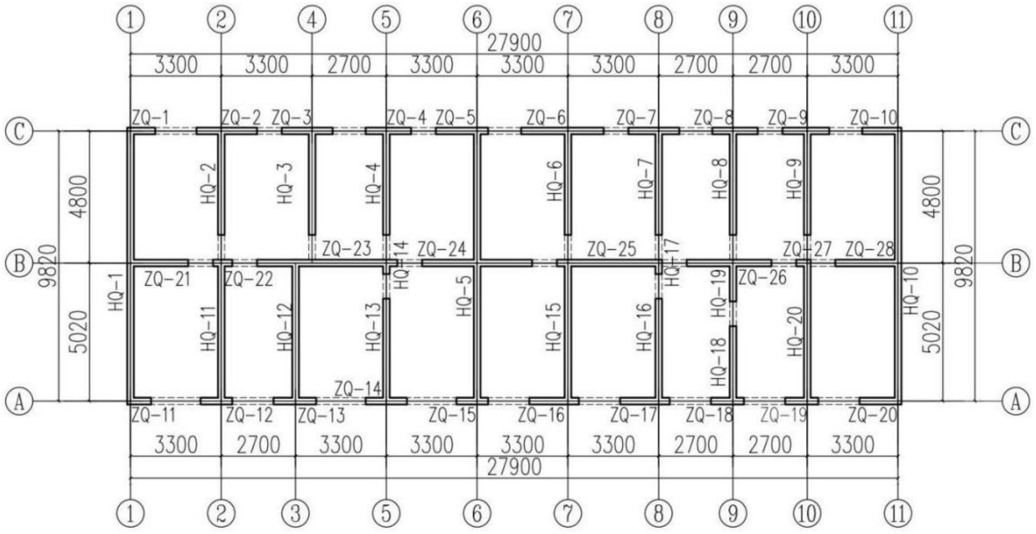


Fig. 4 First floor structural layout plan

In this section, $\kappa_{\mathrm{R}} = \kappa_{\Omega\mathrm{f}} \cdot \kappa_{\Omega\mathrm{p}}$, $\delta_{\mathrm{R}} = \sqrt{\delta_{\Omega\mathrm{f}}^2 + \delta_{\Omega\mathrm{f}}^2}$, the values of $\kappa_{\Omega\mathrm{p}}$, $\kappa_{\Omega\mathrm{f}}$, $\delta_{\Omega\mathrm{f}}$ and $\delta_{\Omega\mathrm{p}}$ see Table 2. The values of γ_{G} is taken as 1.0 while the value of γ_{Q} are taken as 1.3、1.4 and 1.5, respectively, for the subsequent working years of 30a, 40a and 50a . The resistance follows a log-normal distribution while the variable action adheres to a Gumbel distribution. Following the method outlined in section 2.4 for determining partial factors, the value of the resistance partial factor γ_{R} can be taken as 2.1.

4. Engineering case studies

A 5-story masonry residential house with a subsequent work life of 30 years. After reviewing previous drawings and on-site inspection, the following information was obtained:

(1) Structural wall thickness is 240 mm, masonry mortar strength grade is M1.0, and brick strength grade is MU10.

(2) Floor dead load (including self-weight) is 4.0 kN/m^2, floor live load is 2.0 kN/m^2, stairwell dead load (including self-weight) is 6.0 kN/m^2, stairwell live load is 3.5 kN/m^2, roof dead load (including self-weight) is 5.0 kN/m^2, roof live load is 0.5 kN/m^2, masonry volumetric weight is 20 kN/m^3.

(3) The first-floor plan of the structure is shown in Fig. 4.

Fig. 4 illustrates the distribution of 48 compressed members on the first floor, 28 in the longitudinal wall labeled as ZQ-1 to ZQ-28, and 20 compressed members in the transverse wall labeled as HQ-1 to HQ-20. In accordance with the regulations outlined in the "Standard for Appraisal of Reliability of Civil Buildings" (GB 50292-2015. 2015), the safety grading coefficients S_{D} for these members can be calculated by Eq. (11).

Table 6 Wall segment safety assessment results

Wall number	ϕ	f(MPa)	A(m^2)	R_k(kN)	S_{Gk}(kN)	S_{Qk}(kN)	S_{D1}	Result with S_D	RF	Result with RF
HQ-2	0.72	1.18	0.9072	1618.59	535.85	172.16	0.81	d_u	1.05	a_u
HQ-3	0.81	1.18	0.9072	1820.91	569.41	168.29	0.81	d_u	1.36	a_u
HQ-4	0.72	1.18	0.9100	1623.59	541.71	168.29	0.81	d_u	1.06	a_u
HQ-5	0.72	1.18	2.3568	4204.91	1346.12	399.67	0.85	d_u	1.26	a_u
HQ-6	0.72	1.18	0.9072	1618.59	535.84	172.16	0.81	d_u	1.05	a_u
HQ-7	0.72	1.18	0.9072	1618.59	521.95	168.29	0.83	d_u	1.14	a_u
HQ-8	0.72	1.18	0.9072	1618.59	500.90	153.30	0.88	d_u	1.35	a_u
HQ-9	0.72	1.18	0.9072	1618.59	514.82	156.51	0.85	d_u	1.26	a_u
HQ-11	0.72	1.18	1.2336	2200.94	735.34	208.94	0.82	d_u	1.15	a_u
HQ-12	0.71	1.18	1.2048	2119.70	663.14	185.74	0.88	d_u	1.43	a_u
HQ-13	0.71	1.18	0.8928	1570.77	530.14	169.72	0.79	d_u	0.99	b_u
HQ-14	0.72	1.18	0.1248	222.66	181.06	57.27	0.27	d_u	-1.01	$\mathbf{d_u}$
HQ-15	0.72	1.18	1.2336	1678.68	760.31	227.51	0.78	d_u	0.97	b_u
HQ-16	0.71	1.18	0.8928	1198.05	509.41	154.29	0.84	d_u	1.19	a_u
HQ-17	0.72	1.18	0.1248	169.83	174.17	54.13	0.28	d_u	-0.97	$\mathbf{d_u}$
HQ-18	0.71	1.18	0.6528	875.99	371.48	105.56	0.85	d_u	1.28	a_u
HQ-19	0.72	1.18	0.3648	496.42	284.29	82.17	0.62	d_u	0.24	$\mathbf{d_u}$
HQ-20	0.72	1.18	1.2336	1,678.68	732.51	206.83	0.82	d_u	1.17	a_u

$$S_D = \frac{R}{\gamma_0 S} = \frac{\phi f A}{\gamma_0 (1.3 S_{\mathrm{Gk}} + 1.5 S_{\mathrm{Qk}})} \tag{11}$$

where ϕ is the pressure influence coefficient; f is the design value of compressive strength; A is the cross-sectional area of the wall section; the importance coefficient γ_0 is 1.0.

When evaluating the bearing capacity of compressed members using Eq. (11), the masonry members' safety grades are clearly outlined in Table 6, revealing that the proportion of d_u grade members stands at 39%. Notably, the first floor has been comprehensively classified as D_u grade, and the masonry house itself has been assessed as D_{su} grade, both indications pointing towards the urgent need for strengthening measures within the house.

Moreover, when undertaking the load capacity assessment of compression members, utilizing the partial factors proposed in this paper, the safety rating factors (*RF*) of the respective members can be precisely calculated through the application of Eq. (12).

$$RF = \frac{R_{\mathrm{k}} / \gamma_{\mathrm{R}} - \gamma_{\mathrm{G}} S_{\mathrm{Gk}}}{\gamma_0 \gamma_{\mathrm{Q}} S_{\mathrm{Qk}}} = \frac{R_{\mathrm{k}} / 2.1 - 1.0 S_{\mathrm{Gk}}}{1.0 \times 1.3 S_{\mathrm{Qk}}} \tag{12}$$

where R_k is the standard value of the resistance of the compressed wall section, R_k=2.1$\phi f A$; according to the study in this paper, the resistance partial factor γ_R is 2.1.

When conducting the load-bearing capacity assessment of compression members with Eq. (12), the safety grades of masonry members are shown in Table 6. There are two d_u grade members (HQ-14, HQ-17 and HQ-19) ; the first floor of the d_u grade members accounted for 5% of the total, which is less than the requirement of 10%, the first floor of the comprehensive evaluation of the C_u grade, and the masonry house of this masonry house comprehensively evaluated as the C_{su} grade (GB 50292-2015. 2015).

The evaluation results, based on the current appraisal standard (GB 50292-2015), indicate that a significant number of 19 masonry members, encompassing all internal load-bearing transverse walls, necessitate strengthening. This poses not only a considerable workload but also challenges in facilitating the practical implementation of the work. However, the assessment method presented in this paper reveals that only three members require strengthening in the first floor. The contrast between these two assessment outcomes highlights that the methodology that takes into account the deterministic characteristics of the existing structure is more aligned with the actual condition of such structures. It mitigates damage to the original structure due to excessive strengthening and aligns with the principles of "Green Strengthening and Reconstruction, Reducing Carbon Emission" and the sustainable development imperatives of urban renewal.

5. Conclusions

The existing structures are characterized by numerous deterministic attributes. By maximizing the utilization of the original structure's inherent resistance, the owner can efficiently circumvent over-strengthening. Consequently, a safety assessment methodology tailored to account for these deterministic characteristics of existing structures has been devised. Adhering to this new safety assessment framework, the following regulations ought to be adhered to while conducting a safety assessment of existing masonry structures:

(1) The resistance partial factor (γ_R) is set at 2.1, while the permanent action partial factor (γ_G) is assigned a value of 1.0. For variable action partial factors (γ_Q), the values of 1.3, 1.4, and 1.5 are assigned respectively for the subsequent working years of 30a, 40a, and 50a.

(2) Depending on the structural safety levels, the importance factor (γ_0) is assigned as follows: 1.2 for level I, 1.0 for level II, and 0.8 for level III.

(3) With regard to safety assessment grading principles, the following ranges apply: $RF \geq 1.0$ for a_u grade, $1 > RF \geq 0.9$ for b_u grade, $0.9 > RF \geq 0.8$ for c_u grade, and RF < 0.8 for d_u grade.

(4) This assessment method is applicable to the safety assessment of existing masonry houses under normal use and operation. For the assessment of existing structures under complex conditions and environments, it is necessary to make a comprehensive judgment in combination with exterior quality inspection and seismic measures investigation.

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