Reliability Analysis of Safety Assessment of Buildings above Twin Shield Tunnels Based on Random Field Theory

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1. Introduction

Urban underground tunnels often pass through old brick-concrete buildings with a shallow foundation, which are extremely sensitive to stratum deformation. Tunnel construction inevitably causes stratum movement, and large deformations will easily damage such buildings, ranging from cracks to dangerous buildings. Therefore, while ensuring the tunnel’s safety, real-time analysis and assessment of the risk status of adjacent buildings are also critical in the engineering construction process.

The change of deformation performance of adjacent buildings caused by tunnel construction disturbance is a complex issue. Domestic and international scholars have carried out much research work using theoretical analysis, numerical simulation, and field measurements. Mair et al. [1] proposed a risk assessment method of building damage for tunnel excavation by combining the research on the classification of masonry building damage with the calculation of the ultimate tensile strains in buildings due to ground deformation. Haji et al. [2] proposed a mixed empirical-numerical two-stage method to estimate the effect of tunnel construction on the response of surface buildings. Peduto et al. [3] collected the damage survey data from over 700 masonry buildings in four cities in the Netherlands. They proposed a multiparameter probabilistic analysis...
method for damage analysis of masonry buildings with different foundation types. Based on many monitoring data on building deformation, Wu [4] studied the relationship between building settlement, differential settlement, and damage level, inclination (angular variable), and discussed the relationship between building size and deformation, damage level. Regarding assessing the risk status and damage level of buildings, Ge et al. [5] put forward the double control index considering the total amount and increment of building deformation during shield tunneling. Yiu et al. [6] proposed a new method to quantify the degree of damage to buildings caused by tunneling using the characteristic strain of masonry structures. Franza et al. [7] presented a design diagram for the limit radius of the settlement curve associated with soil-beam separation based on a two-stage Euler-Bernoulli beam model of a linear elastic continuum. Burland et al. [8] proposed the classification criteria of the damage level of masonry buildings based on the ultimate tensile strain, which is widely used in design and construction. It is considered that when the ultimate tensile strain of general masonry buildings is less than 0.15%, it will not lead to the loss of building functions, and when it reaches 0.15–0.30%, reinforcement measures should be taken.

In general, the current studies on deformation performance analysis and damage assessment of buildings caused by tunnel construction have mainly focused on the effect of excavation disturbance of the single-line tunnel on building safety and seldom involved double-line tunnels. In fact, most existing urban underground tunnels are designed with double lines. A study by Ma et al. [9] showed that different construction sequences and different configurations of twin tunnels have a significant effect on stratum deformation and structural bending strain. Similarly, different working conditions of twin tunnels also have a different effect on building deformation. Therefore, studying the effect of twin tunnel configurations and construction sequences on building safety is necessary. Furthermore, due to complex geological effects, there is significant spatial variability of the mechanical parameters of natural rock and soil in nature [10]. Relevant studies [11, 12] have shown the considerable role of spatial variability of soil parameters in the mechanical responses of strata and structures and the environmental effects caused by tunnel construction. Cheng et al. [13] and Luo et al. [14] assessed the effects of the spatial variability of soil parameters on the safety of buildings, stating that ignoring the spatial variability of the soil parameters can underestimate the damage probability of the structure, which will lead to unsafe engineering designs.

Given this, based on previous studies, this paper distinguishes different ground surface deformation modes, further improves the safety assessment system of buildings under crossed by tunnels, and proposes the safety assessment analysis method of buildings above twin tunnels based on random field theory. It is applied to the safety assessment analysis of adjacent masonry buildings under crossed by twin shield tunnels, considering the influencing factors such as the configurations and construction sequences of twin tunnels and the vertical and transverse scales of fluctuation of soil elastic modulus $E$. The development and change law of the maximum tensile strain in upper masonry buildings caused by the construction of twin shield tunnels are studied systematically. The probabilistic analysis of the building safety is carried out referring to the assessment criteria of the ultimate tensile strain. On this basis, the effect of building size on its safety is explored, and an assessment analysis of potential damage zones of buildings is carried out.

2. Random Field Theory

Geotechnical parameters are characterized by local stochasticity and overall structure, that is, the duality of spatial variability. Vanmarcke [15] introduced the random field theory and constructed a random field model to describe the spatial variability of soil parameters. The random field theory regards the geotechnical parameter at any point as a random variable that approximately obeys a specific probability distribution and characterizes the spatial structure of the parameter through spatial concepts such as the scale of fluctuation and the autocorrelation structure. Based on the central limit theorem, the lognormal distribution can be regarded as the form of the limit distribution multiplied by a large number of uncertainties, which coincides with the formation process of rock and soil mass [16, 17]. The lognormal distribution is strictly nonnegative, which is more in line with the statistical characteristics of the parameter of soil properties. Therefore, the lognormal distribution is used in this paper to describe the uncertainty of the soil elastic modulus. The logarithmic modulus field satisfies the anisotropic exponential autocorrelation function [17, 18], which can be expressed as

$$\rho_{lnE}(\tau_x, \tau_z) = \exp\left(-\frac{2\tau_x}{\theta_x} - \frac{2\tau_z}{\theta_z}\right),$$

where $\rho_{lnE}(\tau_x, \tau_z)$ represents the autocorrelation coefficient of two points in the logarithmic modulus field, whose value indicates the strength of the correlation between two points. The higher the value, the stronger the correlation, and $0 \leq \rho_{lnE}(\tau_x, \tau_z) \leq 1$. $\tau_x$ and $\tau_z$ are horizontal and vertical distances, respectively; $\theta_x$ and $\theta_z$ are the transverse and vertical scales of fluctuation of the logarithmic modulus field. The results of the relevant studies [19] show that the transverse scale of fluctuation is generally from 10 to 80 m, while the vertical scale of fluctuation is from 1 to 3 m.

3. Safety Assessment Method of Masonry Buildings above Twin Shield Tunnels Based on Random Field Theory

3.1. Numerical Calculation Model. The configurations of twin shield tunnels can be roughly divided into three types: parallel, overlapping, and shoulder-mounted. The twin tunnel construction problem is simplified into a two-dimensional plane strain model, and FLAC3D finite difference software is used to simulate the sequential construction of twin tunnels. The excavation and support process of the shield tunnels are simulated using the stress release method, and the instantaneous stress release rate of excavation is taken as 50% [20]. The numerical calculation models for three different configurations (parallel, overlapping, and
shoulder-mounted) of twin shield tunnels are given in Figure 1. Table 1 shows the analytical working conditions for the different configurations.

The dimension of the model for numerical calculations in all three configurations is 90 m x 40 m (width x height), with a tunnel diameter of $D = 6.2$ m and a maximum mesh size of approximately 1.0 m. The ground surface is the free boundary, and all other boundaries impose normal constraints. The soil is an ideal elastic-plastic mass obeying the M-C yield criterion, and the lining structure is simulated using shell units. Table 2 shows the physical and mechanical parameters of the soil and the lining structure materials. The elastic modulus of the lower layer of the model is set to three times that of the upper layer, to consider the soil’s loading and unloading characteristics; that is, the unloading stiffness of the soil is greater than the loading stiffness. Its simulation results are more in line with reality [20].

3.2. Analysis of the Surface Deformation Mode. According to relevant studies [21], the surface deformation modes caused by the construction of twin tunnels can be roughly divided into single peak and double peak types. Figure 2 shows the surface settlement curves obtained from the deterministic analysis for different configurations and different
construction sequences of twin tunnels. Note that "up" and "down" represent the condition of the overlying and the underlying tunnel being excavated first, respectively; other legends are similar. As shown in Figure 2(a), the final surface settlement curve due to excavation is a single peak when twin tunnels are arranged in parallel, and the axis spacing is small; the double peak characteristics of the curve tend to become significant as the spacing increases. It can be seen from Figure 2(b) that the surface deformation curve caused by the excavation of overlapping and shoulder-mounted twin tunnels has a single peak type. Furthermore, considering the effect of the spatial variability of the parameters on surface deformation during tunnel construction, the settlement curves obtained can vary for different stochastic calculation orders under the same working conditions [17], as shown in Figure 3 for details. The surface deformation curves of different modes differ in their contraflexure points, deflection ratios, and other characteristics, with single peak Gaussian curves generally having two contraflexure points and double peak curves more often having four contraflexure points.

3.3. Building Safety Assessment Methods considering the Surface Deformation Mode. Drawing on the building safety assessment method adopted in the Jubilee Extension Project of London Metro [22], the masonry buildings are regarded as homogeneous weightless elastic foundation beams. The buildings are zoned according to the locations of the contraflexure points of the surface settlement curve. The locations of the contraflexure points vary for different modes of deformation curves. Therefore, before carrying out the building safety assessment analysis, a self-programmed contraflexure point search procedure was compiled for different modes of deformation curves to effectively determine the locations of the contraflexure points of the settlement curves.

It is considered that the inflection points of the curve are the requested contraflexure points, starting from the definition of the contraflexure point of the settlement curve and the curvature of the curve. At the same time, referring to the results of the stochastic calculation given in Figure 3, the problem is simplified by assuming that, compared to the deterministic results, the surface deformation curve obtained from the stochastic analysis only differs in the location of the contraflexure points and there is no change in the number of contraflexure points. The critical steps in determining the exact location of the contraflexure point (i.e., the inflection point) of the surface settlement curve obtained from each stochastic analysis with the help of Matlab are as follows:

(1) Calculation of the Slope Value at Each Node. The settlement value and the x coordinate of each node on the surface can be extracted in any stochastic analysis, and the forward difference can calculate the
slope value at that node location. The slope value at the back point is subtracted by a small value (e.g., 0.0001) for cases where the slope values at the front and back points are equal to facilitate subsequent analysis of the slope extremum (inflection point).

(2) Search for the Exact Locations of the Contraflexure Points. An analysis of extreme values is performed on the slope value obtained in Step (1) to determine the location of the inflection point. However, the results may not be satisfactory; therefore, the extreme value points of the slope are selected according to the study of the relevant literature [23, 24] and engineering experience; (i) - (iv) are the setting conditions for the screening of the extreme value points of the slope.

After screening the above conditions, there are three conditional statements (if), of which the second and third conditional statements are exemplified. (i) The contraflexure point is not one of the 10 points on the model's left or right boundary range. (ii) In the parallel arrangement of twin tunnels, the location of the contraflexure point should not exceed a range of 15 m on either side of the tunnel axis (in shoulder-mounted and overlapping twin tunnels, the range value could be relaxed to 20 to 25 m). (iii) The slope value at the contraflexure point should not be less than (or not be greater than) the slope value at the several nearby points (10 points on the left and 10 points on the right). (iv) There are a maximum and minimum value of slope for the single peak curve and two maximum and minimum values for the double peak curve. After screening the above conditions, if there is no corresponding extreme value point, the contraflexure point of the deterministic settlement curve will be regarded as the contraflexure point of the curve obtained from this stochastic analysis; if it is less than the required number of extreme value points, it will be filled with the deterministic result. For example, there are two maximum value points in double peak type, and only one satisfies the conditions after screening. Then, the extreme maximum value point in the deterministic results, which is farther away from previous selected point, is selected as another extreme maximum value for this stochastic analysis; if the number of extreme value points is greater than the required number, then one needs to be eliminated. For example, there are still three extreme maximum values after the screening in a double peak curve, which are in order of No. 1, No. 2, and No. 3. According to the distance between points No. 1, No. 3, and No. 2, the point with the more considerable distance is selected as one of the contraflexure points, and the other is chosen from the extreme value of the remaining two; that is, assuming that No. 1 and No. 2 are closer to each other, then point No. 3 is the second maximum point (contraflexure point) of the curve obtained from this stochastic analysis. Combined with the slope values of points No. 1 and No. 2 and their nearby points (taking a point on the left and right to sum up), the extreme maximum value of both is selected as the first extreme value point (contraflexure point) required.

The 1 mm settlement line is taken as the effect boundary [1] of surface settlement on buildings (shown as a dashed line in Figure 4). Assuming that the height of the flexible simply supported beam model is $H$, the length is $L$, and $l$ is greater than the effect range of surface settlement, the aforementioned contraflexure point location searching method can determine the specific positions of the contraflexure points of the curves obtained from each stochastic analysis. According to the positions of the contraflexure points of the surface settlement curve, the building is divided into hogged and sagged sections, as shown in Figure 4. It can be seen from the figure that, in the single peak settlement mode, the building can be divided into three sections: hogged section 1 ($l_1$), sagged section 1 ($l_2$), and hogged section 2 ($l_3$). In the double peak settlement deformation, the building can be divided into five parts: hogged section 1 ($l_{11}$), sagged section 1 ($l_{12}$), hogged section 2 ($l_{22}$), sagged section 2 ($l_{21}$), and hogged section 3 ($l_{33}$). Subsequently, the full surface deformations are applied to the buildings in the zonal sections, and the tensile strains generated in each section are analyzed.

The bending tensile strain ($\varepsilon_{b1}$, $\varepsilon_{b2}$, $\varepsilon_{b3}$) and shear tensile strain ($\varepsilon_{d1}$, $\varepsilon_{d2}$, $\varepsilon_{d3}$) generated in each section of the building are sequentially calculated by (2) and (3) [25], considering only the effect of surface settlement, geometric size, and stiffness of the building.

$$\varepsilon_{b} = \frac{\Delta l}{(l/12t + (3l/2tH)(E/G))}$$  \hspace{1cm} (2)

$$\varepsilon_{d} = \frac{\Delta l}{(1 + (HL^2/181))(G/E)}$$  \hspace{1cm} (3)

where $\Delta l$ is the deflection ratio, which is equal to the ratio of the relative deflection of the bending section to the span; $H$ is the building height; $l$ is the moment of inertia of the elastic beam per unit width; $E$ and $G$ are the elastic modulus and shear modulus of the building, respectively; $t$ is the maximum distance of the neutral axis of the elastic beam from the beam boundary, with $t = H$ in the hogged section and $t = H/2$ in the sagged section.

Tensile strain generated within the building by horizontal displacement of the ground surface can be derived from (4). Relevant studies [1] have shown that the average horizontal strain is more suitable for assessing the potential damage to a building than the local horizontal strain. The average horizontal strain value within the hogged section (sagged section) is calculated by dividing the horizontal displacement difference between the ends of the section by the span.

$$\varepsilon_{h} = \frac{\partial U}{\partial x} = \frac{U_1 - U_2}{x_1 - x_2}$$  \hspace{1cm} (4)

where $U$ is the horizontal displacement at a certain point on the building; $x$ is the horizontal coordinate value of a point.
on the building. The horizontal strain value in the sagged section of the building is considered zero, neglecting the effect of the negative horizontal strain in the sagged section [1].

The maximum bending tensile strain \( \varepsilon_{br} \) and the maximum shear tensile strain \( \varepsilon_{dr} \) in buildings caused by surface settlement and horizontal displacement are calculated using (5) and (6).

\[
\varepsilon_{br_{max}} = \varepsilon_h + \varepsilon_{br_{max}},
\]
\[
\varepsilon_{dr_{max}} = \varepsilon_h \left( 1 - \nu \right) + \sqrt{\varepsilon_h^2 \left( 1 + \nu \right)^2 + \varepsilon_{dr_{max}}^2},
\]

where \( \nu \) is the Poisson ratio, taken as 0.3.

The maximum value of the maximum bending tensile strain \( \varepsilon_{br_{max}} \) and the maximum shear tensile strain \( \varepsilon_{dr_{max}} \) in each zone is selected as the maximum tensile strain value \( \varepsilon_{max} \) in the building obtained from this stochastic analysis. Compared with the ultimate tensile strain \( \varepsilon_{ut} \) to assess the damage level of the building, the correspondence is shown in Table 3 [8].

4. Random Reliability Analysis of Building Safety

4.1. Stochastic Analysis Schemes. Combining with random field theory, finite difference method, and Monte Carlo simulation, the effect of the variation of the vertical and transverse scales of fluctuation \( \theta z \) of soil elastic modulus on the surface deformation mode and the maximum tensile strain in buildings is studied under different configurations and construction sequences of twin shield tunnels. At the same time, the other physical and mechanical parameters are constant. Drawing on the assessment criteria of the ultimate tensile strain, a probabilistic analysis of the safety of masonry buildings is carried out. Referring to studies by relevant scholars [19] on the value of the scales of fluctuation of the soil parameter, the vertical scale of fluctuation of the soil parameter of \( \theta z = 0.25D = 1.55 \) m and the transverse scale of fluctuation of \( \theta z = 8.0D = 49.6 \) m are selected as reference values of the scales of fluctuation of the parameter. Table 4 shows the values of the parameters calculated for each of the 14 stochastic analysis conditions in parallel twin tunnels with a spacing of \( L_n = 2.0D \). In the working conditions of overlapping twin tunnels and shoulder-mounted twin tunnels, the effect of the excavation sequence on the analysis results is emphatically studied, with the transverse scale of fluctuation fixed and different vertical scales of fluctuation, and 14 working conditions are designed, respectively. Table 5 lists the working conditions of shoulder-mounted twin tunnels with an inclination angle of \( \alpha = 45 \). In addition, 1000 times are selected as the number of stochastic analyses for each condition.

4.2. Analysis of the Surface Deformation Mode Caused by the Construction of Twin Tunnels. Surface deformation is one of the most critical indicators for reflecting the effect of tunnel construction on the surrounding environment. It is also a prerequisite for performing a safety assessment of the upper buildings. Relevant studies [17] have shown that considering the effect of modulus spatial variability, the 95% quantile value and the coefficient of variation of the maximum surface settlement caused by the construction of twin tunnels increase with increasing scales of fluctuation, and the degree of dispersion of the distribution of the settlement curve becomes greater. This subsection focuses on analyzing the distribution of surface deformation mode obtained from each stochastic analysis condition under different configurations and construction sequences of twin shield tunnels, which provides a basis for the subsequent analysis of the safety assessment of the building.

The surface deformation modes caused by the construction of twin tunnels can be divided into two categories: single peak type and double peak type. The settlement curve is divided into left and right sections from the central symmetry point. The locations of the maximum settlement points that appear in the left and right sections are judged, respectively. According to the maximum settlement value of the left and right sections and their occurrence locations, the surface deformation modes are divided into six types; I to VI are represented in order: the double peak type and the left and right wave peaks are the same, the double peak type and the left wave peak are more prominent, the double peak type
and the right wave peak are more prominent, the single peak type and the wave peak are located at the center symmetry point, the single peak type and the wave peak are to the left, and the single peak type and the wave peak are to the right.

4.2.1. Parallel Arrangement Form. Figure 5 shows the distribution of the various surface deformation curves obtained from the stochastic analysis in three parallel twin tunnels ($L_x = 2.0D, 2.5D, 5.0D$) under different scales of fluctuation conditions. Note that “the deterministic model is V” represents that the surface deformation mode obtained from the deterministic calculation is V, the same below. The statistical results under various transverse scales of fluctuation cases (MCS-$z$ case group, $\theta_z = 0.25D$) are shown on the left side of Figure 5. It can be seen that the random characteristics of the surface deformation mode gradually weaken with increasing transverse scales of fluctuation at the vertical scale of fluctuation of $0.25D$ and mostly exhibit the same surface settlement deformation mode as the deterministic analysis results, with a high peak distribution. This is mainly because in an anisotropic random field, the

| Damage level | Severity degree  | Maximum tensile strain (%) | Ultimate tensile strain (%) |
|--------------|-----------------|-----------------------------|-----------------------------|
| 0            | Negligible      | 0–0.050                     | 0.050                       |
| 1            | Very slight     | 0.050–0.075                 | 0.075                       |
| 2            | Slight          | 0.075–0.150                 | 0.150                       |
| 3            | Medium          | 0.150–0.300                 | 0.300                       |
| 4            | Serious         | >0.300                      | -                           |
| 5            | Very serious    | -                           | -                           |

Table 3: Relationship between the category of building damage and ultimate tensile strain.

| Working condition name | Variable | Construction sequence | Coefficient of variation COV | Scales of fluctuation |
|------------------------|----------|-----------------------|------------------------------|-----------------------|
| MCS-z1                 | $\theta_z$ | Right first and then left | 0.2                          | $L_z/4$ 8.0D 16       |
| MCS-z2                 |          |                      |                              | $L_z/2$ 8            |
| MCS-z3                 |          |                      |                              | $L_z/8$ 32           |
| MCS-z5                 |          |                      |                              | $L_z/16$ 64          |
| MCS-z6                 |          |                      |                              | $L_z/32$ 128         |
| MCS-z7                 |          |                      |                              | $L_z/64$            |
| MCS-x1                 |          |                      |                              | $L_z/8$ 16           |
| MCS-x2                 |          |                      |                              | $L_z/32$ 128         |

| Working condition name | Variable | Construction sequence | Coefficient of variation COV | Scales of fluctuation |
|------------------------|----------|-----------------------|------------------------------|-----------------------|
| MCSd-z1                | $\theta_z$ | Down first and then up | 0.2                          | $L_z/4$ 8.0D 16       |
| MCSd-z2                |          |                      |                              | $L_z/2$ 8            |
| MCSd-z3                |          |                      |                              | $L_z/8$ 32           |
| MCSd-z5                |          |                      |                              | $L_z/16$ 64          |
| MCSd-z6                |          |                      |                              | $L_z/32$ 128         |
| MCSd-z7                |          |                      |                              | $L_z/64$            |
| MCSu-z1                |          |                      |                              | $L_z/8$ 16           |
| MCSu-z2                |          |                      |                              | $L_z/32$ 128         |

* $\xi$ is the anisotropic coefficient, and $\xi = \theta_z/\theta_x$, the same below.

Table 4: Calculated cases of stochastic analysis of parallel twin tunnels ($L_x = 2.0D$).

Table 5: Calculated cases of stochastic analysis of shoulder-mounted twin tunnels ($\alpha = 45^\circ$).
Figure 5: Statistical results of the mode of the surface settlement curve under different scales of fluctuation. (a) MCS-(x) working condition group in parallel arrangement, $L_x = 2.0(D)$; (b) MCS-(z) working condition group in parallel arrangement, $L_x = 2.0(D)$; MCS-(x) working condition group is on the left of the figure, and the right shows MCS-(z) working condition group, the same below; (c) MCS-(x) in parallel arrangement, $L_x = 2.5(D)$; (d) MCS-(z) in parallel arrangement, $L_x = 2.5(D)$; (e) MCS-(x) in parallel arrangement, $L_x = 5.0(D)$; (f) MCS-(z) in parallel arrangement, $L_x = 5.0(D)$. 

The deterministic model is V
The deterministic model is IV.

There are a few types of II and III.

In addition, by comparing the analysis results under different horizontal spacing conditions, it can be seen that, regardless of the transverse or vertical scales of fluctuation conditions, the surface settlement curve type is primarily single peak, and the peak is to the left when the spacing $L_x = 2.0D$, that is, the surface deformation tilts towards the backward tunnel; when the spacing $L_x = 2.5D$, the surface deformation modes obtained from the stochastic analysis are abundant. The mutual effect of construction is weaker under this horizontal spacing condition than when $L_x = 2.0D$. The spatial variability of the parameters plays a dominant role in the diversity of surface deformation mode; that is, the mutual effect degree of construction is controlled by the spacing of the tunnel axis and the spatial variability of the parameters. With the further increase of horizontal spacing, the effect of scales of fluctuation on surface deformation mode gradually weakens, and the number of settlement curves of double peak type increases.

### 4.2.2. Overlapping Arrangement Form

Figure 6 shows the variation of the number of surface settlement curves’ types with vertical scales of fluctuation in the overlapping twin tunnels with different excavation sequences. In the overlapping twin tunnels, the distribution law of the surface deformation mode caused by construction with vertical scales of fluctuation is consistent with that of the working condition of the parallel twin tunnels, which shows that, with increasing vertical scales of fluctuation, the distribution of the surface deformation mode has low peak distribution characteristics. The surface deformation mode is a single peak when the underlying tunnel is excavated first. The peak is located above the tunnel axis in most cases, and a few deviate from the axis. When the overlying tunnel is excavated first, there are a few double peak settlement curves besides single peak curves; that is, the surface deformation mode caused by the overlying tunnel excavation first is more complicated.

### 4.2.3. Shoulder-Mounted Arrangement Form

The deterministic model is V.

Figure 7 shows the distribution of the types of settlement curves obtained by stochastic analysis with the various vertical scales of fluctuation in the shoulder-mounted twin tunnels with different excavation sequences. For shoulder-mounted tunnels ($\alpha = 39, 45$), due to the effect of tunnel depth, regardless of the excavation sequence, the final surface settlement mode is of a single peak type, and the peak
is to the left (inclined to the side of the shallow buried tunnel).

4.3. Building Safety Assessment Analysis. In Section 4.2, the surface deformation modes caused by the twin tunnel construction under different conditions of spatial variability were analyzed. In this subsection, an analysis method based on random field theory for assessing the safety of buildings above twin tunnels is used to investigate the variation law of maximum tensile strains within adjacent buildings caused by the twin tunnel construction under various working conditions and to make a probabilistic analysis of the safety of masonry buildings by drawing on the ultimate tensile strain assessment criteria.

The building on the ground surface is assumed to be a masonry structure \((E/G = 2.6)\) with a height of \(H = 10.0\) m, and the length \(L\) is greater than the effect range of surface settlement. According to (2)–(6), the maximum tensile strain \(\varepsilon_{\text{max}}\) generated in the building can be calculated under different working conditions. The damage level of the building can be determined according to Table 1. The probability that the maximum tensile strain \(\varepsilon_{\text{max}}\) in the building exceeds the ultimate tensile strain \(\varepsilon_{\text{ult}}\) in the stochastic analysis is defined as the probability that the deformation of the building exceeds the criteria; that is,

\[
P_f = \frac{N_f}{N} \times 100%,
\]

where \(N\) is the number of stochastic calculations corresponding to the working conditions, and \(N = 1000\); \(N_f\) is the number of times that the maximum tensile strain \(\varepsilon_{\text{max}}\) in the building exceeds the ultimate tensile strain \(\varepsilon_{\text{ult}}\) in \(N\) stochastic calculations.

4.3.1. Parallel Arrangement Form

(1) Analysis of Maximum Tensile Strains in Buildings. 1000 Monte Carlo stochastic analyses had been carried out for any working conditions. The spatial variability of the soil elastic modulus has an effect on each of the digital characteristics (mean, quantile values, and coefficient of variation) of the maximum surface settlement caused by twin tunnel construction [17]. Figure 8 shows the variation curves of the average, 95\% quantile, and coefficient of variation of the maximum tensile strains in the building caused by surface deformation with the vertical scales of fluctuation for each parallel arrangement stochastic analysis case. The dashed and solid lines in the figure represent the maximum tensile strains in the building under the effect of the first tunnel excavation and the joint effect of twin tunnels, respectively. Note that “first” and “final” indicate the effect of the first tunnel excavation and the joint effect of twin tunnels, respectively. It can be seen that, regardless of whether it is affected by single tunnel excavation or the joint effect of twin tunnels, the average maximum tensile strain in the building is little affected by the change in scales of fluctuation; the 95\% quantile and coefficient of variation of the maximum tensile strain increase with the increase in scales of fluctuation. Furthermore, the maximum tensile strain in the building under the joint effect of twin tunnels is more excellent than that under the single tunnel working condition. Generally speaking, considering the spatial variability of the elastic modulus of the soil, the variation law of maximum tensile strain in buildings caused by twin shield tunneling with scales of fluctuation is basically consistent with that of maximum surface deformation [17].

Figure 9 shows the comparison of the average maximum tensile strain in the building obtained by stochastic analysis and the results of the deterministic analysis in each parallel twin tunnel with the scales of fluctuation of \(\theta_{z} = 0.25D; \theta_{z} = \infty\). As can be seen from the figure, the average maximum tensile strain obtained by stochastic analysis is slightly greater than that obtained by deterministic analysis under different horizontal spacing. In addition, the maximum tensile strain in the building tends to decrease and then increase with the horizontal spacing \(L_x\). When the spacing is small, the mutual effect of excavation is significant, the final surface settlement is large, and the deflection ratio of the hogged section of the settlement trough is also larger. As the horizontal spacing increases, the mutual effect between the construction of twin tunnels diminishes, and the average maximum tensile strain in the building also decreases. When the tunnel spacing changes from 3.0\(D\) to 4.0\(D\), the maximum tensile strain variation curve increases again significantly, mainly because the construction of twin tunnels has little mutual effect after the spacing increases to 3.0\(D\). The significant double peak settlement curve results in severe deformation of the hogged Section 2 (middle) of the building, and the internal tensile strain of the building is also large.

(2) Analysis of Probabilities of the Safety and Damage Levels of Buildings. Taking parallel twin tunnels of \(L_x = 5.0D\) as an example, the effects of vertical scales of fluctuation on the probability of buildings’ safety and damage level in an anisotropic random field are analyzed. Figure 10 shows the probability distribution of the maximum tensile strain in the building exceeding the ultimate tensile strain for four cases of the vertical scales of fluctuation \(\theta_{z} = L_z/32, L_z/8, L_z/2,\) and...
under the effect of single tunnel excavation and the joint effect of twin tunnels, respectively. The dashed line in the figure shows the result of the deterministic analysis. It can be seen that the higher the vertical scales of fluctuation, the greater the degree of dispersion of the distribution of the maximum tensile strain in the building obtained from the stochastic analysis. According to the surface deformation mode analysis in Section 4.2, when the tunnel spacing $L_x$ is 5.0$D$, the double peak characteristics of the final settlement curve are significant, and the tensile strain in the building at the location of the hogged Section 2 often tends to be higher. The results in Figure 10(b) show that the current damage state of the building is mostly at level 2 (slight damage degree) with a few occurrences at level 3 (moderate damage level) under the joint effect of twin tunnels. This result cannot appear in the deterministic analysis (level 1) at the same parameter value level. The literature [1] pointed out that many factors contribute to the damage of level 2 of a building, including shrinkage and temperature effects; however, these factors generally do not make the building reach level 3 damage. Therefore, the classification criterion of level 2 and level 3 of building damage ($\epsilon_{ult} = 0.15\%$) is very critical. Once the damage level of a building reaches level 3, it can be considered that the surface deformation has caused a certain degree of impact on the safety of the building. In the other parallel twin tunnel cases, the variation of the safety probability and damage level of buildings with scales of fluctuation is the same as that in the condition of $L_x = 5.0D$. Furthermore, the maximum tensile strains in the building are bending strains under various cases and are primarily found in the hogged section of the building. In general, taking into account the spatial variability of the parameters,
the probability of higher damage levels in the building increases significantly, and the more significant the scales of fluctuation, the greater the probability of damage.

4.3.2. Overlapping Arrangement Form

(1) Analysis of Maximum Tensile Strains in Buildings. In the overlapping twin tunnels, the variation law of the maximum tensile strain in the building obtained by stochastic analysis with the scales of fluctuation is the same as that of the parallel arrangement, so this subsection will not repeat it because of the space limitation. Figure 11 shows the distribution of the average maximum tensile strain in the building obtained by stochastic analysis under different excavation sequences in the overlapping twin tunnels. Note that “Overlapping (up)” and “Overlapping (down)” indicate that the overlying tunnel is excavated first and the underlying tunnel is excavated first, respectively. It can be seen that when the overlying tunnel is

![Figure 9: Relationships between horizontal spacing and maximum tensile strain.](image)

![Figure 10: Probability of exceeding εult under different vertical scales of fluctuation. (a) The effect of the first tunnel excavation. (b) Joint effect of twin tunnels excavation.](image)

![Figure 11: Effects of vertical scales of fluctuation on the average maximum tensile strain.](image)
Probability of exceeding ultimate tensile strain \( P_{\text{f}} \)

- 0.75
- 0.78
- 0.81
- 0.84
- 0.87
- 0.90
- 0.93

Ultimate tensile strain (%)

- 1/32
- 1/16
- 1/8
- 1/4
- 1/2
- 1
- 2


Figure 12: Probability of exceeding \( \varepsilon_{\text{ult}} \) under different vertical scales of fluctuation.

excavated first, the average maximum tensile strain in the building is approximately 3.0% larger than that of the underlying tunnel. In the working conditions of overlapping twin tunnels, the construction of the overlying tunnel is completed first, and when the underlying tunnel is excavated, the existence of the first tunnel has a sure “occlusion” effect on the settlement of the strata above it. Similarly, when the underlying tunnel is excavated first, the existence of the first tunnel will restrain the rebound effect of the soil at the bottom of the tunnel and slow the settlement trend of the soil. The maximum surface settlement value under the two construction sequences is roughly the same. It should be noted that the disturbance of the underlying tunnel excavation will cause a more significant additional settlement of the overlying tunnel and stratum in the case where the upper tunnel is excavated first, causing more significant tensile strain in the building. Thus, during the construction, effective measures should be taken to control the upward transmission of the deformation caused by the excavation of the underlying tunnel to protect the safety of the overlying tunnel.

(2) Analysis of Probabilities of the Safety and Damage Levels of Buildings. Figure 14 shows the probability distribution of the maximum tensile strain in the building that exceeds the ultimate tensile strain in shoulder-mounted twin tunnels with different excavation sequences under the two cases of the vertical scales of fluctuation \( (\theta_z = L_z/32 \text{ and } 2L_z) \). It can be seen that the higher the vertical scales of fluctuation, the higher the probability of higher damage levels in the building. Furthermore, the comparison reveals that buildings are safer when the overlying tunnel is excavated first in shoulder-mounted twin tunnels.

4.3.3. Shoulder-Mounted Arrangement Form

(1) Analysis of Maximum Tensile Strains in Buildings. Figure 13 shows the distribution of the average maximum tensile strain in the building obtained from stochastic calculation under different excavation sequences in the shoulder-mounted twin tunnels. In the shoulder-mounted twin tunnels, the effect law of excavation sequence on the average maximum tensile strain in the building is opposite to that of the overlapping cases. When the underlying tunnel is excavated first, the average maximum tensile strain in the building is about 5.2% greater. When comparing the two cases of horizontal spacing, it can be seen that with increasing horizontal spacing \((\alpha)\) changes from 45 to 39), the average maximum tensile strain in the building decreases.

4.4. Analysis of the Impact of the Length and Location of the Building. In fact, a building rarely spans the entire zone affected by surface deformation. In order to provide more valuable engineering reference suggestions, this subsection selects the results of the surface deformation analysis of twin tunnel cases with the scales of fluctuation closest to reality (roughly \( \theta_z = 0.25D_a; \theta_z = 8.0D \)) and investigates the variation of the maximum tensile strain in a building when its position is moved within the effect range of surface deformation for three different lengths of the building \((L_b = 10.0 \text{ m}, 20.0 \text{ m}, \text{ and } 30.0 \text{ m}; \text{ the position is characterized by the coordinates of the midpoint of the building})\). At the same time, referring to the suggestions given in the European Geotechnical Engineering Design Code [26] and the Chinese Structural Reliability Design Standard [27], it is defined that the probability of the maximum tensile strain of buildings exceeding the allowable ultimate tensile strain is not less than 5% in the
Probability of exceeding $\epsilon_{\text{ult}}$ under different vertical scales of fluctuation. 

The average of maximum tensile strain ($\%_o$)

The coordinates of building midpoint $x$ (m)

Figure 14: Probability of exceeding $\epsilon_{\text{ult}}$ under different vertical scales of fluctuation.

The coordinates of building midpoint $x$ (m)

Tunnel axis

Lb=30m

Lb=20m

Lb=10m

0.60

0.30

0.20

0.10

The average of maximum tensile strain ($\%_o$)

0.00

0.10

0.20

0.30

0.40

0.50

-50.0

-40.0

-30.0

-20.0

-10.0

0.0

10.0

20.0

30.0

40.0

50.0

Figure 15: Relationships between the average maximum tensile strain and the location of the building.

Figure 16: Relationships between the probability of exceeding $\epsilon_{\text{ult}}$ and the location of the building.

Figure 17: Relationships between the probability of exceeding $\epsilon_{\text{ult}}$ and the location of the building.

potential damage zone. Then, the potential damage zone of the buildings is analyzed to provide a more appropriate assessment for the current risk state of the buildings.

Figure 15 shows the distribution of the average maximum tensile strain in buildings with different lengths caused by the excavation of the first tunnel (right tunnel) in the stochastic analysis case of parallel twin tunnels with $L_x = 3.0D$. Note that "critical value" is the critical value between level 0 and level 1. It can be seen that, within the effect range of surface settlement, the longer the building’s length, the greater the maximum tensile strain in the building caused by single tunnel excavation. When the building’s length is 10.0 m or 20.0 m, and its midpoint is located near the center axis of the tunnel, the maximum tensile strain in the building is relatively small. This is consistent with the conclusion of the previous analysis that the maximum tensile strains in the sagged section of the building often tend to be smaller than that in the hogged section. Under the above conditions, the sagged section of the building occupies the majority of the entire building, so the maximum tensile strain is relatively small at this location.

Figure 16 shows the variation in the probability that the damage level of the building will reach level 2 ($\epsilon_{\text{ult}} = 0.75\%$) with the coordinates of the midpoint of the building in a stochastic analysis case of parallel twin tunnels with $L_x = 4.0D$ when the building’s length is 20.0 m after the excavation of twin tunnels is completed. It can be seen that the damage level reaches level 2 in the deterministic analysis only when the center of the building is within the interval ($-7.0$ m, $6.5$ m); in the stochastic analysis, not only does this damage interval widen to ($-8.0$ m, $7.5$ m), but also when the center point of the building is located at ($-30.0$ m, $27.5$ m) and ($26.5$ m, $30.0$ m), the probability of the damage level reaching level 2 is about 6.5%. This means that the range of the potential damage zone of buildings will be expanded compared to the results of the deterministic analysis, considering the effect of spatial variability of the parameters.
There is a certain probability that some of the zones with higher tensile strain in buildings at the time of the deterministic analysis will become a potential damage zone.

Figure 17 shows the variation of the probability that the damage level of the building of $L_b = 30.0$ m will reach level 1 ($\epsilon_{ult} = 0.5\%$) with the coordinates of the midpoint of the building in each parallel twin tunnel with the first tunnel (right tunnel) excavation. None of the damage levels of the building reaches level 2 under the effect of a single tunnel. As can be seen in the figure, the interval of the potential damage zone of the building at level 1 (the interval where the probability of damage level up to level 1 exceeds 5%) obtained from the stochastic analysis with horizontal spacing from $2.0D$ to $5.0D$ is in the order of ($-21.0$ m, $34.0$ m), ($-19.0$ m, $35.0$ m), ($-17.5$ m, $37.0$ m), ($-14.5$ m, $40.0$ m), and ($-11.5$ m, $43.5$ m). When converting the potential damage interval into the distance between the midpoint of the building and the axis of the excavated tunnel, it is revealed that the distance value is approximately $27.0$ m. The furthest distances of the level 1 potential damage zone are $22.0$ m and $17.0$ m for the building’s lengths of $L_b = 20.0$ m and $10.0$ m, respectively, which are 1.5 to 2.5 m greater than the deterministic analysis results. In general, when the distance between the edge of the building and the tunnel axis is about $12.0$ m, most zones begin to show hogged deformation, and the maximum tensile strain quickly reaches the ultimate tensile strain of the corresponding level.

Figure 18 shows the distribution of the probability of the potential damage zone with a damage level 2 ($\epsilon_{ult} = 0.5\%$) when the position of the building ($L_b = 30.0$ m) is moved within the effect range of surface settlement after the excavation of twin tunnels is completed in each of the parallel twin tunnels. It can be seen that when the horizontal spacing is $2.0D$, $2.5D$, and $3.0D$, the potential damage zones of level 2 of the buildings (the interval where the probability of the damage level reaching level 2 exceeds 5%) are primarily located outside the tunnel axis. It is roughly symmetrical to the left and right. The building’s potential damage zones are ($-32.5$ m, $-12.5$ m) and ($12.0$ m, $33.0$ m) for a horizontal spacing of $L_x = 2.5D$, and the zone between the two tunnel axes is relatively safer. When the spacing is $L_x = 4.0D$ or $5.0D$, the potential damage zone of level 2 of the building mostly appears between the two tunnel axes.

Figure 19 shows the probability distribution of the potential damage zone with a damage level 2 of buildings caused by twin tunnels excavation in overlapping and shoulder-mounted twin tunnels. It can be seen that, in the shoulder-mounted twin tunnels, the potential damage intervals of level 2 of buildings are all located on the left side (the side of the shallow tunnel), which is roughly ($-32.5$ m, $-11.5$ m). When the overlying tunnel is excavated first in the overlapping twin tunnels, the two intervals of ($-21.5$ m, $-11.5$ m) and ($11.5$ m, $21.5$ m) are potential damage zones. In the working condition that the underlaying tunnel is excavated first, no matter where the building is located, its damage level does not reach level 2.

In summary, regardless of the building’s length, as long as the hogged section of the building occupies most of the building, the damage possibility will be significantly improved. Compared with deterministic analysis, the damage level of buildings obtained from the stochastic analysis is higher. The probability that the maximum tensile strain within the building exceeds the ultimate tensile strain value of the corresponding level in the relatively safe zone of the deterministic analysis, considering the effect of the spatial variability of the parameters, is most likely to be greater than the permissible engineering probability (5%). Particular attention needs to be paid to the case where the maximum tensile strain in the building obtained from the deterministic analysis is close to the ultimate tensile strain of the corresponding level. In addition, in the process of tunnel design and construction, the safe distance of the building can be appropriately widened by 1.5–2.5 m outwards compared with the deterministic analysis results to ensure that the damage probability of the building is less than the allowable reliability value (5%).
5. Conclusions

This paper has proposed a safety assessment method of masonry buildings above twin tunnels based on the random field theory, using the random reliability analysis method and considering the surface deformation mode. The effects of vertical and transverse scales of fluctuation on the surface deformation mode and the change law of the maximum tensile strain in upper masonry buildings caused by the construction of twin shield tunnels are studied systematically, considering configurations and construction sequences of twin tunnels and the size of the building. The safety and potential damage zones of buildings are evaluated by referring to the assessment criteria of the ultimate tensile strain. Several conclusions can be drawn from this study:

(1) Taking into account the effect of spatial variability of elastic modulus of the soil, the diversity of surface deformation modes caused by construction in parallel and overlapping twin shield tunnels increases with decreasing transverse scales of fluctuation or increasing vertical scales of fluctuation, and the effect of vertical scales of fluctuation is more significant. The surface deformation modes caused by the excavation of shoulder-mounted twin shield tunnels are all single peak types, and the peaks are biased towards the shallowly buried tunnel side.

(2) The average maximum tensile strain in buildings caused by the construction of twin shield tunnels is little affected by the scales of fluctuation. 95% quantile and coefficient of variation of the maximum tensile strain increase with the increase of the scales of fluctuation. The damage of buildings is mainly caused by bending tensile strain and mostly occurs in the hogged section.

(3) For the parallel twin tunnel conditions, the average maximum tensile strain in the building obtained from the stochastic analysis shows a trend of first decreasing and then increasing with the horizontal spacing $L_x$. At $L_x = 3.0D$, the maximum tensile strain in the building is slight, and the safety is relatively high. At $L_x = 5.0D$, the significant double peak characteristics of the settlement curve lead to severe hogging in the middle of the building and higher tensile strain in the building. In the overlapping twin tunnels, the average maximum tensile strain in the building caused by the excavation first of the overlapping tunnel is approximately 3.0% greater than that of the underlying tunnel. In shoulder-mounted twin tunnels, the effect law of excavation sequences on building safety is the opposite. The average maximum tensile strain of the excavation first condition of the underlying tunnel is approximately 5.2% larger.

(4) Within the effect range of surface settlement, the greater the building’s length, the higher the damage probability. Compared to the results of the deterministic analysis, the range of potential damage zones of the building will be expanded, taking into account the effect of the spatial variability of the parameters. There is a certain probability that some of the zones with considerable internal tensile strain in the buildings at the deterministic analysis will become damage zones. The potential damage zone of the building is approximately 12.0 m from the edge of the building to the tunnel axis under different scales of fluctuation conditions with a coefficient of variation of 0.2 when the single tunnel is excavated. The range of the potential damage zone of the building determined by stochastic analysis can be widened by about 1.5~2.5 m compared to the results of the deterministic analysis.

Data Availability

The data presented in this study are available upon request from the corresponding author.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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