

Article

Prediction and Engineering Verification of Building Settlement in Loess High-Fill Areas

Zhipeng Huo ^{1,*}, Xukun Han ² and Yafei He ²

¹ Department of Civil Engineering, School of Architectural Engineering, Yan'an University, Yan'an 716000, China

² Department of Civil Engineering, Yan'an University, Yan'an 716000, China; hanxukun@stu.edu.cn (X.H.); yafeihe@stu.edu.cn (Y.H.)

* Correspondence: huozhipeng1988@163.com

Abstract

To identify suitable settlement prediction methods for buildings constructed in loess high-fill areas, this study analyzes settlement monitoring data from a residential building in the Yan'an New District. Considering the pronounced compressibility of loess high fills and the depth-sensitive characteristics of differential settlement, four prediction approaches—the exponential curve method, the hyperbolic method, the code-based method, and the finite element method—are employed to forecast settlement. Fitting functions are established for each method to extrapolate the ultimate settlement. The research results indicate that, within the monitoring period, the exponential curve method and the hyperbolic method exhibit higher agreement with the measured settlement data, with prediction errors of 2.56% and 0.62%, respectively. These errors are significantly lower than those of the code-based method and the finite element method, which reach 45.6% and 77.4%, respectively. In particular, the hyperbolic method, through adaptive parameter iteration, controls the discrepancies between calculated and measured settlements at all monitoring points within a range of 0.62% to 8.89%. This method is therefore capable of more accurately capturing the settlement evolution characteristics, consistent with the long-term creep behavior of loess high-fill foundations, and provides a feasible reference method and practical decision-making support for building settlement prediction and engineering design in loess high-fill areas.

Keywords: loess high-fill area; building settlement prediction; hyperbolic method; creep settlement; engineering verification



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1. Introduction

In recent years, many cities in the Loess Plateau region of Western China have implemented large-scale “mountain leveling and valley filling” projects to effectively alleviate the shortage of urban construction land. Unlike natural loess, compacted loess fills are constructed layer by layer under controlled compaction conditions with different mechanical behaviors [1]. However, variations in compaction quality, moisture content, and fill thickness inevitably introduce spatial heterogeneity, which significantly affects the deformation behavior of the foundation [2]. For loess high-fill grounds with a thickness exceeding 20 m, long-term consolidation and creep deformation become prominent, making settlement prediction more complex and time-dependent [3]. Given the high compressibility, depth-dependent settlement sensitivity, and long-term creep characteristics of loess high-fill

foundations, it is necessary to evaluate the applicability of different settlement prediction methods based on field monitoring data.

Representative loess high-fill projects include Lanzhou New Area in Gansu Province, Lüliang Airport in Shanxi Province, and the Yan'an New District (North Area). Owing to the large variability in fill thickness and relatively high settlement rates, together with the instability of loess mechanical properties under dry–wet cycling conditions [4], such projects are prone to inducing uneven settlement of large buildings. This can lead to engineering hazards such as wall cracking, structural component damage, and overall building tilting. Moreover, creep settlement effects remain significant [5,6], resulting in a pronounced long-term settlement behavior, and the attainment of settlement stabilization often requires a considerable period of time. Therefore, it is essential to monitor and predict building settlement in loess high-fill areas to accurately characterize settlement development trends and to establish reliable predictive models for settlement evolution.

At present, a wide range of methods are available for the prediction and calculation of building settlement, which can generally be classified into two categories. The first category comprises theoretical approaches based on loess consolidation theory and constitutive models, such as the code-based method [7] and the finite element method [8,9]. The second category includes semi-theoretical and semi-empirical approaches that establish numerical models based on measured settlement data, such as time-series analysis [10], the logarithmic curve method [11], the hyperbolic method [12], the Hoshino method [13], and practical creep-based algorithms [14]. In recent years, advanced prediction techniques and improved analytical frameworks have also been proposed to enhance the accuracy of settlement prediction for high-fill and complex ground conditions, especially in transportation and large-scale infrastructure projects [15].

Extensive research on building settlement prediction has been conducted by scholars both in China and abroad. Wu Jiangbin et al. [16], taking the Tianjin Goldin 117 Tower as a case study, analyzed and compared measured settlement data with predictions obtained from hyperbolic curve fitting, thereby elucidating the settlement distribution and development characteristics. Wu Tong et al. [17] employed a grey model to predict settlement of existing buildings and compared the results with field measurements, demonstrating that the model exhibits relatively small prediction errors and is suitable for short-term settlement forecasting. He Yingpeng et al. [18] developed prediction models for cumulative settlement, settlement rate, and time for a high-rise building based on time series analysis, enabling short-term settlement prediction. Wang Junyu et al. [19] constructed an optimal combination settlement prediction model using measured data, effectively overcoming the limitations of the logistic and three-parameter power function models and improving prediction accuracy. Zhang et al. [20] established a regional multivariate settlement prediction model based on monitored settlement data from a high-rise residential building; their results showed that, compared with the logistic model, the proposed model offers higher accuracy and reliability. Utenov et al. [21], building upon a modulus-free calculation approach, developed a comprehensive methodology for predicting settlement evolution of building foundations during soaking processes before and after construction. Zhao Zhuangfu et al. [22] employed the finite element software PLAXIS 3D to establish several typical numerical models for loess high-fill foundations, systematically analyzed the settlement and deformation behavior of high-fill ground, and verified the validity of the proposed models.

In summary, although significant progress has been made in the study of building settlement prediction and calculation, research focusing on settlement prediction under the unique geological conditions of loess high-fill areas remains limited, and targeted engineering verification is still lacking. Owing to the distinctive soil mechanical characteristics

of loess high fills—such as high compressibility and pronounced sensitivity to differential settlement induced by depth-dependent behavior [23]—the settlement behavior of buildings in these areas is considerably more complex. In addition, the inherent uncertainty and time-dependent deformation of the foundation make it challenging to accurately identify model parameters and to capture long-term settlement evolution based on monitoring data. Consequently, inverse analysis methods have increasingly been introduced to address these issues [24].

Therefore, this study is based on long-term settlement monitoring data obtained from an actual residential building constructed on ultra-thick loess high-fill ground in the Yan'an New District. Four settlement prediction approaches—the exponential curve method, the hyperbolic method, the code-based method, and the finite element method—are systematically compared and evaluated to identify methods that are more suitable for settlement prediction in loess high-fill areas. Furthermore, a correction factor is proposed to adjust the prediction results of methods that exhibit poor agreement with measured data, and the applicability boundaries of each method in engineering practice are clarified.

2. Measured Settlement of Buildings in Loess High-Fill Areas

2.1. Project Overview

The residential project is located on a loess high-fill site in the Yan'an New District. The building is a seven-story residential structure with one underground level, with a total building height of 22.85 m. The foundation soil at the site is non-collapsible, and the bearing stratum consists of Layer ② compacted fill. The thickness of the original terrain fill exceeds 22 m. The main building and the adjacent one-bay perimeter zone are separated from the underground garage by settlement joints.

The foundation system adopts a raft foundation with dimensions of 40 m × 24 m × 0.5 m (length × width × thickness), and the concrete strength grade is C30. Construction of the project commenced on 15 March 2022, and the building has since been completed and delivered for use. The settlement monitoring data are complete, and the structure has reached a stable settlement state. The raft foundation and the soil stratigraphic profile are shown in Figure 1.

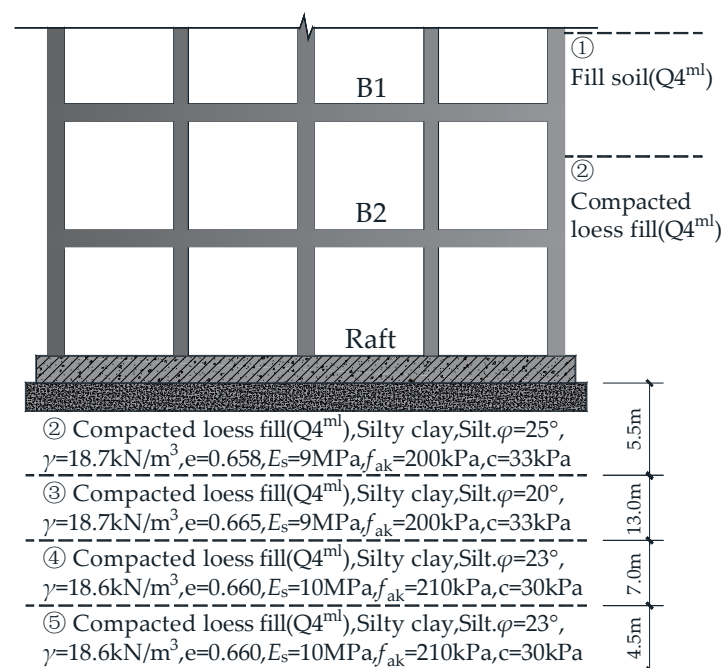


Figure 1. Raft foundation and soil stratigraphic profile.

2.2. Monitoring Scheme

Settlement monitoring for this project was conducted with second-order precision. One elevation control network was established, consisting of six elevation benchmark (BM) points and several working reference points. The elevation benchmarks were designated as BM1–BM6, and an independent elevation datum was adopted.

In determining the layout of monitoring points, both the deformation characteristics of the building and foundation were comprehensively considered, together with the geological conditions and structural features of the building. Accordingly, eight monitoring points, labeled A–H, were arranged, as shown in Figure 2.

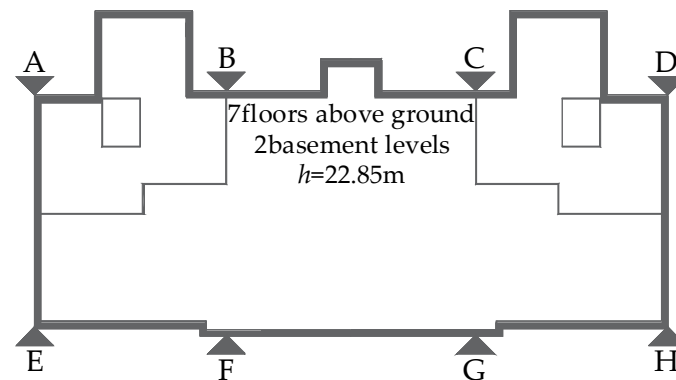


Figure 2. Layout of monitoring points.

Settlement measurements were carried out using an electronic precision level (Trimble DINI 03, Trimble Inc., Sunnyvale, CA, USA). Observations were performed along closed leveling routes using the geometric leveling method to determine settlement values. The monitoring schedule was as follows: one observation was conducted upon completion of the basement exterior walls up to the ± 0.000 level; thereafter, two observations were carried out for each additional construction story. After roof completion, six observations were conducted in the first year, two in the second year, and two in the third year.

2.3. Analysis of Monitoring Results

Based on the settlement monitoring data, 2 May 2022 was taken as the initial observation time ($t = 0$), and 11 May 2024, was chosen as the final observation time. The subsequent observation times correspond to days 8, 14, 21, 25, 35, 42, 49, 66, 73, 83, 91, 103, 120, 134, 142, 147, 152, 252, 318, 430, 624, and 740, respectively.

The maximum measured settlements at monitoring points A–H, denoted as S (i.e., the settlement values at $t = 740$ d), are -11.26 , -11.93 , -12.42 , -12.35 , -14.46 , -13.67 , -13.77 , and -13.38 mm, respectively. The measured settlement–time curves for each monitoring point are shown in Figure 3.

As shown in Figure 3, during the early stage of construction loading (approximately 0–120 d), the structural load increased rapidly with the progressive construction of the superstructure. Instantaneous settlement was dominant during this period, and the settlement rate increased relatively quickly. During the middle to late stages of construction loading (approximately 120–430 d), as secondary structural works and mechanical–electrical installations were gradually carried out, the rate of load increase slowed. Settlement during this stage consisted of both instantaneous settlement and consolidation settlement, and the settlement rate decreased; the cumulative settlement in this period constituted the main portion of the total settlement. In the post-construction stage (approximately 430–740 d), following the completion of interior finishing works and the transition to exterior works and maintenance activities, construction loads tended to stabilize. Settle-

ment in this stage was primarily governed by consolidation and secondary consolidation. The settlement rate during the final 100 d was approximately 0.006 mm/d, which, according to the current design code [25], satisfies the criterion for settlement stabilization (settlement rate $\leq 0.02\text{--}0.04$ mm/d, with the lower value adopted for cut areas and the higher value for fill areas).

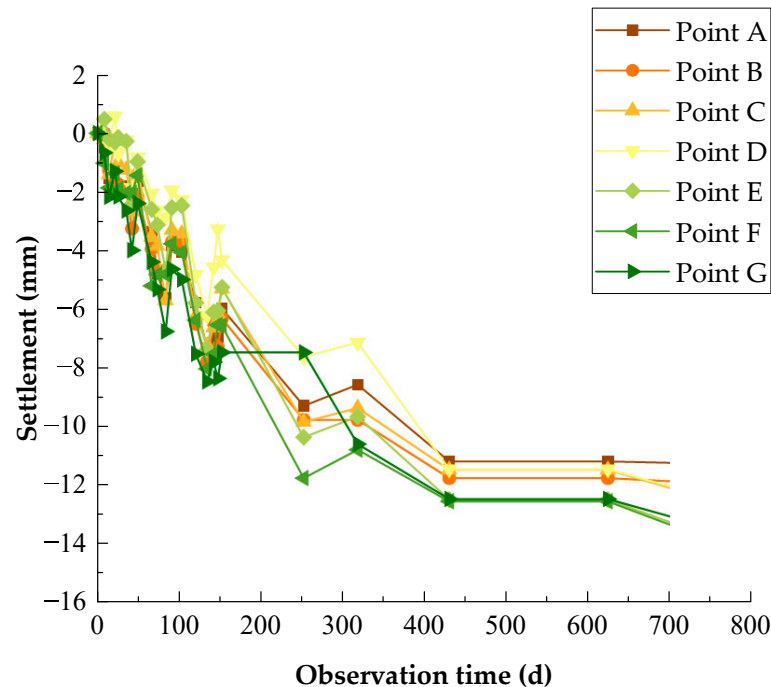


Figure 3. Measured settlement curves.

3. Settlement Prediction and Verification for Buildings in Loess High-Fill Areas

Taking a seven-story residential building located in the loess high-fill area of the Yan'an New District as a case study, existing measured settlement data were used to construct fitting curve models based on the exponential curve method and the hyperbolic method in order to analyze the settlement development trend and predict the ultimate settlement. Meanwhile, with reference to available geological data, foundation type, and superstructure loads, the ultimate building settlement was also calculated using the code-based method and the finite element method.

3.1. Calculation of Building Settlement in Loess High-Fill Areas

3.1.1. Exponential Curve Method

The core expression of the exponential curve method is given by

$$S_{\infty} = [S_{t3}(S_{t2} - S_{t1}) - \frac{S_{t2}(S_{t3} - S_{t2})}{(S_{t2} - S_{t1})} - (S_{t3} - S_{t2})] \quad (1)$$

$$S_d = \frac{S_{\infty}[S_{t1} - S_{\infty}(1 - \alpha e^{-\beta t})]}{\alpha e^{-\beta t}} \quad (2)$$

$$S_t = S_{\infty}(1 - \alpha e^{-\beta t}) + S_d \alpha e^{-\beta t} \quad (3)$$

$$\beta = \frac{1}{(t_2 - t_1)} \ln \left[\frac{(S_{t2} - S_{t1})}{(S_{t3} - S_{t2})} \right] \quad (4)$$

where S_{∞} denotes the ultimate settlement (mm), S_d is the instantaneous settlement (mm), S_t represent the delayed settlement at an arbitrary time t (mm), and α is a parameter

approximated as $8/\pi^2$ based on one-dimensional consolidation theory. S_{t1} , S_{t2} , and S_{t3} are the measured settlement values corresponding to times t_1 , t_2 , and t_3 , respectively, with t_3 corresponding to the end of the fitting curve, i.e., the last observation time. Through back-calculation and trial iterations, t_2 and t_1 are successively determined such that t_1 is minimized, while the time intervals $(t_3 - t_2)$ and $(t_2 - t_1)$ are equal. According to the actual settlement monitoring schedule described in Section 2.3, the final observation time in this case is 740 d; therefore, $t_3 = 740$ d. Trial calculations indicate that the conditions of equal time intervals and minimum t_1 simultaneously satisfied only when $t_1 = 120$ d and $t_2 = 430$ d, yielding a time interval of 310 d. The settlement values calculated using the exponential curve functions for the eight monitoring points (A–H) are listed in Table 1.

Table 1. Calculated settlement values based on the exponential curve method.

Observation Point	S_{t3}/mm	S_{∞}/mm	Functional Expression of S_t
A	−11.26	−11.26	$S_t = -11.26 + 29.57 \times e^{-0.014t}$
B	−11.93	−11.94	$S_t = -11.94 + 25.82 \times e^{-0.0111t}$
C	−12.42	−12.63	$S_t = -12.63 + 15.36 \times e^{-0.0055t}$
D	−12.35	−12.49	$S_t = -12.49 + 20.56 \times e^{-0.0065t}$
E	−14.46	−14.83	$S_t = -14.83 + 19.81 \times e^{-0.0051t}$
F	−13.67	−13.92	$S_t = -13.92 + 19.75 \times e^{-0.0056t}$
G	−13.77	−14.06	$S_t = -14.06 + 17.85 \times e^{-0.0053t}$
H	−13.38	−13.58	$S_t = -13.58 + 14.47 \times e^{-0.0055t}$

As shown in Table 1, at $t = 740$ d, the maximum settlement predicted by the exponential curve method is taken as the maximum value among the measured data at monitoring points A–H, i.e., $S_{t3,\max} = -14.46$ mm. As the time approaches to infinity, the maximum predicted ultimate settlement among all monitoring points, calculated using the exponential curve formula, is $S_{\infty} = -14.83$ mm. This value shows only a small deviation from $S_{t3,\max}$, with a relative error of merely 2.56%.

As indicated in Table 1, at $t = 740$ d, the maximum settlement predicted by the exponential curve method is taken as the maximum value among the available measured data at monitoring points A–H, namely $S_{t3,\max} = -14.46$ mm.

When time approaches to infinity, the maximum predicted ultimate settlement among all monitoring points calculated using the exponential curve formulation is $S_{\infty} = -14.83$ mm. The deviation between S_{∞} and S_t is small, with a relative error of only 2.56%.

3.1.2. Hyperbolic Method

The hyperbolic method, also referred as the hyperbolic fitting method, introduces four unknown parameters to construct a hyperbolic settlement function:

$$S_t = [-a(t - b)^c + d']^{-1} \quad (5)$$

where a and b , and c are parameters used to adjust the time variable t and d is the key parameter governing the ultimate settlement. By performing iterative fitting based on the measured building settlement data, these four unknown parameters can be determined, thereby yielding the settlement S_t at different times and the corresponding ultimate settlement S_{∞} for the each monitoring point. Similarly, using the measured settlement monitoring data of this seven-story Building, fitted hyperbolic settlement functions were established, and the calculated settlement values are summarized in Table 2.

Table 2. Calculated settlement values based on the hyperbolic function.

Observation Point	S_{t3}/mm	S_{∞}/mm	Functional Expression of S_t
A	−11.44	−12.37	$S_t = -1/[206.81 \times (t + 18.37) - 1.562 + 0.08087]$
B	−10.87	−13.12	$S_t = -1/[628.11 \times (t + 17.65) - 1.597 + 0.07620]$
C	−12.24	−13.48	$S_t = -1/[134.98 \times (t + 5.48) - 1.481 + 0.07419]$
D	−12.51	−18.18	$S_t = -1/[27.72 \times (t - 19.76) - 1.066 + 0.055]$
E	−14.37	−16.46	$S_t = -1/[119.85 \times (t + 14.65) - 1.436 + 0.06076]$
F	−13.42	−14.87	$S_t = -1/[175.86 \times (t - 7.66) - 1.530 + 0.06723]$
G	−13.33	−13.75	$S_t = -1/[34,805.55 \times (t + 62.28) - 2.474 + 0.07272]$
H	−13.20	−16.42	$S_t = -1/[9.08 \times (t + 0.002) - 0.971 + 0.06087]$

As indicated in Table 2, at $t = 740$ d, the maximum settlement among monitoring points A–H calculated using the hyperbolic method is $S_{t3,\max} = -14.37$ mm. Compared with the measured maximum settlement $S = -14.46$ mm, the error is only 0.62%. When time approaches to infinity, the maximum predicted ultimate settlement among monitoring points A–H obtained by the hyperbolic method is $S_{\infty} = -18.18$ mm, which shows the relative deviations of 25.7% and 26.5% with respect to S_t' and $S_{t3,\max}$, respectively.

Although the ultimate settlement S_{∞} has extrapolated by the hyperbolic method exhibits relatively large deviations compared with S_t' and $S_{t3,\max}$, the calculated settlements at all monitoring points within the time span of 0–740 d are very close to the measured values, with error ratios ranging from 0.62% to 8.89%. Moreover, the overall goodness of fit is high ($R^2_{\max} = 0.9942$, $R^2_{\text{avg}} = 0.9736$), which allowing the settlement to continue developing over a longer time period with a gradually decaying rate. This characteristic is more consistent with the long-term creep and secondary compression behavior commonly observed in compacted loess fills.

On the contrary, the exponential method shows good agreement with monitored settlement data at early stages, it inherently assumes an exponentially decaying settlement rate, which leads to a rapid convergence of settlement with time. This mathematical characteristic limits its ability to capture the long-term creep and secondary compression behavior of compacted loess fills. Consequently, when long-term monitoring data are not fully available, the exponential method tends to underestimate the ultimate settlement. That's why beyond the end of the settlement monitoring period (740 d), the hyperbolic method provides a more reasonable representation of the ultimate settlement behavior.

It should be noted that the ultimate settlement predicted by the hyperbolic method is obtained through extrapolation beyond the monitoring period, and therefore inherently involves uncertainty.

The uncertainty mainly arises from the limited duration of monitoring data, the assumption that the settlement mechanism remains unchanged in the future, and the inability to explicitly separate primary consolidation from secondary compression and creep using field data alone. Therefore, the predicted S_{∞} should be interpreted as an upper-bound estimate of long-term settlement rather than a deterministic value.

3.1.3. Code-Based Method

According to the code-based method [7], the ultimate settlement S_{∞} is calculated as

$$S_{\infty} = \psi_s S' = \psi_s \sum_{i=1}^n \frac{P_0}{E_{si}} (z_i \bar{\alpha}_i - z_{i-1} \bar{\alpha}_{i-1}) \quad (6)$$

where S' is the foundation deformation calculated using the layer-by-layer summation method (mm), and ψ_s is an empirical settlement coefficient determined based on regional settlement monitoring data and engineering experience. In the absence of regional experi-

ence, ψ_s may be determined according to the equivalent compression modulus (E_s) within the calculated deformation depth range and the additional stress at the foundation base, as specified in the current code (Code for Design of Building Foundations). Here, n denotes the number of soil layers within the calculated deformation depth; P_0 is the additional pressure at the foundation base under the quasi-permanent load combination (kPa); E_{si} is the compression modulus of the i -th soil layer beneath the foundation base (MPa), which should be evaluated over the stress range from self-weight stress to the combined self-weight and additional stress; Z_i and Z_{i-1} are the distances from the foundation base to the bottom of the i th and $(i - 1)$ th soil layers (m), respectively; and α_i and α_{i-1} are the average additional stress influence coefficients within the corresponding depth ranges, which may be obtained in accordance with the current design code.

This method considers only settlement caused by compressive deformation of soil layers and has clear physical and mechanical significance, representing a semi-empirical, semi-theoretical approach. When applying Equation (6), the physical properties of each soil layer shown in Figure 1 must be incorporated. In this study, the additional stress P_0 is 20.5 kPa, the empirical settlement coefficient ψ_s is 0.602, and the calculated ultimate settlement is $S_\infty = -21.06$ mm.

The code-based method can only provide the overall ultimate settlement, S_∞ , of the foundation based on geological conditions and additional stresses of each soil layer. Compared with the maximum measured settlement, S , presented in Section 2.3, the deviation reaches as high as 45.6%.

3.1.4. Finite Element Method

In this study, a numerical model was established using the finite element software ABAQUS (Dassault Systèmes Simulia Corp., Version 2020, Providence, RI, USA) [26]. The superstructure load was simplified as a uniformly distributed load acting on the raft foundation, and soil–structure interaction between the foundation and the superstructure was not considered. The soil layers were assumed to be homogeneous and continuous and to follow the Mohr–Coulomb elastoplastic constitutive model, while the raft foundation was modeled as a linear elastic body. During the initial geostatic stress equilibrium stage, the influence of the raft foundation and the lime–soil cushion on soil physical parameters was neglected.

To improve computational efficiency and considering the extent of foundation influence, the foundation soil model was defined with plan dimensions approximately three times those of the raft foundation. Further enlargement of the model domain resulted in only marginal changes in the calculated ultimate settlement at the foundation center, so the adopted domain size was considered sufficient for capturing the overall settlement behavior under static loading conditions. The soil domain dimensions were 120 m $\times$ 80 m $\times$ 50 m (length $\times$ width $\times$ height). The soil layers were divided according to Figure 1, and soils below Layer ⑤ were modeled using the properties of Layer ⑤. The soil parameters used in the analysis are listed in Table 3. The raft foundation concrete grade is C30, with an elastic modulus of 30 GPa, a density of 26 kN/m³, and a Poisson's ratio of 0.2.

In this study, the finite element analysis was performed based on a solid mechanics framework, aiming to estimate the ultimate settlement S_∞ of the loess high-fill foundation under completed construction loading. Time-dependent settlement S_t was not simulated in the FEM analysis. When solid mechanics-based FEM is adopted without fully coupled hydro-mechanical formulation, the computed settlement represents the equilibrium deformation under applied loads, corresponding to the ultimate settlement. Although time-dependent consolidation and creep settlement can be simulated using coupled hydro-mechanical analysis, such approaches require detailed permeability, consolidation,

and creep parameters, which are often difficult to obtain reliably for large-scale loess high-fill projects.

Table 3. Soil parameters used in the finite element analysis.

Material Parameters	Unit Weight $\gamma/(\text{kN/m}^3)$	Poisson's Ratio μ	Elastic Modulus E/MPa	Internal Friction Angle $\varphi/^\circ$	Cohesion c/kPa
Soil layer ②	18.7	0.3	40	25	35
Soil layer ③	18.7	0.3	45	20	33
Soil layer ④	18.6	0.3	45	23	30
Soil layer ⑤	18.6	0.3	40	21	25

Hexahedral elements were used for meshing, with 260 elements for the raft foundation and 66,160 elements for the soil mass. The element type adopted was C3D8R. The mesh size was determined considering both engineering experience and numerical accuracy. Finer meshes were adopted in regions with high stress gradients, such as the foundation slab and underlying soil, while coarser meshes were used in regions far from the structure. Trial analyses confirmed that further mesh refinement had a negligible influence on the calculated settlement result. The load was applied in stages corresponding to floor construction; within the main building footprint, the load per floor was taken as 14 kPa, applied over eight analysis steps. The lateral earth pressure coefficient was calculated as [27] $K = 1 - \sin 25^\circ = 0.577$. Hard contact was defined in the normal direction at the raft–soil interface, while tangential behavior was modeled using a penalty formulation with a friction coefficient of $\tan 25^\circ = 0.577$. Given the lack of long-term in situ modulus measurements, an equivalent elastic modulus was adopted for each soil layer to represent the average deformation behavior under working stress conditions. Boundary conditions were specified as normal constraints in the X and Y directions, full fixation at the bottom in the Z direction, and a free top surface. The numerical model is shown in Figures 4 and 5.

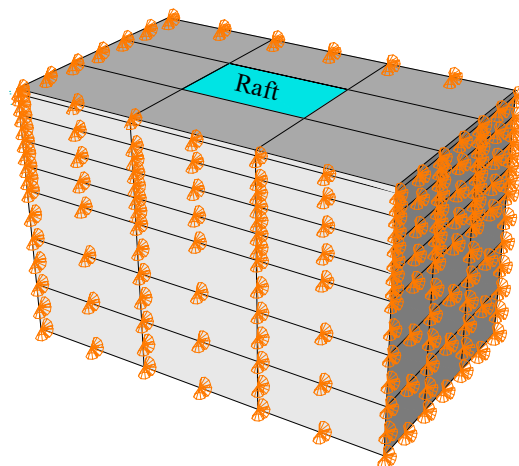


Figure 4. Model Boundary conditions.

After completion of the geostatic stress analysis, the initial geostatic stress equilibrium state of the foundation is shown in Figures 6 and 7. As illustrated in Figures 6 and 7, the settlement of the soil mass is nearly zero, and the stress distribution within the foundation under self-weight is uniform. The maximum von Mises stress at the bottom of the model is 467.4 kPa, which is consistent with the general characteristics of stress distribution. This indicates that the initial geostatic stress equilibrium of the foundation has been well established, providing a reliable basis for the subsequent settlement and stress analyses.

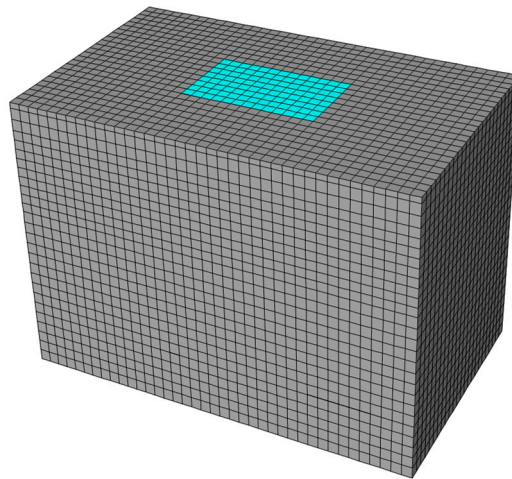


Figure 5. Model mesh discretization.

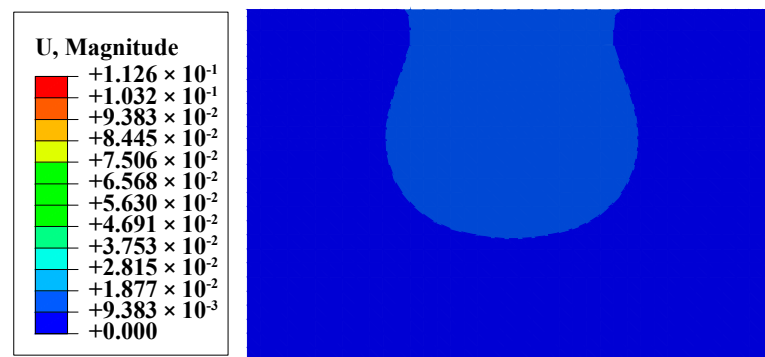


Figure 6. Initial soil settlement.

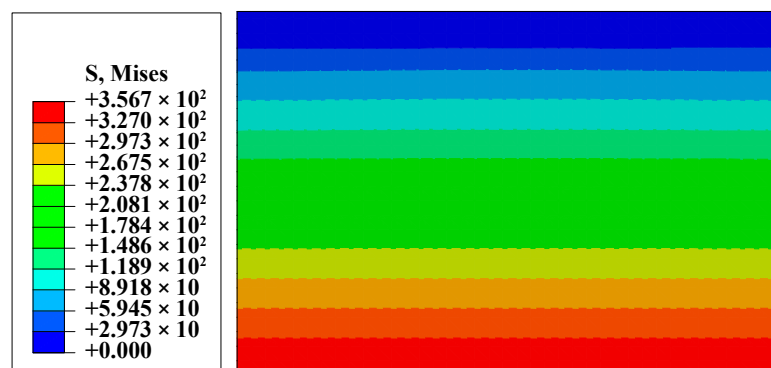


Figure 7. Initial geostatic stress.

After all loading steps in the analysis were completed, the settlement contour map and settlement curves of the foundation beneath the raft foundation are shown in Figures 8–10. As illustrated in Figures 8–10, the foundation exhibits a typical “bowl-shaped” concave deformation pattern, with larger settlement at the center and smaller settlement toward the edges. The maximum settlement occurs at the central area of the foundation, and the settlement along the depth direction is approximately linear. The ultimate settlement at the center of the foundation is $S_{\infty} = 25.65$ mm, which deviates from the maximum measured settlement S by as much as 77.4%. Moreover, the settlement decreases with increasing depth, indicating that the influence of depth on compressive deformation gradually diminishes.

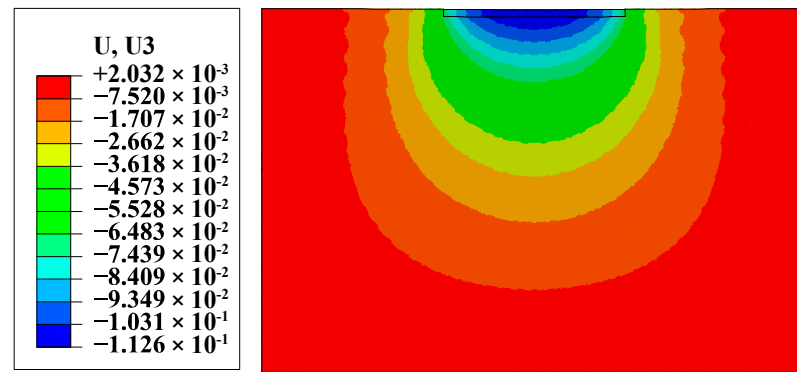


Figure 8. Foundation settlement contour (bird's-eye view).

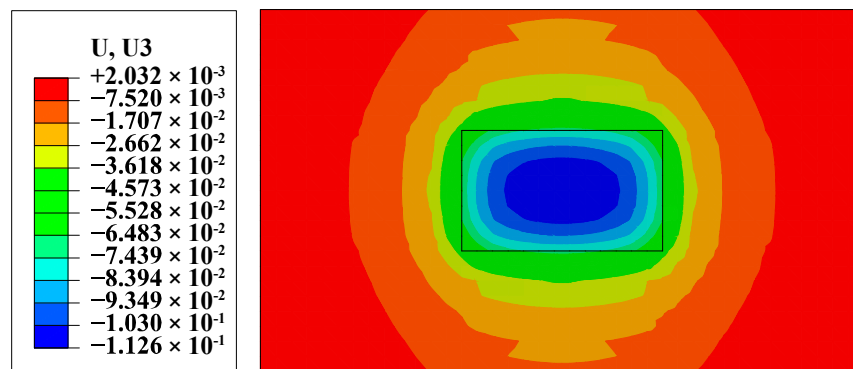


Figure 9. Foundation settlement contour (Sectional view).

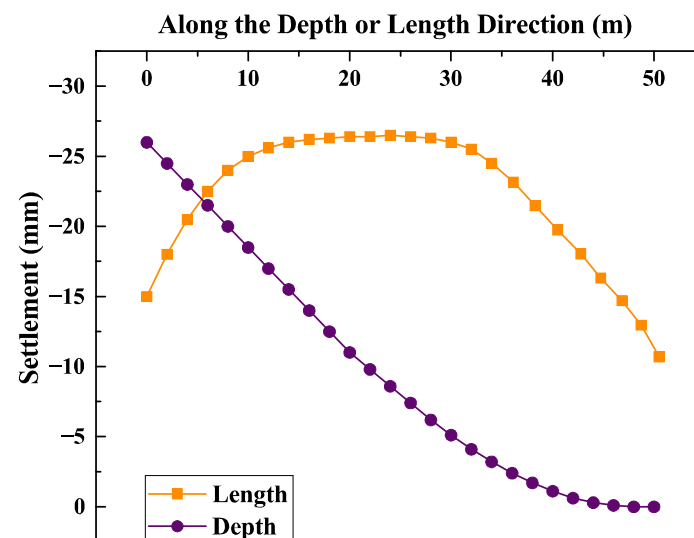


Figure 10. Foundation settlement curve.

3.2. Comparative Analysis of Calculation Results

A comparison between the settlements predicted by the four methods and the measured settlement values is presented in Table 4. As shown in Table 4, the exponential curve method and the hyperbolic method are capable of fitting settlement values at arbitrary times as well as predicting the ultimate settlement, and their fitting accuracy is significantly higher than that of the code-based method and the finite element method. The ratio of the ultimate settlements predicted by the former two methods to those predicted by the latter two methods ranges from 0.58 to 0.86.

Table 4. Comparison between predicted and measured settlement values.

Parameter	Measured Value	Exponential Curve Method	Hyperbolic Method	Code-Based Method	Finite Element Method
S_{t3} /mm	−14.46	−14.46	−14.37	/	/
Deviation from measured value/%	0.0	0.0	0.6	/	/
S_{∞} /mm	/	−14.83	−18.18	−21.06	−25.65
Deviation from minimum value/%	/	0.0	22.6	42.0	73.0

In contrast, the code-based method and the finite element method can only provide the ultimate settlement, and their predicted values are markedly larger—by approximately 42% to 73%—than those obtained using the exponential curve and hyperbolic methods. Compared with the measured settlement values, the errors are excessively large, indicating that these two methods cannot be directly used for predicting the ultimate settlement in this case.

The values of S_{t3} obtained using the exponential curve method and the hyperbolic method are very close; however, the ultimate settlement S_{∞} extrapolated by the exponential curve method is significantly smaller than that predicted by the hyperbolic method. Although both methods reflect the creep settlement behavior of loess high fills, the larger of the two predicted ultimate settlements should be adopted for design purposes. Accordingly, the ultimate settlement for this case study is taken as $S_{\infty} = -18.18$ mm.

The underlying reasons for these findings are twofold.

On the one hand, the code-based method does not account for the time-dependent creep behavior of fill soils. Its layer-by-layer summation parameters are primarily derived from laboratory test results, which may deviate from the complex in situ soil conditions, and the method neglects the lateral confinement of the soil. This conclusion is consistent with the finite element analysis results reported by Zhuangfu Zhao et al. [22].

On the other hand, the exponential curve method relies on only three observation points, and its formulation does not explicitly consider the future secondary consolidation (i.e., creep settlement) of loess high fills, thereby underestimating the ultimate settlement development [28–31]. In contrast, the hyperbolic method, through adaptive parameter adjustment, more effectively captures the nonlinear characteristics of settlement evolution and provides a more realistic representation of the long-term creep settlement behavior of loess high-fill foundations.

The overestimation of settlement predicted by the finite element model can be mainly attributed to several modeling assumptions. First, the analysis was conducted using a static elastic–plastic framework based on the Mohr–Coulomb constitutive model, in which time-dependent creep behavior of compacted loess was not explicitly considered.

Second, the Mohr–Coulomb model cannot fully capture the structural characteristics and stress-dependent stiffness degradation of compacted loess, which further contributes to conservative settlement predictions. Compared with these factors, the influence of mesh size, model dimensions, and load discretization is considered secondary in this study.

4. Conclusions

4.1. Time-Dependent Settlement Behavior of Loess High-Fill Foundations

Long-term monitoring data confirm that settlement evolution in loess high-fill foundations is governed by time-dependent consolidation and creep, with a progressively decreasing settlement rate.

This clear temporal pattern provides a reliable and necessary basis for evaluating the applicability of different settlement prediction methods, particularly for distinguishing short-term fitting performance from long-term extrapolation behavior.

4.2. Quantitative Comparison and Uncertainty of Settlement Prediction Methods

To further evaluate the applicability of different settlement prediction methods, a comparison of settlement evolution patterns was conducted in addition to numerical indicators. Figure 11 presents the measured and predicted settlement—time curves for representative monitoring point—point E.

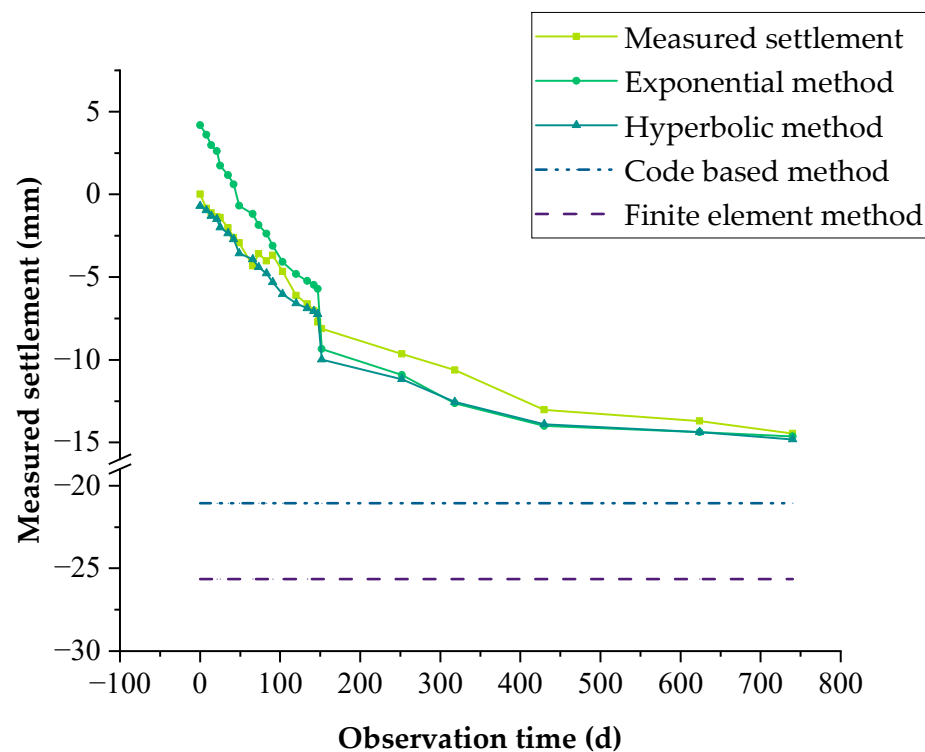


Figure 11. Settlement–time curves for representative monitoring points.

It can be observed that the code-based method and the finite element method significantly overestimate ultimate settlement, yielding values of -21.06 mm and -25.65 mm, respectively, compared with the measured maximum settlement of -14.46 mm. Both the exponential and hyperbolic methods achieve good agreement with monitoring data within the observation period, with prediction errors below 10%. However, notable divergence arises in long-term extrapolation, indicating that prediction uncertainty beyond the monitoring period is method dependent and should be explicitly considered in engineering applications.

4.3. Engineering Recommendation and Applicability

The hyperbolic method is more suitable for long-term settlement assessment due to its ability to reflect nonlinear creep behavior of compacted loess fills. An empirical correction factor of 0.6–0.8 is proposed to improve the accuracy of code-based settlement predictions. This factor is case specific and derived from a single project; its applicability depends on site conditions, fill properties, and construction quality. Regional settlement monitoring databases can be used to statistically calibrate correction factors through regression or probabilistic approaches, improving robustness and transferability in engineering practice.

4.4. Limitations of the Study and Future Research

1. This study is based on a single representative building project in the Yan'an New District. Although the monitoring data are continuous and reliable, the geological conditions, fill thickness, and construction methods may vary among different loess high-fill projects. Therefore, the conclusions should be applied with caution to other sites.
2. The finite element analysis in this study mainly focuses on the prediction of ultimate settlement. Time-dependent settlement evolution, such as consolidation and creep processes, was not explicitly simulated using fully coupled hydro-mechanical models. This simplification may limit the ability of the FEM results to capture long-term settlement development.
3. The constitutive behavior of loess high-fill soils was simplified in the numerical model by using equivalent elastic parameters. The degradation of soil structure, moisture variation, and cyclic wetting–drying effects were not fully incorporated, which may affect the accuracy of long-term settlement prediction.
4. It should be noted that this study is mainly based on field monitoring data and numerical analysis. Laboratory creep or consolidation tests on representative compacted loess samples were not conducted, which limits the direct verification of long-term creep mechanisms at the material level.
5. Future research may focus on the development of fully coupled hydro-mechanical numerical models to more accurately capture the time-dependent consolidation and creep behavior of loess high fills. In addition, it should incorporate laboratory creep and consolidation tests on compacted loess specimens to better quantify time-dependent deformation behavior and to establish more physically based settlement prediction models. Moreover, using multiple engineering cases from different loess high-fill sites is also recommended to enhance the general applicability of the proposed analysis framework. At last, integrating real-time monitoring data with adaptive prediction models may provide an effective approach for dynamic settlement assessment and long-term deformation control in loess high-fill engineering projects.

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