

Review of Soil Creep Characteristics and Advances in Modelling Research

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Abstract: Creep is recognised to be an important physical property of soils, exerting a profound influence on the stability of structures. In order to gain a comprehensive understanding of the advancements and focal points in soil creep research, the relevant literature was accessed from the Web of Science Core Collection database, totalling 3907 papers (as of 25 March 2024). Statistical analyses on publication volume, keyword co-occurrence, and clustering were conducted using the visualization software VOSviewer (1.6.20). The current hotspots in soil creep research were identified, and a systematic review was undertaken on the influencing factors of soil creep and the corrective methods of creep models. The research findings indicate that the number of papers on creep research exhibits a trend of increase followed by a decrease over time. Developed countries, such as those in Europe and America, initiated research in this field earlier than developing countries like China. Currently, the research focus is primarily centred on creep models. Significant differences exist in the creep deformation of soils under different influencing factors, with soil microstructure, moisture content, and stress path being important factors affecting soil creep deformation. Creep deformation in unsaturated soils primarily considers the influence of matric suction, while indoor creep tests are mainly conducted based on vertical loading, which differs significantly from the stress conditions experienced by soils in engineering construction sites. Currently, adjustments to soil parameters are mainly made through single-factor adjustments involving stress, time, damage, and matric suction to determine creep models under specific influencing factors, and then to modify the models accordingly. However, research on the creep deformation mechanism and creep models under multiple factors is relatively limited. Future research directions are expected to focus on the microscopic scale of creep mechanisms and multi-factor creep models.

Keywords: creep; soil; constitutive model; time dependence; visualization analysis



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1. Introduction

In the field of geotechnical engineering and scientific research, soil creep plays a crucial role. Its significant time dependency often leads to engineering issues such as excessive settlement, differential settlement, and slope instability. Given that the lifespan of engineering structures typically spans several decades or even centuries [1], elucidating the creep characteristics of soil is particularly critical [2].

Soil deformation comprises elastic deformation, primary consolidation settlement, and creep, with deformation gradually increasing and the rate decreasing over time. Numerous scholars have conducted research on the creep characteristics of different soils. Soft soil is particularly prone to creep deformation, characterised by low shear strength, high water content, and strong creep properties, and is widely distributed in river basins, deltas, and coastal plains. With the continuous development of society and the comprehensive

utilisation of underground space, numerous engineering structures need to be constructed in these areas, making it inevitable for foundation engineering to be situated in soft soil layers [3]. However, under long-term loading, various types of soil exhibit corresponding creep deformation. Creep refers to the slow movement and permanent deformation of soil particles under constant stress, which can have long-term impacts on the deformation of building structures [4]. Some scholars believe that the settlement of sandy soils is solely composed of elastic deformation, and is independent of time, and creep deformation generally does not occur at higher stress levels [5]. Although the creep of sandy soils is weaker than that of soft soils, if the thickness of the sandy soil is sufficient, its creep deformation will be significant. In addition, the plastic-viscous deformation of saturated sand is much greater than the viscoelastic deformation [6]. Loess exhibits significant creep deformation characteristics due to its high porosity, loose structure, and strong water sensitivity [7]. In geotechnical engineering practice, soil creep deformation often manifests as foundation settlement, slow slope slippage, and tunnel surrounding rock deformation. If these issues are not effectively controlled, they may lead to engineering structure failures, environmental damage, and economic losses.

To investigate the creep deformation of soils, the primary methods currently employed include one-dimensional consolidation creep tests [8], triaxial creep tests [9], shear creep tests [10], and ring shear tests [11]. To accurately describe soil creep deformation, numerous creep models have been proposed, including empirical models, component models, and semi-empirical plate theory models [12]. In recent years, numerous researchers have conducted in-depth studies on soil creep, achieving significant progress and generating a wealth of literature. However, due to the complexity and heterogeneity of soils, and the significant differences in creep characteristics among different types of soils, existing models still require continuous modification and improvement during application. Therefore, it is necessary to review and summarize the current status and hotspots of research in this field [13,14]. Although there have been reviews on creep deformation of soils, there are relatively few outcomes concerning the visualization analysis of research stages, hotspots, and trends in this field.

This paper conducts a visual study to perform a statistical analysis of the current publication volume, countries, institutions, and keywords related to soil creep. This is aimed at providing a better insight into the research hotspots and trends in this field, serving as a reference for scholars in the domain. Additionally, the paper provides a summary of ongoing research on factors influencing creep and methods for modifying creep models, while identifying current challenges and future development trends.

2. Data Source and Analyzation Method

To track the developmental trends of soil creep, this study employed bibliometric methods to statistically and visually analyse the relevant literature published in recent years. The specific research process is depicted in Figure 1. Firstly, the literature searches were conducted in the Web of Science Core Collection database targeting this field, and the search results were exported as text files. Subsequently, bibliometric analysis was conducted on the exported literature information using visualization software (VOSviewer) to uncover the primary research directions, hotspots, and trends in the study of soil creep and constitutive models.

The literature data were sourced from the Web of Science Core Collection database. The literature retrieval was conducted using the “all fields” method, and included keywords such as “creep” and “soil”. The cut-off date was 25 March 2024, resulting in the retrieval of a total of 3907 documents. The main research fields included Engineering Geological, Geosciences Multidisciplinary, and Engineering Civil, among others. The specific number of papers in the main research fields is shown in Figure 2. Subsequently, all the information of the literature was exported in text document format for scientific bibliometric analysis.

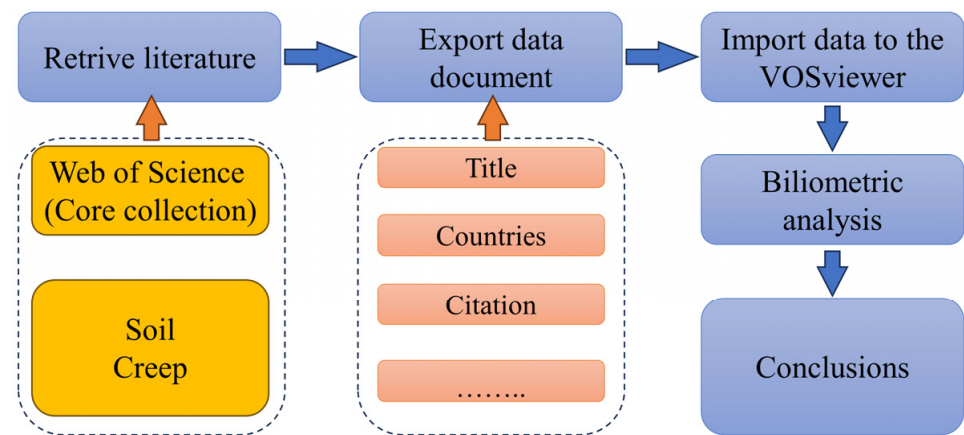


Figure 1. The research process.

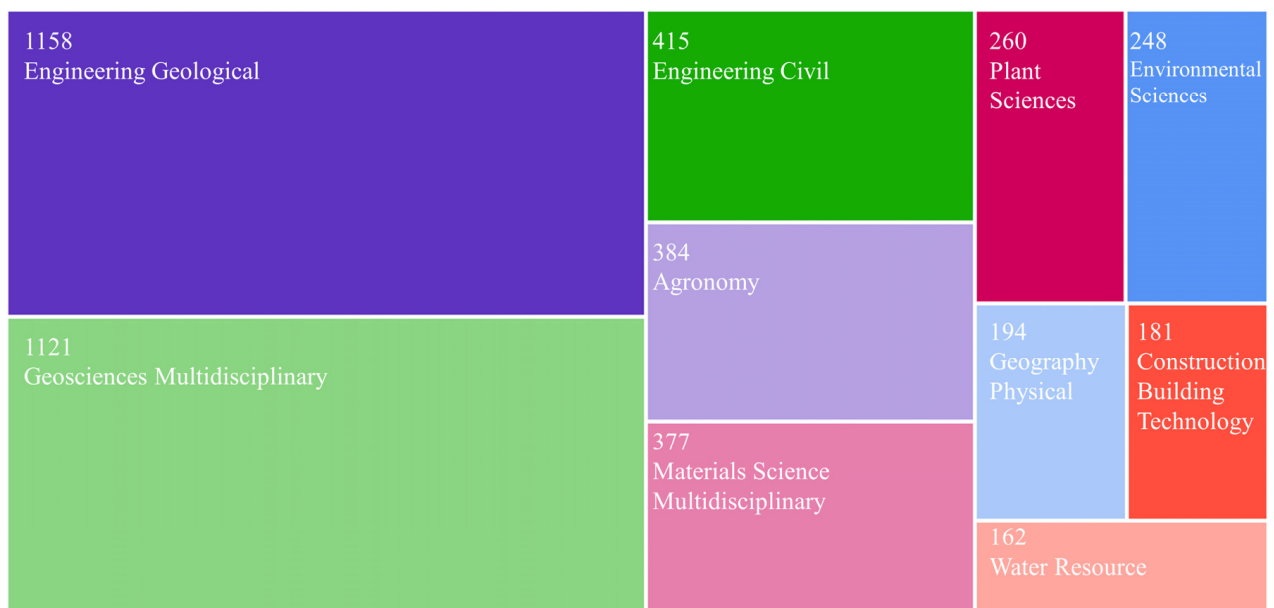


Figure 2. The research areas covered by the retrieved literature.

The paper generated knowledge maps including countries, journals, and keywords. It visually analysed the publishing status, research hotspots, and development trends in this field, providing a reference for researchers to understand the research progress and hotspots in this area.

2.1. Analysis of Annual Publications

As of 25 March 2024, a total of 3907 papers in the field of soil creep have been retrieved. The annual number of publications and the total annual citations are shown in Figure 3. Figure 3 illustrates a gradual increase in the number of papers on “soil” and “creep”. Total annual citations exhibited a generally increasing trend before 2003, stabilized between 2003 and 2020, and then gradually declined thereafter.

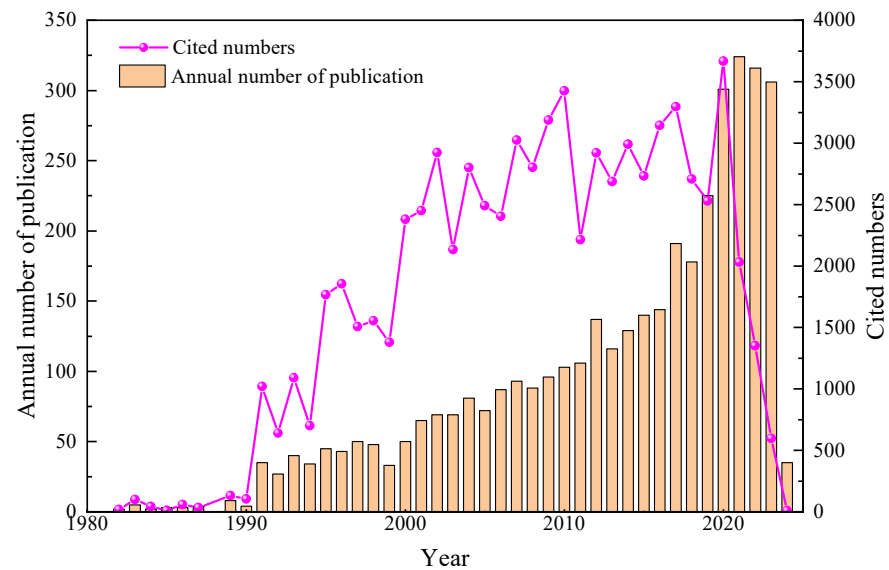


Figure 3. Annual trends in the number of publications and citations in the field of soil and creep.

2.2. Bibliometric Analysis on Countries

This study encompasses the literature from 73 countries and regions. Figure 4 displays a co-occurrence map of countries involved in at least five articles. In Figure 4, the size of the nodes represents the number of publications from each country or region, while the colours reflect the timeline of research, with blue indicating earlier studies and red indicating more recent ones. The connections between nodes represent the collaborative relationships between different countries or regions. Figure 4 reveals that China, the United States, Canada, Japan, France, and the United Kingdom are among the countries with a higher number of publications. Developed countries such as the United States, Canada, and Japan are among the earlier contributors to research.

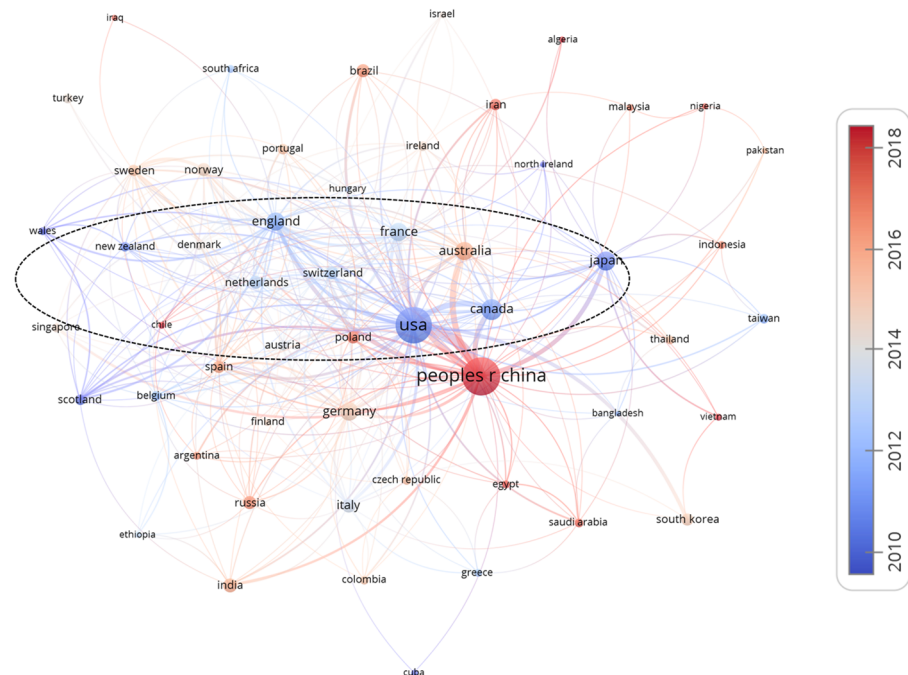


Figure 4. Countries analysis of soil and creep.

Table 1. Keywords of top 20 occurrence frequency.

Rank	Keyword	Weight <Oc- currences>	Weight <Total Link Strength>	Rank	Keyword	Weight <Oc- currences>	Weight <Total Link Strength>
1	creep	562	1554	11	settlement	45	134
2	turfgrass	91	170	12	temperature	43	128
3	consolidation	85	275	13	viscoplasticity	42	140
4	constitutive model	80	191	14	clays	40	159
5	geosynthetics	67	204	15	deformation	40	149
6	sand	54	180	16	frozen soil	38	71
7	creep model	47	82	17	numerical simulation	38	66
8	clay	46	164	18	stress relaxation	38	113
9	creeping bent grass	46	146	19	permafrost	37	87
10	landslide	45	64	20	soft clay	37	105

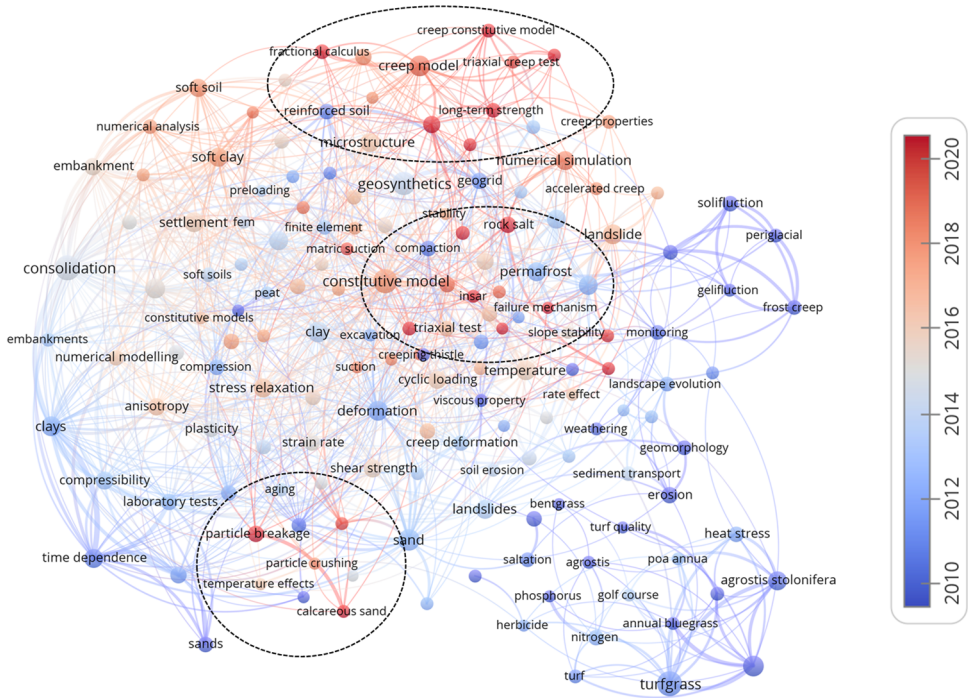


Figure 6. Keywords analysis of soil and creep.

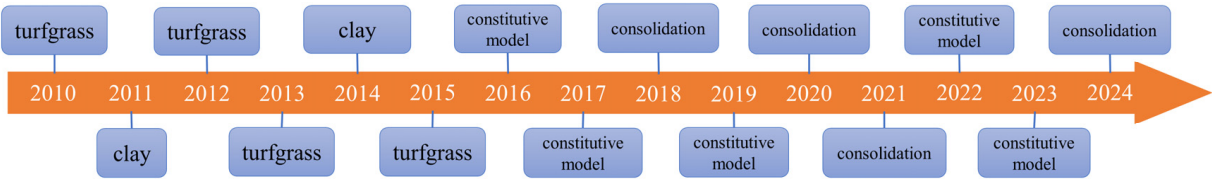


Figure 7. Keywords that appear most frequently each year.

3. Soil Creep Characteristics

Under long-term loading, soil often exhibits creep deformation. Therefore, creep characteristics are considered to be one of the important features of soil. The deformation

process of soil is considered to be a combination of reversible deformation generated by the bending and rotation of individual particles, and irreversible deformation resulting from the relative movement of adjacent particles at contact points. This relative movement between particles is regarded as a rate process. During this creep deformation process, the bonds between particles continuously fracture and recombine [15]. After creep, the number of particles and pores increases, while their diameters decrease, leading to a decrease in porosity and complexity. Currently, common creep tests include one-dimensional consolidation tests, triaxial creep tests, and shear creep tests, among others. The loading methods primarily involve single loading and step loading. Given the irreproducibility of soil sample structures, step loading is often adopted. Subsequently, analyses are conducted using the Boltzmann superposition principle or methods such as the “Chen method [16]” to obtain the strain–time relationship under different loading conditions. Based on different creep rates, creep can be divided into three stages: instantaneous creep, steady-state creep, and accelerated creep, as shown in Figure 8. In one-dimensional consolidation tests, due to the influence of lateral confinement, only instantaneous creep and decay creep occur in the soil sample. However, in triaxial shear tests, steady-state creep or even accelerated creep may occur. During the instantaneous creep stage, the deformation rate gradually decreases; during the steady-state creep stage, the deformation rate stabilizes at a fixed value; and during the accelerated creep stage, the creep rate gradually increases, indicating the failure of the soil sample.

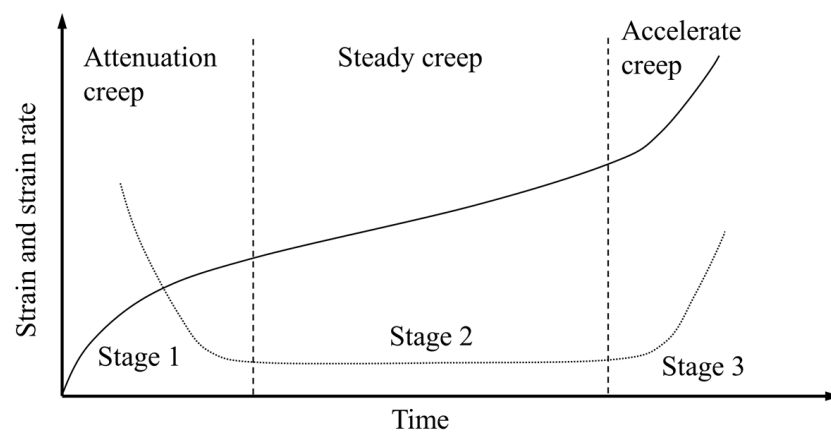


Figure 8. Creep stages of soils.

Taylor and Merchant [17], were the first to consider the secondary consolidation of soils, viewing it as the plastic adjustment of soil structure, and they provided an initial mathematical treatment of it. To better understand creep behaviour, Buisman observed in 1963 that the deformation of soils during the secondary consolidation phase in one-dimensional consolidation tests followed a linear relationship with the logarithm of time, thereby introducing the concept of the secondary consolidation coefficient, C_a .

$$C_a = \frac{e_1 - e_2}{\lg t_2 - \lg t_1}$$

In the equation, t_2 represents the end time of the test, t_1 denotes the primary consolidation time; and e_1 and e_2 represent the void ratios at times t_1 and t_2 , respectively. The secondary consolidation coefficient is dependent on the end time of primary consolidation and is significantly influenced by it. Casagrande proposed a widely adopted method for distinguishing primary and secondary consolidation based on one-dimensional experiments. However, this method has subjective judgmental drawbacks, leading many scholars to propose alternative methods. Yu et al. [18], starting from the soil constitutive relationships, and based on the principle that pore pressure dissipates at the end of primary consolidation and that the entire load is borne by the soil skeleton, which equates to total stress being equal to effective stress, proposed that secondary consolidation begins when

the constitutive curve is linear. The Terzaghi theory suggests that deformations after the inflection point of the logarithmic time-deformation curve are primarily due to secondary consolidation deformation. Currently, there are two main hypotheses regarding the relationship between soil creep and primary and secondary consolidation. Hypothesis A considers creep deformation to be a secondary consolidation, occurring after primary consolidation, with no creep deformation during primary consolidation. Hypothesis B suggests that creep occurs throughout the entire deformation phase of the soil. Hypothesis A posits that secondary consolidation begins only after primary consolidation ends, regardless of soil thickness, attributing both primary and secondary consolidation to particle deformation and relative particle sliding. Hypothesis B argues that there exists an adsorbed water film around the soil skeleton, with the viscous action of this film operating throughout the entire consolidation process. During the dissipation of excess pore water pressure in the initial consolidation stage, there are two extreme viewpoints from Hypothesis A and Hypothesis B regarding the occurrence of creep. The predicted strain–time curves associated with hypotheses A and B are shown in Figure 9. Degago et al. [19], summarised experimental research from the previous literature and found that the compression of clay over time aligns well with Hypothesis B. However, for permeable soils such as sand, there is no distinction between Hypothesis A and Hypothesis B [14].

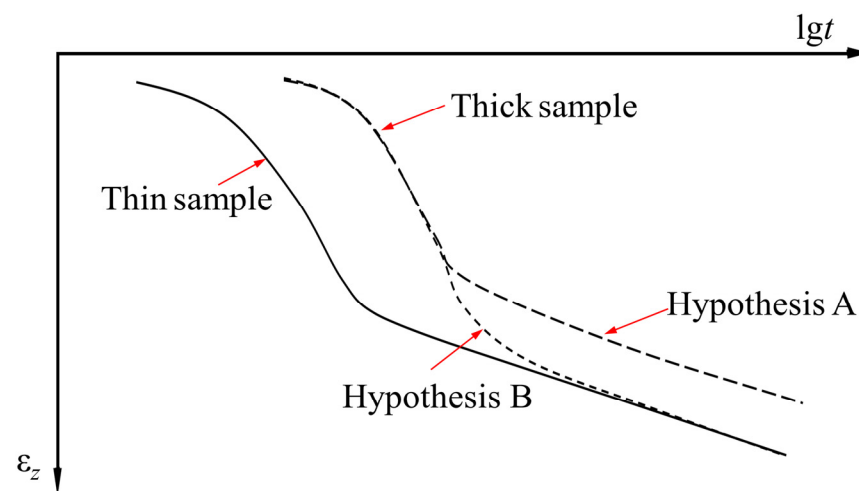


Figure 9. Predicted strain–time curves for hypothesis A and B [14].

4. Creep Influencing Factors

Through bibliometric analysis of the literature on creep deformation, it is evident that the current research focus primarily revolves around exploring soil creep models. Various creep deformations, influenced by soil properties and environmental factors, contribute to the complexity of formulating these models. The following analysis focuses on common influencing factors.

4.1. Microstructural Features

In the early 20th century, foreign scholars proposed the concept of microstructural features of soils, recognizing their significance on macroscopic mechanical behaviour, including particle size, shape, and arrangement. The particle size and shape directly influence soil hardness and viscosity. When the soil is relatively hard, excess pore water pressure dissipates more rapidly, leading to an increase in the consolidation rate [20]. As soil viscosity increases, the creep time gradually lengthens [21]. From a microstructural perspective, cohesive and non-cohesive soils exhibit different deformation mechanisms. Cohesive soils are influenced by viscosity and the soil–water system [22], while non-cohesive soils are mainly affected by particle sliding and fragmentation [23]. For the creep behaviour of soils, Pusch pointed out that its process largely depends on microstructure [24]. During soil deformation, large pores are more prone to change than small pores, and the distribution of

pores gradually becomes more uniform, known as the principle of preferential deformation of large pores. The contours of particles and pores tend to become 'smooth', gradually assuming a flattened circular shape [25,26]. The creep process enhances the random orientation of microcracks in the soil, enlarging their length and diameter, as shown in Figure 10. The orientation of particles, pores, and their interactions significantly affect creep [27]. In soil, the sand content also affects its compressibility. When sand particles disperse within a clay–sand matrix, compressibility remains unaffected by the sand component. However, with an increase in vertical stress, sand particles coalesce to form sand clusters constituting the framework. These sand clusters absorb some stress, resulting in lower stress on the clay matrix than the surface-calculated average stress, thereby overestimating soil compressibility [28]. Moreover, an increase in organic matter content enhances soil creep deformation [29]. Soil structure also influences creep, with remoulded samples exhibiting less creep than intact samples [30]. Additionally, an increased concentration of strongly hydrophilic minerals, such as montmorillonite, in the soil tends to elevate both the water absorption rate and plasticity index, significantly enhancing the relaxation rate and creep characteristics of soil samples [31].

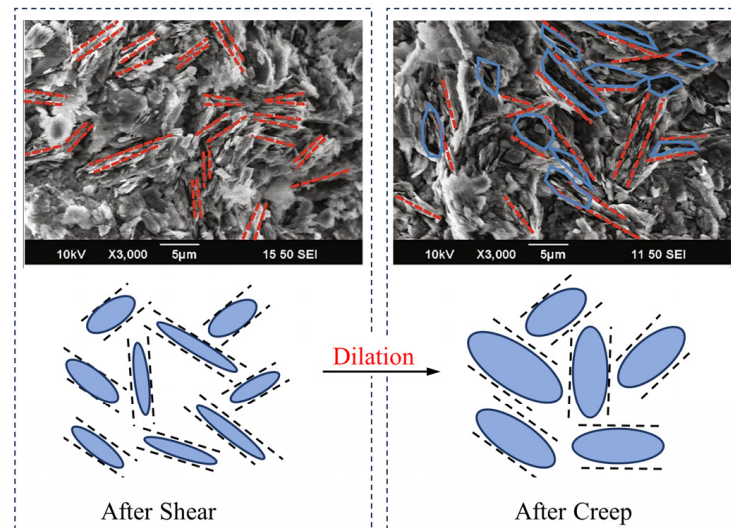


Figure 10. Evolution of pores relates to creep dilation [32].

4.2. Moisture Content

Variations in moisture content alter the soil's saturation, thereby influencing its fundamental mechanical properties. Hicher [33] noted that the viscosity of clay is not only related to particle size distribution and composition, but also to adsorbed water. As the moisture content increases, the strength of the soil gradually decreases [34], and creep deformation increases [35]. The viscosity of the soil originates from the flow of pore water and the viscosity of the solid framework [36]. In the initial stages of creep, the expulsion of pore water generates viscosity [37]. If the soil contains highly absorbent expansive minerals, this high moisture content can cause the expansion of these minerals, leading to pore shrinkage, especially for particles smaller than 0.1 mm, as illustrated in Figure 11. Wang [38] et al. pointed out that an increase in soil moisture content and porosity leads to a significant decrease in molecular forces between soil particles and bound water. In areas with high groundwater levels, construction vibrations can induce excess pore water pressure. This, in turn, weakens the soil particle structure and exacerbates creep deformation [39]. Conversely, under conditions of long-term groundwater extraction, the deformation characteristics of aquifers are closely related to the groundwater level changes experienced by the aquifer units [6].

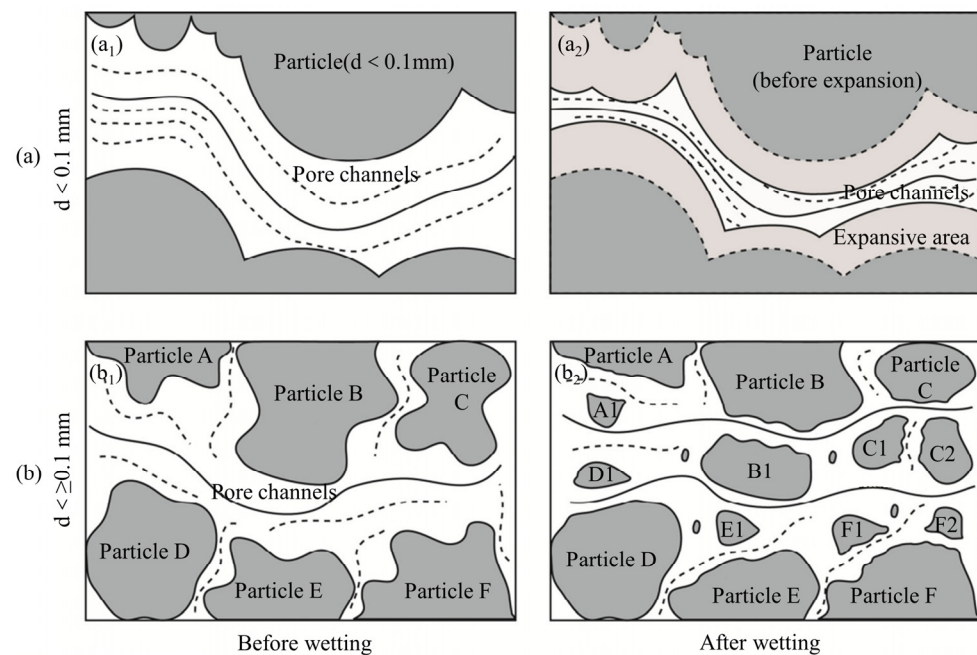


Figure 11. Changes before and after wetting: (a) $d < 0.1 \text{ mm}$; (b) $d \geq 0.1 \text{ mm}$; (a₁, b₁) before wetting; and (a₂, b₂) after wetting [40].

In undrained creep processes, the effective stress on the soil skeleton gradually decreases over time, while the pore water pressure gradually increases. The shear strength of the soil decreases over time, eventually leading to soil failure [41]. Creep deformation is greater in soils with poorer permeability under similar soil and test conditions [42]. For drained creep, the deformation of saturated soil can be divided into deformation caused by dissipation of excess pore water pressure and consolidation compression caused by dissipation of pore water pressure, illustrated in Figure 12. At lower stresses, the deformation under undrained consolidation conditions is significantly smaller than that under drained consolidation conditions. With increasing stress, the soil exhibits viscoplastic behaviour, resulting in the deformation rate under undrained conditions exceeding that under drained conditions. When stress reaches a certain level, the deformation under undrained conditions eventually surpasses the deformation under drained conditions [43]. Augustesen et al. [14] posit that creep occurs as strain gradually accumulates over time in soil while the effective stress remains constant. Drawing from their insights, Xu et al. conclude that experiments conducted under drained conditions represent creep behaviour [9]. Yin [44] categorises the volumetric deformation of one-dimensional drained creep into three stages: the first stage is primarily deformation caused by compression, the second stage is deformation induced by dissipation of pore water pressure, and the third stage is creep deformation following the dissipation of pore water pressure.

In situations of water level fluctuations and rainfall infiltration, soils undergo transitions between saturated and unsaturated states. Some scholars have pointed out that all soils may potentially exist in an unsaturated state [45]. A key characteristic of unsaturated soil behaviour is its volume-change criterion, which is closely related to the yield stress–suction relationship and shear-strength–suction relationship [46,47]. Conducting creep experiments is essential for gaining a deeper understanding of the creep deformation behaviour of unsaturated soils. In terms of experimental design, Guan et al. [48] have proposed specific operational steps, key techniques, and issues to be noted during testing. Matric suction in unsaturated soils, along with conventional cohesive forces, significantly affects soil strength, while the internal microstructure of the soil also influences its suction characteristics. An increase in matric suction enhances soil strength, causing changes in soil volume and porosity. To better describe the creep deformation of unsaturated soils, many

scholars have put forward creep models related to matric suction [49–51]. Additionally, fluctuations in water levels influence the interaction between the flow field and stress field within the soil [52]. Both the number of flow cycles and the pressure of seepage affect soil strength [53]. The movement of pore water directly affects the mechanical properties of the soil composed of pore water and the solid skeleton [37]. With ongoing research into soil deformation, more and more scholars have recognised that precipitation-induced soil deformation includes a certain proportion of creep deformation, which is evident not only in cohesive soils, but also in sands or gravels [54,55]. Continuous seepage leads to soil creep, thereby causing engineering accidents [56]. Under the same deviatoric stress, soil creep increases significantly with the initial excess pore water pressure [57]. Creep also influences excess pore water pressure, with larger creep parameters resulting in higher pore water pressures and greater deformations within the soil [58]. Moreover, higher permeability pressures lead to larger accumulated creep strain and smaller yield stresses [59].

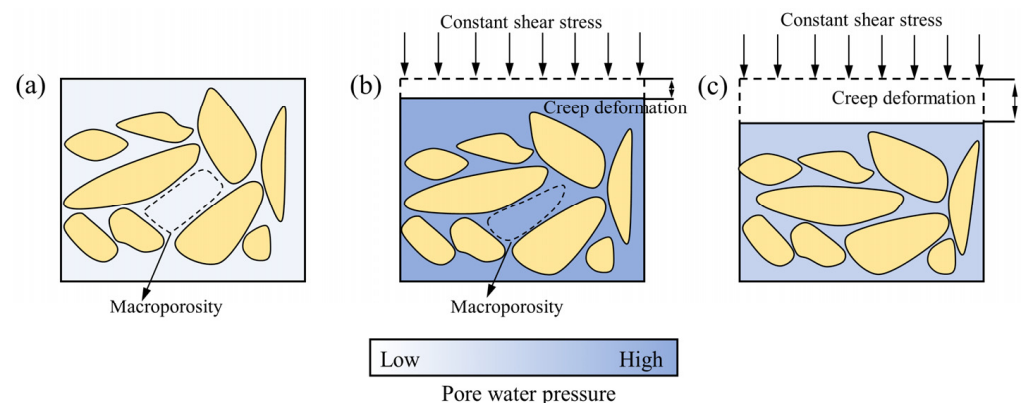


Figure 12. Diagram illustrating the evolution of microstructure during drained triaxial creep: ((a) initial state; (b) increase in pore water pressure; (c) dissipation of pore water pressure).

4.3. Stress Path

In engineering excavation and design processes, the stress state of soil undergoes changes as construction activities progress, experiencing different static stress paths. As illustrated in Figure 13, during slope, excavation, and tunnel construction, there are variations in stress paths at different locations within the soil mass. These differences in stress paths directly impact the mechanical properties of the soil [60]. Ng [61] provided a detailed description of the stress paths of soil elements around deep excavation walls and compared them with actual results, finding significant differences between the in situ stress paths behind the wall and the results of laboratory undrained compression tests. Furthermore, Lade and Duncan [62] found, through experimental studies, that even when the initial and final stresses are consistent, there are noticeable differences in the stress–strain relationship of soil under different stress paths. Therefore, it is particularly important to conduct creep tests in the laboratory to simulate in situ stress paths [63,64]. Wang et al. concluded that the volume strain is greatly influenced by the loading modes of different stress paths during axial loading and lateral unloading of soil [65]. Additionally, Zhu et al. [66] demonstrated, through triaxial unloading creep tests, that under low deviatoric stress, the creep deformation of soil due to unloading can be negligible. However, as the axial unloading stress increases, the creep deformation of soft clay becomes larger, and the non-linear creep characteristics become more pronounced. Apart from axial unloading and radial unloading, there are also cases of cyclic loading in reality, such as traffic loading [67]. For soils subjected to cyclic loading, under undrained conditions, the increase in excess pore water pressure usually leads the soil to a state of over-consolidation [68].

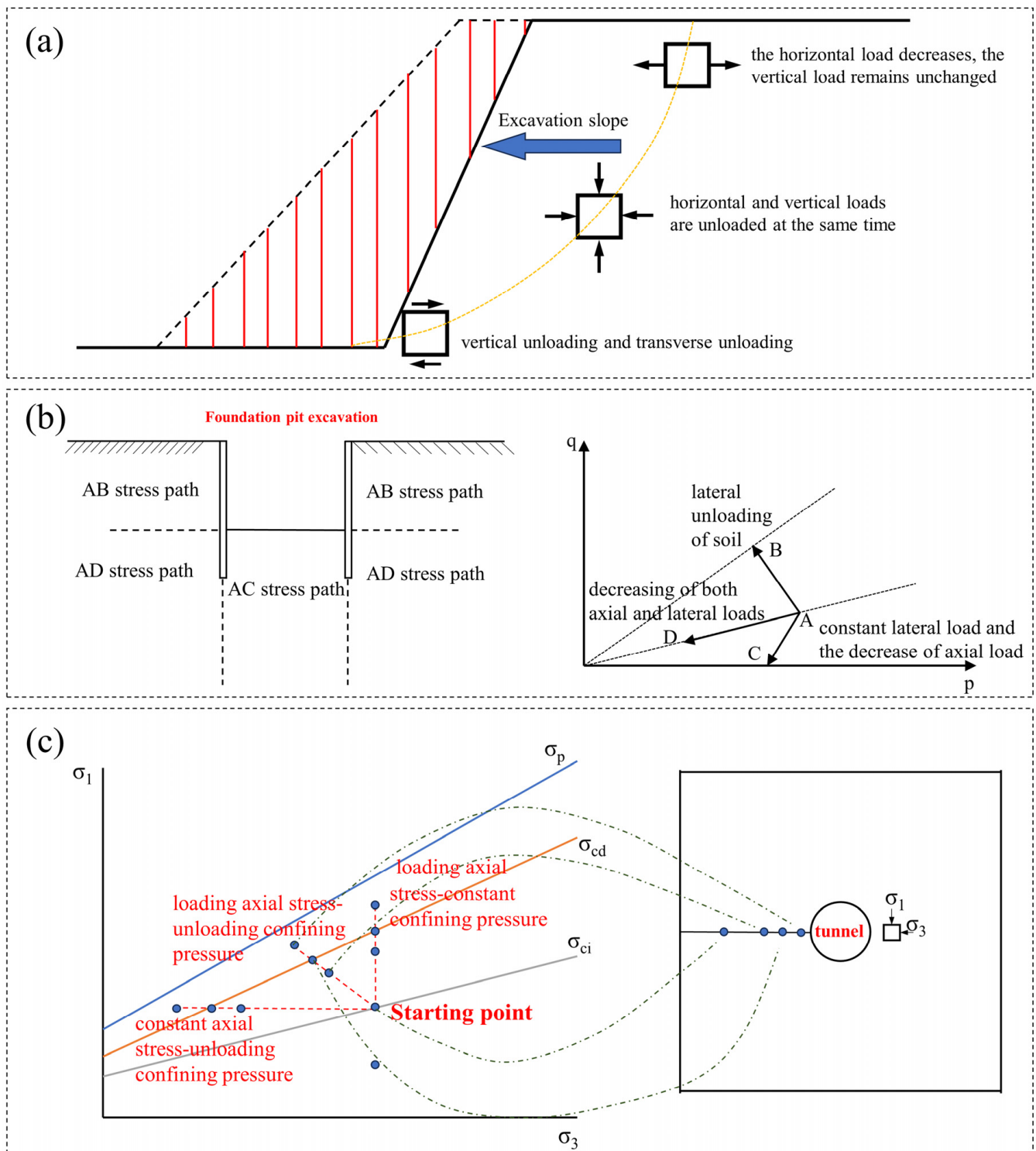


Figure 13. Illustration of different engineering stress paths ((a) slope construction [69]; (b) excavation construction [70]; (c) tunnel construction [71]).

5. Creep Models

In Section 2.4, it is evident that the current focus of research in the geotechnical engineering field primarily centres on the establishment of soil creep models. Soil creep models serve as crucial foundations for predicting soil creep deformation, and play an important role in elucidating the laws of soil creep. Accurate prediction of soil creep deformation behaviour relies on establishing stress–strain–time constitutive relationships. Currently, the widely used soil creep models include empirical models and constitutive element models. The following text summarises the modification methods for the commonly used empirical models and constitutive element models.

5.1. Empirical Models

Empirical models are derived from a synthesis of extensive experimental results and engineering experience, aiming to describe the empirical relationships governing the non-linear rheological behaviour of soil. These models are primarily categorised into stress–strain relationship types and stress–strain rate types, typically composed of power functions, exponential functions, hyperbolic functions, and logarithmic functions. However, logarithmic functions tend to overestimate creep settlement, as creep deformation approaches infinity over time [44]. In order to better capture non-linear creep characteristics, Singh and Mitchell proposed the Singh–Mitchell creep model in 1968. Building upon this model, Lin and Wang developed a creep model considering the over-consolidation ratio [72]. Nonetheless, the Singh–Mitchell model exhibits flaws such as yielding non-zero results when $D1 = 0$ in stress–strain relationships and the absence of extrema in strain–time relationships [73]. Moreover, the Singh–Mitchell model is only applicable to describe strain–time relationships within the 20–80% range of deviator stress levels. In 1981, Mesri improved upon the Singh–Mitchell empirical model by proposing the Mesri creep model, capable of characterising the deformation behaviour of soil at any shear stress level, without the limitation of the 20–80% range. However, both the Singh–Mitchell and Mesri models struggle to simulate all types of soils, necessitating modifications to these models. Presently, the primary forms of modifications to empirical models include adjustments to stress–strain relationships and strain–time relationships, detailed in Tables 2 and 3 for reference.

Table 2. The Mesri model and its corresponding modifications.

Models	Equations	Modified Models	Model Additional Parameters	Testing Type and Conditions	Correction Methods
Mesri	$\varepsilon = \frac{2}{E_u/S_u} \frac{D_1}{1-R_f D_1} \left(\frac{t}{t_1}\right)^\lambda$	$\varepsilon = \begin{cases} \left[\frac{(\sigma_1 - \sigma_3)_f}{A p_a (s/p_a)^n} \right]_r \frac{D_r}{1-(R_f)_r D_r} \left(\frac{t}{t_0}\right)^{m_1}, 1 \leq t \leq t_1 \\ \left[\frac{(\sigma_1 - \sigma_3)_f}{A p_a (s/p_a)^n} \right]_r \frac{D_r}{1-(R_f)_r D_r} \left(\frac{t_1}{t_0}\right)^{m_1} \left(\frac{t}{t_1}\right)^{m_2}, t_1 \leq t \leq t_2 \\ \left[\frac{(\sigma_1 - \sigma_3)_f}{A p_a (s/p_a)^n} \right]_r \frac{D_r}{1-(R_f)_r D_r} \left(\frac{t_1}{t_0}\right)^{m_1} \left(\frac{t_2}{t_1}\right)^{m_2} \left(\frac{t}{t_2}\right)^{m_3}, t_2 \leq t \end{cases}$	m_1, m_2, m_3, A, n (the parameters of m_1, m_2 and m_3 are the slopes of linear fitting of $\ln \varepsilon$ and $\ln t$ at different creep stages; A and n are defined as material constants).	Unsaturated soil triaxial tests	The strain–time is divided into three stages, taking into account the influence of matric suction on creep [49].
		$\varepsilon = \frac{(\sigma_1 - \sigma_3)_f}{F p_a (s/p_a)^n} \frac{D_1}{1-R_f D_1} \left(\frac{t}{t_1}\right)^m$	F, n (F and n are defined as material constants).	Unsaturated soil triaxial tests	The influence of matric suction on creep has been taken into consideration [74].
		$\varepsilon = \frac{\sigma}{E_u} + \frac{\sigma^2}{E_u(1-R_f D)} \frac{1}{\eta_\alpha} \frac{t^\alpha}{\Gamma(1+\alpha)}$	E_u, R_f, η_α (E_u is the initial tangent modulus; R_f is the failure ratio; η_α is the fractional-order viscosity coefficient).	Consolidated Undrained Triaxial Test	By employing fractional calculus theory to depict the strain–time relationship of soil [75].
		$\varepsilon = \left[\frac{(\sigma_1 - \sigma_3)_f}{A p_a (s/p_a)^n} \right] \frac{D_r}{1-(R_f)_r D_r} \left(\frac{t}{t_0}\right)^a D_r^b$	A, a, b (A defined as material constants; a and b are fitting parameters).	Unsaturated soil triaxial tests	The influence of matrix suction on the elastic modulus and the effect of stress on λ have been taken into account [76].
		$\varepsilon = \varepsilon_0 + \frac{2}{E_u/S_u} \frac{D_1}{1-R_f D_1} \frac{t}{t+T^*}$	ε_0, T^* (ε_0 is the initial strain; T^* is the fitting parameter).	Triaxial consolidation drained creep test	In the second stage of deformation, the combination of stress–strain relationship and strain–time relationship is described by a hyperbolic function [77].
		$\frac{s}{\sigma_1 - u_a} \varepsilon = \varsigma \left[\frac{\sigma_r}{E_d} \right]_1 \left(\frac{t}{t_1}\right)^{\partial_1 s^{\partial_2}} + \eta (R_f)_1 \varepsilon$	$\varsigma, \eta, \partial_1, \partial_2$ (ς and η are the coefficients; ∂_1 and ∂_2 are the corresponding fitting parameters).	Unsaturated soil triaxial tests	A modified creep model considering the unsaturated stress levels has been established [51].
		$\varepsilon = \frac{2}{((1-\omega)E_{i1} + \omega E_{i2})/S_u} \frac{\bar{D}_1}{1-R_f \bar{D}_1} \left(\frac{t}{t_1}\right)^\lambda$	ω, E_{i1}, E_{i2} (ω is the damage ratio; E_{i1} and E_{i2} are the initial tangent moduli for stages 1 and 2, respectively).	Triaxial creep test	The inclusion of structural damage variables into the model has facilitated the development of a rheological model that accounts for structural influences [78].

Table 3. The Singh–Mitchell model and its corresponding modifications.

Models	Equations	Modified Models	Model Additional Parameters	Testing Type and Conditions	Correction Methods
Singh-Mitchell	$\varepsilon = Ae^{\beta D} \left(\frac{t}{t_1}\right)^m$	$\varepsilon = \varepsilon_0 + \frac{CD}{\sigma - bD} \left(\frac{t}{t_1}\right)^n$	C, b, n (C, b , and n are constant coefficients).	Triaxial drained and undrained creep tests	A creep model has been established with the stress–strain relationship described with a hyperbolic function, and the strain–time relationship described with a power function [79].
		$\varepsilon = \frac{\sigma}{\bar{a} + b\sigma} \left(\frac{t}{t_1}\right)^{\frac{1+c\sigma}{d+e\sigma}}$	$\bar{a}, \bar{b}, c, d, e$ ($\bar{a}$ and $\bar{b}$ are normalised hyperbolic fitting parameters; c, d , and e are fitting parameters).	One-dimensional consolidation creep test	A creep model has been developed with the stress–strain relationship modelled using a hyperbolic function, and λ described with a hyperbolic function [80].
		$\varepsilon = a + \frac{A}{1-(\theta+\xi D)} e^{\alpha D} t^{1-(\theta+\xi D)}$	a, θ, ξ, α (ξ and θ are the slope and the intercept of the fitting straight line; a and α are constant parameters).	Triaxial creep test	Considering the influence of deviatoric stress on the parameter “ m ” [81].
		$\varepsilon = A(e^{\beta D_r} - 1) \frac{t}{t+T}$	T (T is the pending time parameter).	Triaxial consolidated undrained creep test	Improved the exponential function component and the power function component [73].
		$\varepsilon = Ae^{\beta D} \frac{t}{t+T}$	T (T is the constant coefficient).	Triaxial creep test	The time was corrected using a hyperbolic model [82].
		$\varepsilon = Abe^{A\beta D} \left(\frac{t}{t_1}\right)^{\lambda A}, A = \frac{(\sigma_1 - \sigma_3)_f - \Delta\sigma}{(\sigma_1 - \sigma_3)_f}$	A, b, λ, β (A is the constant coefficient; b is the constant coefficient; λ is the slope of $\lg \varepsilon - \lg t$; β is the slope of $\ln \varepsilon - D$).	Triaxial creep test	A creep model considering unloading disturbance was established [83].
		$\varepsilon = \begin{cases} Be^{\beta D_r} t^{\lambda_1}, 0 \leq t \leq t_1 \\ Be^{\beta D_r} t_1^{\lambda_1} \left(\frac{t}{t_1}\right)^{\lambda_2}, t_1 \leq t \leq t_2 \\ Be^{\beta D_r} t_1^{\lambda_1} \left(\frac{t_2}{t_1}\right)^{\lambda_2} \left(\frac{t}{t_2}\right)^{\lambda_3}, t_2 \leq t \end{cases}$	$t_1, t_2, \lambda_1, \lambda_2, \lambda_3$ (t_1, t_2 are the times at the end of stages 1 and 2; $\lambda_1, \lambda_2, \lambda_3$ are the slopes of $\lg \varepsilon - \lg t$ for stages 1, 2, and 3, respectively).	Triaxial drained creep test	A segmented creep model was established by fitting the three stages of attenuation, steady flow, and rapid flow creep [84].

The aforementioned creep models primarily address the vertical deformation of soil. During undrained creep tests, the soil volume remains constant, whereas during drained creep tests, the soil volume changes. Currently, the volumetric creep rate is mainly calculated using empirical models. For example, Li et al. [85] proposed a relationship between the volumetric creep rate and the secondary consolidation coefficient: $\dot{\varepsilon}_v^t = \frac{C_\alpha}{\ln 10(1+e_0)t_v}$, where $\dot{\varepsilon}_v^t$ represents the volumetric strain rate, C_α is the secondary consolidation coefficient, t_v is the volumetric creep time, and e_0 is the initial void ratio. Based on laboratory test results on rockfill materials, Li et al. [86] derived a volumetric creep formula fitted by a power function: $\varepsilon_v = a\left(\frac{t}{t_0}\right)^b$, where a is the creep of the sample at $t = t_0$, and b is the slope of the fitted curve. Acharya et al. [87] proposed a volumetric creep rate formula for the creep deformation stage: $\dot{\varepsilon}_v = \beta e^{\alpha p'} \left(\frac{t_i}{t}\right)^m$, where β is the vertical coordinate value of the strain rate at the start of creep, t_i is the reference time, m is the slope of the $\log \dot{\varepsilon}_v - \log t$ curve, and α is the slope of $\log \dot{\varepsilon}_v - p'$. Additionally, Butterfield et al. [88] introduced the concept of logarithmic strain and established a volumetric strain expression: $\varepsilon_v = k^* \ln \frac{\sigma'_t}{\sigma'_{p0}} + (\lambda^* - k^*) \ln \frac{\sigma'_{pc}}{\sigma'_{p0}} + \mu^* \ln \frac{\tau_c + t'}{\tau_c}$, where σ'_{p0} and σ'_{pc} represent the pre-loading state and the corresponding consolidation pressure at the end of consolidation, respectively, and k^* , λ^* , and μ^* are constant parameters, t' is the effective time, and τ_c is the time scale parameter. However, this formula is valid under constant effective stress, and is not applicable to instantaneous and continuous loads. To address this issue, Vermeer et al. [88] proposed a volumetric creep rate expression: $\dot{\varepsilon}_v = k^* \frac{\dot{\sigma}'}{\sigma'} + \frac{\mu^*}{\tau} \left(\frac{\dot{\sigma}'}{\sigma'_p}\right)^{(\lambda^* - k^*)/\mu^*}$, where τ provides the time scale and additional degrees of freedom in the soil creep model.

5.2. Component Models

Currently, common fundamental component models used to describe soil creep behaviour include three types of elements: spring, dashpot, and friction elements (Figure 14). Springs are typically employed to represent the elastic deformation of soil, dashpots adhere to the principles of Newtonian fluid flow to describe soil viscosity, and friction elements are used to characterise the plastic behaviour of soil. However, the elements in these models are often linear, and cannot adequately capture the non-linear changes in soil behaviour. In order for models composed of components to accurately describe soil creep deformation, numerous empirical studies have shown that the more components there are, the greater the curvature of the equations. This allows for the theoretical representation of rheological properties to more accurately reflect the true characteristics of the rock and soil medium. However, this also necessitates determining a greater number of rheological parameters.

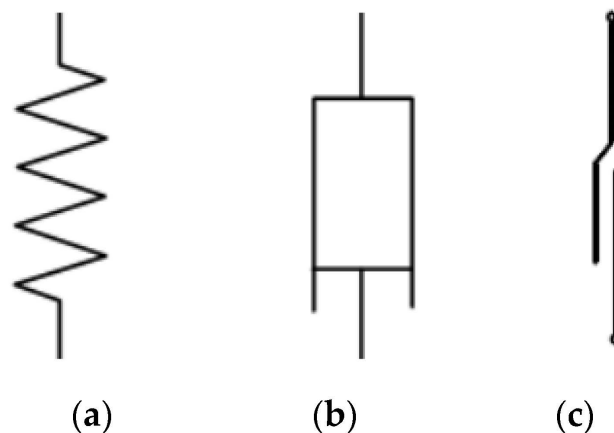


Figure 14. The fundamental rheological elements. (a) Spring element. (b) Dashpot element. (c) Friction element.

To better describe the non-linear characteristics of soils, some scholars have made modifications to the elements. The modifications of the three basic elements mainly focus on the spring and dashpot elements. As shown in Table 4, modifications to the spring element mainly centre around the elastic modulus, considering factors such as stress, time, damage, matric suction, and stress state that influence the elastic modulus. The modification approach typically involves obtaining the relationship between the elastic modulus and relevant influencing factors through experiments, thereby adjusting the spring element. As seen in Table 5, modifications to the dashpot element are more complex, involving adjustments not only to the elastic modulus, but also to the order of strain dashpot. Fractional-order dashpots are commonly utilised in modifications because fractional calculus, an extension of classical calculus, enables the description of dashpots and integrals of arbitrary orders. Due to its ability to describe complex memory processes in engineering practice, it can effectively capture historical dependencies. Applying fractional-order dashpots can reduce errors in classical models used to describe viscoelastic and viscoplastic materials. This approach offers several advantages, including clear physical interpretation, high computational accuracy, and fewer model parameters. In the development of new element models, fractional-order dashpots play a significant role, with the most commonly used being the Riemann–Liouville and Caputo fractional-order dashpots. Although fractional-order dashpots can effectively describe soil creep with fewer parameters, their global characteristics and computational costs, as well as memory requirements, are high. In contrast, fractal dashpots are local operators that can enhance computational efficiency.

Table 4. An overview of the modifications to the spring elements is outlined.

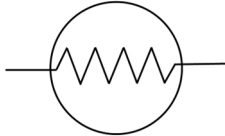
Elements	Modified Elements	Soil	Testing Type and Conditions	Constitutive Relationships	Element Characteristics
 $\sigma = E\epsilon$		Soft clay	Triaxial creep test	$\sigma = E_m \frac{1}{a' + b'(1+t)^{c'}} \epsilon$	Suitable for triaxial creep of clay at low and high stress levels [89].
		Silty clay	Triaxial drained creep test	$\sigma = E(\sigma, t) \epsilon = \frac{E_0 \epsilon}{1 + a(1+t)^b}$	The elastic modulus is a function of time and stress decay [90].
		Loess	Triaxial creep test	$\sigma = E \left[1 + \delta + \delta \left(1 - \frac{t}{t_f} \right)^\alpha \right] \epsilon$	Consideration has been given to the influence of soil damage on the elastic modulus [91].
		Loess	Unsaturated triaxial test	$\sigma = E(S, t) \epsilon = (a \ln(t) + b) S + ct^d$	The influence of matrix suction on the elastic modulus has been taken into account [92].
		Laterite Mixed with Cement	Cyclic loading creep test	$\sigma = G_0 e^{[-\lambda (\frac{N}{f})^\lambda]} \epsilon$	The effect of cyclic loading on the elastic modulus has been considered [93].
		Soft clay	Shear creep test	$\epsilon = a \left(\frac{\sigma}{b} \right)^n$	Describing nonlinear instantaneous elastoplastic deformation [29].

Table 5. An overview of the modifications to the dashpot element is outlined.

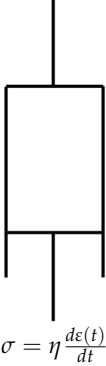
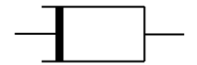
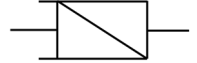
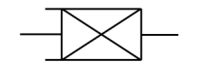
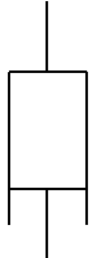
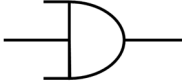
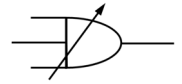
Elements	Modified Elements	Soil	Testing Type and Conditions	Constitutive Relationships	Element Characteristics
		Frozen loess	Triaxial Creep Test	$\varepsilon = \frac{\sigma}{\eta'(H,D)} t = \begin{cases} \frac{\sigma}{\eta^s \left[1 + \left(1 - \frac{2}{\sigma^{2-c}} \right) A \right]} t, & q \leq \sigma_u \\ \frac{\sigma}{\eta^s \left[1 + \left(1 - \frac{2}{\sigma^{2-c}} \right) A \right]} e^{-Bt}, & q > \sigma_u \end{cases}$	Introducing hardening and damage variables to consider the effects of hardening and damage [94].
		Frozen soil	Uniaxial creep tests	$\varepsilon = \frac{\sigma' - \sigma_\infty}{\eta_2(t, \sigma')} = \begin{cases} \frac{\sigma - \sigma_\infty}{\eta_{20} \left[1 + \frac{2\sigma_\infty}{\sigma} \left(1 - e^{-t/t_0} \right) \right]} & \sigma < \sigma_\infty \\ \frac{\sigma e^{t/R} - \sigma_\infty}{\eta_{20} \left[1 + \frac{2\sigma_\infty}{\sigma} \left(1 - e^{-t/t_0} \right) \right]} & \sigma \geq \sigma_\infty \end{cases}$	The stress was adjusted through the damage variable; the viscosity coefficient was adjusted through time and stress [95].
		Laterite Mixed with Cement	Triaxial Creep Test under cyclic loading	$\sigma = \eta_0 \left(\frac{N}{f} \right)^{1-n} \frac{d\varepsilon(t)}{dt}$	A viscosity coefficient related to the number of cycles has been established [93].
		Soft clay for backfilling	Triaxial Consolidated Undrained Creep Test	$\sigma = E \left(\frac{\eta}{E} \right)^\beta D^\beta [\varepsilon], \quad (0 \leq \beta \leq 1)$	Fractional-order dashpots have been introduced to establish elements that lie between ideal solids and ideal fluids [96].
		Ancient ruins	One-dimensional Consolidation Creep Test	$\sigma = \eta \frac{d^\beta \varepsilon(t)}{dt^\beta}, \quad (0 \leq \beta \leq 1)$	Fractional-order dashpots have been introduced to establish elements that lie between ideal solids and ideal fluids [97].
		Loess	Triaxial Creep Test	$\sigma^{2-c} = \frac{\eta(1-c)}{t^{1-c}} \varepsilon$	Based on the fact that the viscosity coefficient during soil creep is a decreasing function of stress and an increasing function of time, a viscosity coefficient considering stress and time has been established [69].
		Silty clay	Triaxial Creep Shear Test with Drainage	$\sigma^{2-c} = \eta_0 t^c \frac{d^\beta \varepsilon(t)}{dt^\beta}, \quad (0 \leq \beta \leq 1)$	During the accelerated creep stage, the viscosity coefficient also increased with time. Based on this, a variable viscosity coefficient element has been established [90].

Table 5. Cont.

Elements	Modified Elements	Soil	Testing Type and Conditions	Constitutive Relationships	Element Characteristics
 $\sigma = \eta \frac{d\varepsilon(t)}{dt}$		All types	Utilising data from other scholars	$\sigma_f = \eta \frac{d\varepsilon_f}{dt^p}$	For the first time, an attempt was made to apply fractal dashpot to simulate viscoelastic behaviour, reducing computational costs and memory storage requirements [98].
		Soft interlayer	Ring shear creep test	$\varepsilon = \frac{\sigma t^p}{\eta e^{-mt}}$	Based on the theory of fractal dashpot, a non-steady-state fractal derivative creep model was established [99].
		Utilising data from other scholars	Utilising data from other scholars	$\varepsilon = \frac{\sigma}{\eta} \left[(1 - \gamma) + \frac{t^\gamma}{\Gamma(\gamma)} \right]$	A fractional-order buffer with Atangana–Baleanu dashpot was established, possessing viscoelasticity at any given moment [100].

6. The Issue and Prospects of Soil Creep

The current research on soil creep has accumulated a wealth of findings that provide significant guidance for engineering. However, several unresolved issues are still faced in gaining a deeper understanding of soil creep deformation. Therefore, in the field of soil creep research, the following aspects should be focused on:

- (1) The changes in soil microstructure are crucial for explaining the mechanism of creep deformation. Given the comprehensive influence of factors such as moisture content, loading, stress path, temperature fluctuations, freeze–thaw cycles, and rainfall, the change in its microstructure is extremely complex. Currently, electron microscopy and optical microscopy are commonly used to observe the microstructure of soil sections, but these methods suffer from limitations in observation range and representativeness [7]. Therefore, in-depth research on changes in soil microstructure will be an important basis for understanding the mechanism of soil creep.
- (2) The creep models proposed at present often target specific soils and influencing factors. However, research on soil creep under the combined action of multiple factors is still lacking. This gap makes it difficult to match creep models with actual engineering sites. Therefore, it is necessary to comprehensively consider multiple factors and explore creep models that incorporate these factors to more accurately predict soil creep behaviour.
- (3) Finite element software, as an important tool for simulating engineering deformation, can embed creep models to simulate changes in engineering construction effectively. However, many proposed models feature complex relationships between parameters and stress, posing challenges for their implementation in finite element software. In addition, fitted parameters, such as elastic modulus, often differ significantly from the actual elastic modulus of the soil. Therefore, developing creep models that accurately fit data and seamlessly integrate into finite element software is crucial for precise predictions of engineering deformations.

7. Conclusions

The relevant information regarding soil creep analysis has been described throughout this entire paper. Moreover, this paper discusses the latest advancements in understanding the influencing factors of soil creep, model modification methods, and future research directions. The main conclusions drawn from this paper are as follows:

- (1) Using VOSviewer visualization software, a comprehensive analysis of the literature in the field of soil creep has been conducted, including aspects such as quantity, citation frequency, institutions, countries, and keywords. The results indicate a trend of increasing literature quantity in the field of soil creep followed by a decrease, with developed countries such as Europe and America having an earlier start in creep research compared to countries like China. Currently, the hotspots in soil creep research mainly focus on soil creep mechanisms and predictive models.
- (2) Soil creep is influenced by multiple factors. Currently, soil creep mechanisms and creep models are mainly proposed based on individual factor effects, with little consideration for the influence of multiple factors on creep.
- (3) Presently, modifications to empirical models primarily focus on adjustments to stress–strain relationships, strain–time relationships, and related parameters. As for component models, modifications primarily concentrate on spring elements and viscous pot elements, usually by introducing other factors through the modification of elastic modulus and viscosity coefficients.
- (4) Future research in soil creep deformation is expected to focus on understanding creep microevolution, multi-factor creep models, and creep models embedded in finite element software.

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