Experimental study on the prediction model of the cumulative strain of the soil–rock mixture backfill under metro train load

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Abstract: An indoor dynamic triaxial test was performed on the soil–rock mixture (SRM) backfill in Chongqing to study the cumulative strain characteristics of the SRM under the cyclic dynamic loading of the metro train. The effects of different consolidation degrees \( U \), rock block contents \( P \), and effective consolidation confining pressures \( \sigma_j \) were then analyzed. Results showed that when \( \sigma_j \) and \( P \) are the same, the lower the \( U \), the faster will be the increase in cumulative strain. When \( U \) and \( P \) were the same, the lower the \( \sigma_j \), the greater will be the increase in cumulative strain and the higher will be the cumulative strain stable value of the stability curve. When \( U \) and \( \sigma_j \) were the same, the lower the \( P \), the faster will be the increase in cumulative strain. An empirical model of the cumulative strain of the SRM considering the influence of \( U \), \( P \), and \( \sigma_j \) was established on the basis of the results of the dynamic triaxial tests. The accuracy of the prediction model was verified against the actual test results. This study provides a theoretical basis for the settlement and deformation evaluation of SRM backfill subgrades under metro train cyclic loading.

1. Introduction

Metro lines, as a modern form of large-scale public transport with high efficiency and passenger-carrying capacity, is often constructed in large- and medium-sized cities, especially those in mountainous areas, to alleviate severe traffic pressure. However, the expansion of mountainous cities has led to the formation of a large number of soil–rock mixtures (SRMs) comprising rock blocks of different sizes and fine-grained matrices [1-4]. When constructing extensive metro projects, loose-structured, low-strength, highly porous, and poorly engineered areas with SRM backfill are inevitably observed [5-7]. During the operation of the metro, long-term cyclic dynamic loading of metro trains could cause cumulative strain and uneven settlement in the backfill area of the SRM, which would affect the stability of the foundation and safety of the trains [8-10]. Therefore, conducting research on the cumulative strain characteristics of SRMs under the cyclic loading of a metro train is significant for ensuring the operational safety and settlement control of metro tunnels.

Many scholars have used different types of SRMs as research objects to study the factors influencing cumulative strain. Researchers [11-16] have conducted several dynamic triaxial tests to study the cumulative strain characteristics of SRMs under cyclic loading and analyze the influence of confining pressure, dynamic stress amplitude, drainage conditions, water content, and initial density on the cumulative strain of the coarse-grained soil. Araei et al. [17-18] studied the effects of loading rate and...
load waveform on the stress–strain characteristics of high-strength rockfill materials using large-scale dynamic triaxial equipment. Ohiduzzaman et al. [19] studied the effect of fine particle content in unsaturated soil on the deformation characteristics of unbonded granular base materials under cyclic axial and radial stresses. Duong et al. [20] studied the effects of three water contents and four fine-grained particle contents on the cumulative strain of coarse-grained soil intercalated in the underground structure of an early French railway. The group’s research results showed that higher fine-grained particle contents in the saturated soil filler promote the growth rate of axial cumulative strain and lead to a higher final axial cumulative strain; by contrast, the opposite effects are observed in unsaturated soil. Sun et al. [21] studied the effect of different loading frequencies of 5–60 Hz on the cumulative strain of coarse-grained soil. Studies have shown that plastic shakedown may occur when \( f < 20 \text{ Hz} \), plastic shakedown and ratcheting may develop when \( 30 \text{ Hz} < f < 50 \text{ Hz} \), and plastic collapse may occur at higher frequencies \( (f > 60 \text{ Hz}) \). Deng et al. [22] used a simple theoretical energy method to study the dynamic responses of soft clay under cyclic loading under dynamic stress amplitude, confining pressure stress, initial stress and loading frequency which could discuss its deformation characteristics, and then tested its effectiveness using cyclic triaxial tests. Jiang et al. [23] combined three different test temperatures, three confining stresses, three loading frequencies, and seven vertical stresses to study the effects of multiple load tests on the permanent deformation of asphalt mixtures. Hou et al. [24] studied the axial strain of deep silty clay before and after freezing–thawing considering the effects of vibration time, vibration frequency, freezing temperature, number of freeze–thaw cycles, and other factors.

Many scholars have established a series of constitutive cumulative strain models, but these models often require massive calculations under a large number of loading cycles \( N \). The available theoretical models are difficult to generalize and, as a result, a large number of empirical models suitable for different engineering practices have been proposed. Monismith et al. [25] introduced an empirical model for predicting the subgrade soil cumulative strain index considering \( N \). Many authors have developed improved models based on Monismith’s model and the related experiments. For example, Chai and Miura [26] proposed a model considering the effects of initial static, dynamic, and static failure deviator stresses. Puppala et al. [27] discussed the effects of compacted water content, confining pressure, and partial stress on the measured values of the permanent deformation of clay, silt, and sand. The permanent strain model has been used to describe the relation between the stress state of the soil and \( N \). Bian et al. [28] proposed a unified prediction model of the cumulative deformation of materials used in railway construction under cyclic loading; in this model, the parameters were corrected via physical model tests. Lei et al. [29] considered the effects of stress amplitude, loading frequency, and freeze–thaw temperature on the cumulative strain of soft clay and proposed an empirical cumulative plastic strain model that could comprehensively reflect the influence of these factors.

At present, most studies on the cumulative strain characteristics of SRM take coarse-grained fillers under the railway or road as research objects. These materials are mainly composed of coarse-grained soil with low fine-grained soil contents. However, little research has been performed on the cumulative strain characteristics of SRMs with fine-grained soil as the main component. Therefore, the present study examines the metro tunnel project of a section of the Chongqing Metro Line 10 crossing the SRM backfill area and uses a dynamic triaxial apparatus to perform cyclic loading tests. The influences of the consolidation degree \( U \), effective consolidation confining pressure \( \sigma_3 \), and rock block content \( P \) on the cumulative strain of the SRM are determined, and a cumulative strain model considering these factors is established.

2. Soil–rock mixture cumulative strain test

2.1. Physical and mechanical parameters of the soil–rock mixture

The SRM backfill used in this study was obtained from a section of the tunnel of Chongqing Metro
Line 10 passing through the area of the SRM backfill. The stone block was mainly composed of sandstone and contained only a small amount of mudstone. The fine-grained soil was mainly composed of silty clay. The physical and mechanical properties of the rock and soil in the SRM are shown in Table 1.

| Source of samples | Rock block content (%) | Natural water content (%) | Silty clay | Sandstone |
|-------------------|------------------------|---------------------------|------------|-----------|
| Chongqing, China  | 20                     | 9.22                      | 1.7        | 2.54      |
|                   |                        |                           | 17.09      | 30.16     |
|                   |                        |                           | 13.07      | 2.28      |
|                   |                        |                           | 2.4        |           |

### Table 1. Physical and mechanical properties of the soil–rock mixture backfill

2.2. Sample preparation

Taking a particle size of 2 mm as the threshold value of soil and rock [30], a cylindrical SRM sample measuring $\phi 50 \text{ mm} \times 100 \text{ mm}$ was prepared. According to the requirements of DLT5355-2006 of the Code for Soil Tests for Hydropower and Water Conservancy Engineering [31], the maximum allowable particle size of the sample with a diameter of 50 mm is 5 mm. Therefore, in this paper, the sandstone was replaced with an artificial block with a particle size of 5 mm that had physical and mechanical properties similar to those of the original block. Fine-grained soil was selected from the original silty clay, as shown in Figure 1.

The soil was dried, passed through a 2 mm sieve, sprayed with a suitable amount of water, and stirred evenly. After standing and sealing for one day and night, the soil from three different positions and soil content without difference of ±1% was mixed with artificial stones. The mixture was rammed in five layers to prepare a cylindrical SRM sample measuring $\phi 50 \text{ mm} \times 100 \text{ mm}$, as shown in Figure 2.

The samples were vacuum-saturated for 24 hours to shorten the test time. Then, the samples were installed in the triaxial pressure chamber, the chamber was closed, the inlet valve was opened, and the chamber was filled with water until the water surface passed the top of the samples. A confining pressure was applied using air pressure, and the samples were saturated when the confining pressure had stabilized. With references to the relevant literature on clay saturation, using graded back-pressure saturation, each stage increment was 30kPa and the effective confining pressure of the sample was maintained at 20kPa, the saturation coefficient $B \geq 0.95$ under continuous three-stage load was measured and then consolidated. The samples were isotropically consolidated. Because of the poor permeability of the silty clay and slow consolidation speed, the consolidation time of the sample was at least 24 hours and the variation in drainage volume was within 1 hour when the consolidation was stable and not exceed 0.1 cm$^3$. 
2.3. Experimental apparatus and methods

2.3.1. Experimental apparatus. This experiment used a DDS-70 microcomputer-controlled electromagnetic vibration triaxial test system, as shown in Figure 3. The hardware of this instrument included four major components: a mainframe, an electrical control cabinet, a static pressure control cabinet, and a microcomputer control system. A dynamic triaxial test system integrating test control and data collection was used for software control.

2.3.2. Experimental methods. The sample was not drained during the test to simulate the actual drainage situation in the metro line. During consolidation, isotropic consolidation was adopted, i.e., the consolidation stress ratio was 1. Combined with on-site samples, the \( \sigma_j \) of the test samples was set to 100, 200, and 400 kPa. When \( \sigma_j \) was 200 kPa, \( U \) was set to 0.8, 0.9, and 0.98. The metro’s running speed in the test was 60 kph, the dynamic load was a biased sine wave cyclic load with a frequency of 2.5 Hz [32], the dynamic stress amplitude was calculated as 60 kPa [33], the cyclic dynamic stress ratio was 0.12, and \( N \) was 15000.
2.4 Cumulative strain test program
In this study, a cumulative strain test program considering three factors was used; these three factors were \( U \), \( \sigma_j \), and \( P \). The design of the cumulative strain test program is shown in Table 2. While samples A1–A9 were used to analyze the effects of different \( U \) and \( \sigma_j \) on the cumulative strain of the SRM under the cyclic loading of the metro train, samples A10–A15 were used to study the effects of different \( P \) and \( \sigma_j \) on the mixture.

**Table 2. Test program for cumulative strain**

| Sample number | Rock block content (%) | Consolidation degree | Consolidation stress (kPa) | Cyclic stress ratio | Number of cycles | Frequency (Hz) |
|---------------|------------------------|----------------------|---------------------------|--------------------|-----------------|---------------|
| A1            | 20                     | 0.98                 | 100                       | 0.12               | 15000           | 1             |
| A2            | 20                     | 0.9                  | 100                       | 0.12               | 15000           | 1             |
| A3            | 20                     | 0.8                  | 100                       | 0.12               | 15000           | 1             |
| A4            | 20                     | 0.98                 | 200                       | 0.12               | 15000           | 1             |
| A5            | 20                     | 0.9                  | 200                       | 0.12               | 15000           | 1             |
| A6            | 20                     | 0.8                  | 200                       | 0.12               | 15000           | 1             |
| A7            | 20                     | 0.98                 | 400                       | 0.12               | 15000           | 1             |
| A8            | 20                     | 0.9                  | 400                       | 0.12               | 15000           | 1             |
| A9            | 20                     | 0.8                  | 400                       | 0.12               | 15000           | 1             |
| A10           | 10                     | 0.98                 | 100                       | 0.12               | 15000           | 1             |
| A11           | 10                     | 0.98                 | 200                       | 0.12               | 15000           | 1             |
| A12           | 10                     | 0.98                 | 400                       | 0.12               | 15000           | 1             |
| A13           | 40                     | 0.98                 | 100                       | 0.12               | 15000           | 1             |
| A14           | 40                     | 0.98                 | 200                       | 0.12               | 15000           | 1             |
| A15           | 40                     | 0.98                 | 400                       | 0.12               | 15000           | 1             |

3. Test results

3.1 Effects of consolidation degree and effective consolidation confining pressure on the cumulative strain
Figures 4(a)–4(c) present the cumulative strain curves obtained under different \( \sigma_j \) and \( U \) when \( P \) is 20%. \( U \) and \( \sigma_j \) clearly had significant effects on the development of the cumulative strain. When \( U=0.8 \), the cumulative strain curve of the SRM showed different shapes under different \( \sigma_j \). When the confining pressure was low (e.g., \( \sigma_i=100kPa \)), the sample failed because it exceeded the failure standard, so it was only carried out 350 times. However, when \( \sigma_j \) was high (e.g., \( \sigma_j=200kPa \) or 400 kPa), the cumulative strain curve tended to stabilize with increasing \( N \). When \( U \) was 0.9 and 0.98, the SRM samples under three different \( \sigma_j \) showed a stable state, but their cumulative strains differed. When \( U=0.98 \), the stable cumulative strains of the samples at \( \sigma_j=100kPa, 200kPa, 400kPa \) were 1.113%, 0.770%, and 0.326%, respectively; when \( U=0.9 \), the stable cumulative strains of the samples at \( \sigma_j=100kPa, 200kPa \) and 400 kPa were 1.832%, 1.102%, and 0.568%, respectively. These data show that the stable cumulative strain under the three \( \sigma_j \) tested increases by 0.719%, 0.332%, and 0.242%, respectively, as \( U \) decreases from 0.98 to 0.9. Test results showed that, under a constant \( \sigma_j \), the lower the \( U \), the faster the increase in cumulative strain and the greater the stable cumulative strain. Under a constant \( U \), the lower the \( \sigma_j \), the greater the increase in cumulative strain and the higher the stable value of the cumulative strain of the stable curve. Reducing \( \sigma_j \) and \( U \) was not conducive to the stability of the SRM.
3.2 Effects of rock block content and effective consolidation confining pressure on the cumulative strain

Figure 5 shows the evolution curve of cumulative strain as a function of $N$ under different $\sigma_j$ and $P$ when $U$ is 0.98. When $\sigma_j$ was 100, 200, and 400 kPa, the development trend of the cumulative strain curves corresponding to $P=10\%$, 20\% and 40\% showed three stages, namely, rapid growth, slow growth, and gradual stabilization. When $P=10\%$, the stable cumulative strains corresponding to $\sigma_j=100\text{kPa}$, 200kPa, 400kPa were 1.551\%, 1.084\%, and 0.546\%, respectively. When $P=20\%$, the stable cumulative strains corresponding to $\sigma_j=100\text{kPa}$, 200kPa, 400 kPa were 1.113\%, 0.770\%, and 0.326\%, respectively. When $P$ was increased to 40\%, the stable cumulative strains corresponding to $\sigma_j=100\text{kPa}$, 200kPa, 400kPa were reduced to 0.395\%, 0.209\%, and 0.150\%, respectively. However, when $N$ reached approximately 4200, the rubber membrane of the sample was worn out by the artificial stone at $\sigma_j=200\text{kPa}$, thereby causing the load to stop, but the cumulative strain of the sample remained basically stable and no longer increased because $N$ was 2000 times, which proves the validity of the test results. The test results showed that, under a constant $\sigma_j$, as $N$ increases, the
cumulative strain of the sample with $P=10\%$ increases fastest, followed by that of the sample with $P=20\%$, and then that of the sample with $P=40\%$. In other words, the lower the $P$, the greater the growth rate of the cumulative strain and the greater the stable cumulative strain. Under the continuous action of a cyclic load, the larger the $P$ of a sample, the larger the area of the stone particles in contact with each other, which limits the growth of the cumulative strain. Under a constant $P$, the higher the $\sigma_3$, the greater the increase in cumulative strain and the greater the stable cumulative strain.

![Figure 5](image)

**Figure 5.** Relationship between cumulative strain and number of loading cycles under different rock block contents and effective consolidation confining pressures.

### 4. Establishment and verification of the cumulative strain model

Many factors influence the cumulative strain of SRMs under cyclic loading. Most of the existing cumulative strain empirical models are based on the results of the test, and fitting analysis was carried out according to the test results. Selection of the main factors influencing the cumulative strain of SRMs is a core challenge. The cumulative strain model in this paper mainly considers the effects of $U$, $\sigma_3$, $P$, and $N$. A cumulative strain model was established by comprehensively analyzing the test results obtained above and ignoring the cumulative strain at the moment of loading as follows:

$$\varepsilon = A - \frac{A}{1 + B \cdot N}$$  \hspace{1cm} (1)

where $\varepsilon$ is the cumulative strain, $N$ is the number of loading cycles, $A$ and $B$ are fitting parameters.
related to $U$, $\sigma_j$, and $P$, and $A$ represents the stable cumulative strain.

Equation (1) is used to fit the cumulative strain test results of the sample. The relevant fitting parameters are shown in Table 3, and the fitting degrees obtained are consistently greater than 0.9. This finding indicates that Equation (1) can reflect the factors studied in this paper well.

Table 3. Cumulative strain model fitting parameters

| Rock block content (%) | Effective consolidation confining pressure (kPa) | Consolidation degree | $A$       | $B$       | $B/A$   | $R^2$    |
|------------------------|-----------------------------------------------|----------------------|-----------|-----------|---------|----------|
| 20                     | 100                                           | 0.9                  | 1.113     | 0.00454   | 0.004075| 0.9947   |
| 20                     | 100                                           | 0.9                  | 1.832     | 0.007     | 0.003821| 0.9958   |
| 20                     | 200                                           | 0.9                  | 0.7704    | 0.008     | 0.010384| 0.9668   |
| 20                     | 200                                           | 0.9                  | 1.102     | 0.00909   | 0.008251| 0.9190   |
| 20                     | 200                                           | 0.8                  | 1.358     | 0.01068   | 0.007865| 0.9623   |
| 20                     | 400                                           | 0.9                  | 0.3263    | 0.01101   | 0.033742| 0.9310   |
| 20                     | 400                                           | 0.9                  | 0.5681    | 0.01613   | 0.028393| 0.9651   |
| 20                     | 400                                           | 0.8                  | 0.8351    | 0.01935   | 0.023171| 0.9338   |
| 10                     | 100                                           | 0.9                  | 1.551     | 0.006     | 0.003868| 0.9791   |
| 10                     | 200                                           | 0.9                  | 1.084     | 0.009     | 0.008303| 0.9287   |
| 10                     | 400                                           | 0.9                  | 0.5462    | 0.01297   | 0.023746| 0.9341   |
| 40                     | 100                                           | 0.9                  | 0.3948    | 0.02173   | 0.055041| 0.9400   |
| 40                     | 200                                           | 0.9                  | 0.2094    | 0.05378   | 0.256829| 0.9226   |
| 40                     | 400                                           | 0.9                  | 0.150     | 0.050     | 0.333333| 0.9268   |

5. Analysis of the fitting parameters of the cumulative strain model

The fitting parameters $A$ and $B$ were analyzed to reflect the influence of $U$, $\sigma_j$, and $P$ on these parameters in Equation (1).

5.1. Influences of degree of consolidation and effective consolidation confining pressure on $A$ and $B$

While the cumulative strain development mode of the samples was of the destructive type when $P=20\%$, $U=0.8$, and $\sigma_j=100$ kPa, the cumulative strain development modes of all other samples showed stable cumulative strain development. Considering that the cyclic load of the metro train does not generally exceed the critical cyclic stress ratio, only SRM samples with a stable cumulative strain development mode and $U$ in the range of 0.8–0.98 are analyzed. In this section, the influences of $U$ and $\sigma_j$ on $A$ and $B$ were studied.

5.1.1. Influences of consolidation degree and effective consolidation confining pressure on parameter $A$. The influence of $U$ on $A$ was analyzed using the parameters in Table 3, and a good linear relationship between these properties was obtained:

$$A = a \times U + b$$  \hspace{1cm} (2)

where $a$ and $b$ are fitting parameters related to $\sigma_j$, as shown in Table 4.

Regression analysis of $a$ and $b$ in Table 4 reveals the following relationship between $\sigma_j$, $a$, and $b$:

$$a = -1.6344 \times 10^8 \times \sigma_j^{-3.71} - 2.773 \hspace{1cm} R^2 = 0.9996$$  \hspace{1cm} (3)

$$b = 4.5703 \times 10^6 \times \sigma_j^{-2.91} + 3.008 \hspace{1cm} R^2 = 1$$  \hspace{1cm} (4)

Substituting Equations (3) and (4) into Equation (2) yields the relationship between $A$, $U$, and $\sigma_j$:

$$A = (-1.6344 \times 10^8 \times \sigma_j^{-3.71} - 2.773) \times U + (4.5703 \times 10^6 \times \sigma_j^{-2.91} + 3.008) \hspace{1cm} R^2 = 0.9872$$  \hspace{1cm} (5)

Because $A$ represents the stable cumulative strain, Equation (5) can be used to predict the cumulative strain of the SRM when the strain development mode is stable.
Table 4. Fitting parameters of Equation (2)

| Effective consolidation confining pressure (kPa) | $a$     | $b$     | $R^2$  |
|------------------------------------------------|---------|---------|--------|
| 100                                            | -8.9875 | 9.9207  | 1      |
| 200                                            | -3.2356 | 3.9672  | 0.981  |
| 400                                            | -2.8202 | 3.0959  | 0.9987 |

5.1.2. Influences of consolidation degree and effective consolidation confining pressure on parameter $B$. The effect of $U$ on $B$ is analyzed using the same method described above. The relationship between $B/A$, $U$, and $\sigma_3$ is obtained by regression analysis:

$$B/A = (-3.695 \times 10^7 \times \sigma_3^2 + 0.001) \times (1 - U)^{0.1} + (4.495 \times 10^7 \times \sigma_3^2 + 0.002)$$

$$R^2 = 0.9974 \quad (6)$$

Figures 6(a)–6(c) show the test curves of the cumulative strain as a function of $N$ under different $U$ and $\sigma_3$ when the cumulative strain development mode of the samples is stable. Comparison of the calculation results of Equations (1), (5), and (6) with the test curves reveals that the results calculated by these formulas are close to the test results, thereby indicating that the model can well reflect the influence of the $U$ and $\sigma_3$ on the cumulative strain of the SRM.

5.2. Influences of rock block content and effective consolidation confining pressure on parameters $A$
and B
Because the cumulative strain test under different $P$ in this paper was only carried out at $U=0.98$, the results cannot reflect the combined effect of $P$ and $U$. Therefore, the influence of $P$ on $A$ and $B$ was not analyzed together with $U$. Only the influences of $P$ in the range of 10%–40% and $\sigma'_3$ on $A$ and $B$ were analyzed.

5.2.1 Influences of rock block content and effective consolidation confining pressure on parameter $A$. According to the parameters in Table 3, the relationship between $A$, $P$, and $\sigma'_3$ could be constructed as follows:

$$ A = (0.03 \times \sigma'_3^{0.8} - 4.872) \times P^{0.5} + (-0.031 \times \sigma'_3^{0.75} + 3.685) \quad R^2 = 0.99 \quad (7) $$

Because $A$ refers to the stable cumulative strain, Equation (7) can be used to predict the cumulative strain of the SRM when the strain development mode is stable.

5.2.2 Influences of rock block content and effective consolidation confining pressure on parameter $B$.
The effect of analyzing $P$ on the parameter $B$ is the same as the above method. The relationship between $B/A$, $P$, and $\sigma'_3$ could be obtained by regression analysis as follows:

$$ B/A = (-2.476 \times 10^{5} \times \sigma'_3^{-1.78} + 80.754) \times P^6 + (1.531 \times 10^{-7} \times \sigma'_3^2 + 0.002) \quad R^2 = 0.9996 \quad (8) $$

Figure 7 shows the changes in cumulative strain as a function of $N$ under different $P$ and $\sigma'_3$, when the cumulative strain development mode of the sample is stable. Comparison of the values calculated from Equations (1), (7), and (8) with the test results reveals that these formulas can accurately reflect the influence of $P$ and $\sigma'_3$ on the cumulative strain of the SRM.
6. Conclusion

1. Under a constant $\sigma_j$, the lower the $U$ of the sample, the faster the increase in cumulative strain and the greater the stable cumulative strain. Under a constant $U$, the higher the $\sigma_j$, the slower the increase in cumulative strain and the smaller the stable value of the cumulative strain of the stable curve.

2. Under varying $P$ and $\sigma_j$, the cumulative strain curve shows a rapid increase when $N$ is 0–500 and a slow increase when $N$ is 500–1500. When $N$ is 1500–15000, the cumulative strain curve gradually stabilizes.

3. Under a constant $\sigma_j$, the lower the $P$, the greater the increase in cumulative strain and the larger the stable cumulative strain. Under a constant $P$, the lower the $\sigma_j$, the larger the increase in cumulative strain and the greater the stable value of the cumulative strain of the curve.

4. A model considering the relationship between the cumulative strain of SRM, $P$, $U$, $\sigma_j$, and $N$ was constructed by comprehensively analyzing the test results, and the strain development of the SRM was obtained. The relationships between $A$, $B$, and these factors could be obtained when the cumulative strain model stabilizes.

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