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The influence of scale effect on the deformation parameters of the Duncan-Chang *E*-μ model for coarse-grained soils: an experimental study

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The influence of scale effect on the deformation parameters of the Duncan–Chang E–μ model (hereinafter abbreviated as D–C E–μ M) for coarse-grained soils remains challenging to quantify. These parameters play a critical role in predicting deformations in earth-rock dams, which in turn directly affect the safety and durability of such structures. Therefore, mitigating the impact of the scale effect on the deformation parameters of the D–C E–μ M is essential for the safe design of earth-rock dam projects. Previous studies suggest that variations in the maximum particle diameter d_{max} and the gradation structure are the primary factors contributing to the scale effect. In this study, the influence of scale effect on the mechanical behavior of coarse-grained soils is systematically investigated. Using a continuous gradation equation, 21 sets of specimens with different gradations were prepared by controlling d_{max} and the gradation area S. A series of triaxial consolidated-drained tests were conducted to analyze the effects of d_{max} and S on the deformation parameters of the D–C E–μ M. The experimental results indicate that parameters *G*, *K*, *F*, and Rf decrease as the gradation area *S* increases. In contrast, parameters n and *D* first increase and then decrease with increasing *S*, eventually stabilizing beyond a certain threshold. Empirical relationships between each model parameter and *S* were established. With increasing d_{max}, the parameters R_f, *lgK*, *n*, *F*, and *D* increase, whereas *G* decreases. All parameters exhibit logarithmic relationships with d_{max}. Based on the similar gradation method, an empirical formula is proposed to predict the deformation parameters of the D–C E–μ M under the influence of scale effect. The applicability of this formula to various types of coarse-grained soils is validated using test data from existing literature. Finally, a method is presented for predicting *in situ* deformation parameters of the D–C E–μ M based on scaled laboratory test results using the similar gradation approach.

KEYWORDS

coarse-grained soil, scale effect, particle gradation, triaxial compression test, E-μ model

1 Introduction

The deformation parameters of the D-C *E-μ M* are basic parameters in the engineering design of earth-rock dams. The values of these parameters need to be determined by conventional triaxial tests. Because of its simplicity and clarity, it is often used to calculate the stress-strain relationship of coarse-grained soils, thus providing guidance for the construction design of earth-rock dams. Coarse-grained soils are widely used as a dam construction material in earth-rock dam projects, and the rockfill materials used for dam construction can be more than 1 m in diameter. Due to the size limitations of the indoor instruments, the parameters of the prototype rockfill dam model cannot be measured directly. Therefore, it is necessary to downsize the original gradation to obtain scaled specimens for testing. However, due to the change in pore size and connectivity between soil particles after scaling, the test results of scaled specimens often fail to accurately reflect the true condition of the original gradation, a phenomenon known as the scale effect (Wang, 1994; Li et al., 2022). Previous studies showed that coarse-grained soil has an obvious scale effect (Wang, 1994; Yuan et al., 2024a; Liang et al., 2022; Yuan et al., 2024b; Pan and Sun, 2023; Li et al., 2022; Liu et al., 2021; Zhu et al., 2022; Fu et al., 2015), and the deformation parameters of the D-C *E-μ* M obtained by scaling test could not accurately calculate the deformation of the dam, which in turn affected the safety of the earth-rock dam project. Therefore, it is necessary to study the influence of scale effect on the deformation parameters of the D-C *E-μ* M for coarse-grained soils.

Researchers have investigated the scale effect on deformation parameters of coarse-grained soils. Ramamurthy and Donaghe (Wang, 1994) suggested that the scale effect significantly influenced the axial strain, volumetric strain, and elastic compression modulus but hardly impacted the peak material strength. Zhu et al. (2012) performed triaxial consolidated-drained tests on three specimens with different diameters. The results showed that after unifying the relative densities, the cohesion and internal friction angle of the coarse granular material tended to decrease with increasing specimen diameter. The variation of deformation modulus with specimen diameter was not significant. By studying the scale effect of rockfill materials by super-large and large triaxial testing machines, Kong et al. (2019) concluded that the cut-line modulus of elasticity and cut-line bulk modulus at 50% of the stress level of the large triaxial test were larger than those of the super-large triaxial test. The parameters K and K_b of the D-C *E-B* model for the large triaxial test were 1.22 and 1.38 times higher than those for the super-large triaxial test, respectively. Based on a series of triaxial tests with different maximum particle diameters, Li et al. (2001) inferred the parameter values of the prototype model by analyzing the relationship between the variation of the D-C *E-μ* M parameters and the scaled particle diameter ratio. Meanwhile, other researchers have also analyzed the impact of scale effects from different perspectives (Zhu et al., 2011; Qing et al., 2023; Zhou et al., 2021). Shao and Chi (2020) derived the correlation between each parameter of the D-C *E-B* M and particle diameter based on the stress-strain relationship between specimens of different diameters. Wang et al. (2013) conducted several sets of numerical experiments on scaled gradation using the PFC2D software. The results showed that the scaling relationship was correlated with the compactness control criteria. At the same relative density, the internal friction angle, bulk elastic modulus, and initial elastic modulus of the specimen with the largest particle diameter tended to increase. At the same dry density, the internal friction angle, bulk elastic modulus, and initial elastic modulus decreased and then increased.

Based on the above analysis, existing studies were mainly regularity studies focusing on the influence of scale effect on deformation parameters. The influence of gradation changes before and after scaling was neglected in these studies, and the influence of scale effect on model deformation parameters was studied only by one or a few gradation parameters, making it difficult to reflect the complete gradation changes. As a result, quantifying the influence of the scale effect on the deformation parameters of the D-C *E-μ* M became challenging. Moreover, the variation patterns of the D-C *E-μ* M parameters with gradation for coarse-grained soils after scaling are still insufficiently explained, remaining in the exploratory stage of in-depth qualitative research. Considering the grade changes before and after scaling, it is necessary to perform a quantitative experimental study on the influence of scale effect on the parameters of the D-C *E-μ* M.

Based on the grading equation proposed by Zhu et al. (2018) for applicable coarse-grained soils, 21 sets of specimens with different gradations were prepared by varying the gradation area S and the maximum grain diameter d_{max}. Large-scale triaxial consolidated-trained tests were conducted on specimens under the same relative compactness to analyze the effects of maximum grain diameter and gradation structure (gradation area) of coarse-grained soils on the parameters of the D-C *E-μ* M. Furthermore, the empirical formula for predicting the deformation parameters of the D-C *E-μ* M under the

Abbreviations: D-C E-μ M, Duncan-Chang E-μ model; d_{max}, Maximum particle size; d_{max0}, Boundary particle size of sand and gravel; d, soil particle size; p, percentage of particles smaller than a certain diameter; k, Calculate the gradation area parameters; m, Determine the morphological parameters of gradation curve; b, Determine the morphological parameters of gradation curve; S, Grading area; $P_{<5}$, The percentage of particle size less than 5 mm; ρ_{min}, minimum dry density; ρ_{max}, maximum dry density; ρ_0, Dry density at relative density; Dr, relative density; lgK, D-C E-μ M parameter; R_f, D-C E-μ M parameter; G, D-C E-μ M parameter; n, D-C E-μ M parameter; D, D-C E-μ M parameter; -F, D-C E-μ M parameter.

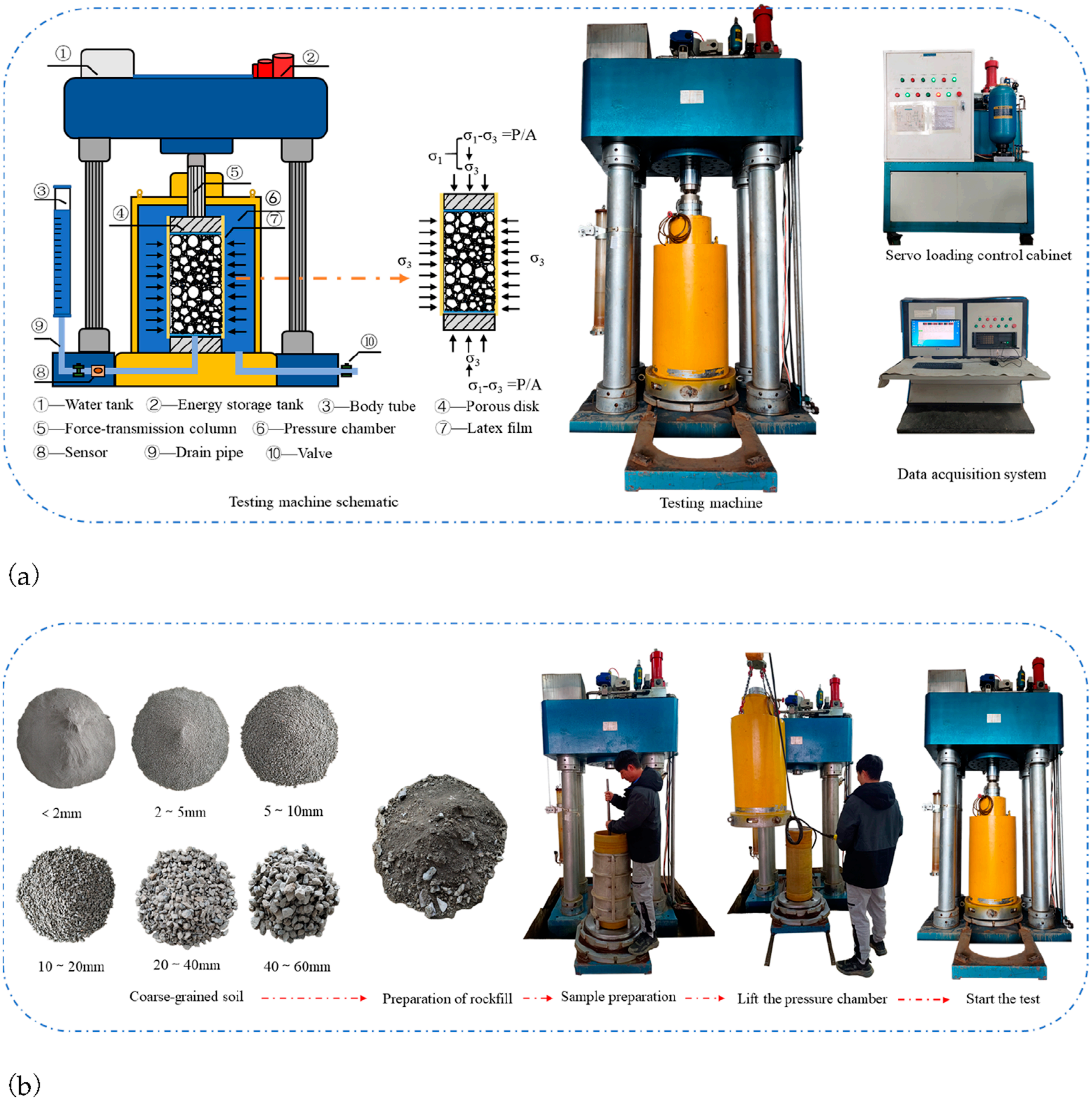

FIGURE 1
Large scale triaxial compression test of coarse-grained soil. **(a)** DJSZ-150 Three-axis compression test machine. **(b)** Triaxial compression test procedure of coarse-grained soil.

influence of the scaling effect was proposed based on the similar gradation method. In order to eliminate the influence of the scale effect, a method for predicting the deformation parameters of the D-C E-μ M for soils with original gradations is proposed based on indoor test results, and the reliability was verified with the data from previous studies. The findings of this study can provide a reference for organizing the parameters of the intrinsic model for high earth-rock dams and other similar projects.

2 Indoor triaxial test

2.1 Test equipment

DJSZ-150 large-scale dynamic and static triaxial testing machine for coarse-grained soil from the Geotechnical Engineering Experiment Center of Kunming College was used (Figure 1). The testing machine can accomplish dynamic and static loading for soil samples of Φ300 × 600 mm. It has an axial loading system,

FIGURE 2
Different particle group test soil material.

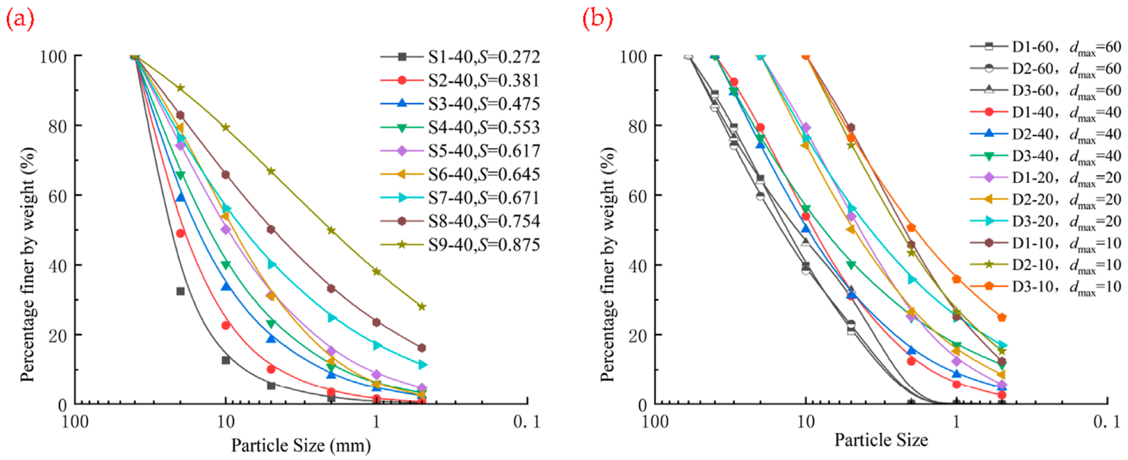

FIGURE 3
Particle gradation cumulative curve of rockfill material. **(a)** Grading curves for soil sample numbers 1 to 9. **(b)** Grading curves for soil sample numbers 10 to 21.

peripheral pressure loading system, and servo loading control system, enabling the realization of different stress path conditions. The maximum peripheral pressure of 3.0 MPa and static loading of 0–1,500 kN can be provided by the testing machine, and the test data can be automatically collected by computer. According to the Standard for Geotechnical Testing Method (GB/T50123-2019) (GT/T50123-2019, 2019), the particle diameter of the test soil should be less than 1/5 to 1/6 of the diameter of the instrument, i.e., this instrument can perform the triaxial consolidated-drained test on specimens with a maximum particle diameter of less than 60 mm. As determined by the pre-tests, the confining pressures for this test were 400, 800, and 1,200 kPa, and the loading rate was 1 mm/min.

2.2 Test material

The test material was selected from the rockfill materials for the dam construction of Lincangdaqiaopo Reservoir in Yunnan Province. This material is artificially blasted and crushed rock with weakly weathered granite as its parent rock. The rock has an average saturated uniaxial compressive strength of 50 MPa, a softening coefficient of 0.79, and a specific gravity of 2.70. The specimens

TABLE 1 Summary of gradation parameters, P_5 content and aperture ratio.

Soil sample number	d_{max} /mm	m	b	S	$P_{<5}$/%	ρ_{min} /(g·cm^{-3})	ρ_{max} /(g·cm^{-3})	ρ_0 /(g·cm^{-3})	Dr
S1-40	40	1.2	−0.6	0.272	5.30	1.56	1.78	1.74	0.85
S2-40	40	1.2	0.2	0.381	10.10	1.67	1.85	1.82	0.85
S3-40	40	0.9	0.2	0.475	18.50	1.75	1.90	1.88	0.85
S4-40	40	0.9	0.4	0.533	23.30	1.78	1.95	1.92	0.85
S5-40	40	0.9	0.6	0.617	31.20	1.69	1.97	1.92	0.85
S6-40	40	1.2	0.8	0.645	31.10	1.66	1.95	1.90	0.85
S7-40	40	0.6	0.4	0.671	40.17	1.72	2.00	1.95	0.85
S8-40	40	0.6	0.6	0.754	50.20	1.80	2.00	1.97	0.85
S9-40	40	0.6	0.8	0.875	66.80	1.84	2.17	2.11	0.85
D1-60	60	0.9	0.6	0.617	23.02	1.70	2.00	1.95	0.85
D1-40	40	0.9	0.6	0.617	31.20	1.69	1.97	1.92	0.85
D1-20	20	0.9	0.6	0.617	50.18	1.73	2.05	2.00	0.85
D1-10	10	0.9	0.6	0.617	74.27	1.52	1.72	1.69	0.85
D2-60	60	1.2	0.8	0.645	21.07	1.68	1.94	1.90	0.85
D2-40	40	1.2	0.8	0.645	31.10	1.66	1.95	1.90	0.85
D2-20	20	1.2	0.8	0.645	53.89	1.68	2.00	1.94	0.85
D2-10	10	1.2	0.8	0.645	79.40	1.51	1.77	1.72	0.85
D3-60	60	0.6	0.4	0.671	32.63	1.72	2.02	1.97	0.85
D3-40	40	0.6	0.4	0.671	40.17	1.72	2.00	1.95	0.85
D3-20	20	0.6	0.4	0.671	56.23	1.73	2.04	1.99	0.85
D3-10	10	0.6	0.4	0.671	76.37	1.76	2.03	1.98	0.85

with original gradations were subjected to particle sieving using a vibratory sieving machine, and groups of specimens with diameters of 60 to 40 mm, 40 to 20 mm, 20 to 10 mm, 10 to 5 mm, 5 to 2 mm, and less than 2 mm were retained as specimen soils, as shown in Figure 2. According to the grading curves in Figure 3, separate compaction tests were performed. After measuring the maximun and minimum of dried-densities, the specimens were prepared at a relative compactness of 0.85. The specimens were filled in 5 layers to ensure uniformity and compactness of each layer and flatness of the top surface.

2.3 Test scheme

Zhu et al. (2018) proposed a gradation equation describing the continuous gradation of soils by studying the gradation of artificially blasted rockfill materials of typical earth-rock dams:

$$P = \frac{1}{(1-b)\left(d_{max}/d\right)^m + b} \times 100\% \tag{1}$$

where P is the percentage of particles smaller than a certain diameter (%); d_{max} is the maximum particle diameter (mm); d is the particle diameter of the soil (mm); m and b are the parameters determining the morphology of the gradation curve, which determines the shape of the curve and the degree of inclination, respectively.

In this paper, 21 groups of graded specimens were designed by setting the values of parameters m and b. The corresponding gradation parameters are listed in Table 1. Among them, 12 groups are specimens with the same maximum particle diameter (d_{max} = 60 mm, 40 mm, 20 mm, and 10 mm) but with different gradations structures, and 9 groups are specimens with the same gradation area (S = 0.272, 0.381, 0.475, 0.533, 0.617, 0.645, 0.671, and 0.875) but with different maximum particle diameters. The corresponding

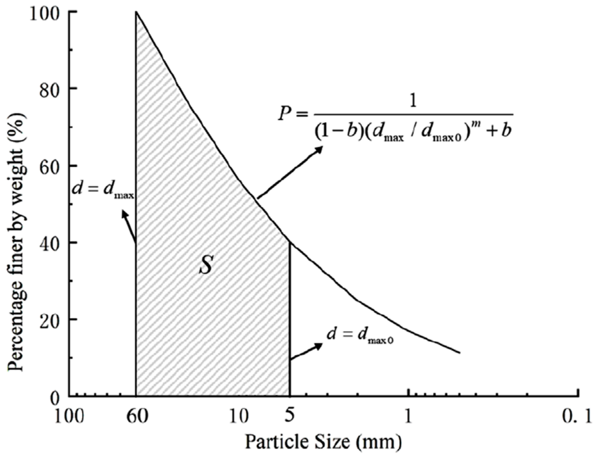

FIGURE 4
Area of gradation curve S.

gradation curves are shown in Figures 3a,b. $P_{<5}$ refers to the percentage of particles with a diameter of less than 5 mm, i.e., the content of fine particles.

By analyzing and summarizing the test data, no correlation was found between the gradation parameters m and b and the parameters of the D-C E-μ M. In contrast, these parameters were correlated with the gradation area S. As shown in Figure 4, the gradation area S is enclosed by the gradation equation curve, the coordinate horizontal axis, the line of $d = d_{max}$, and the line of $d = d_{max0}$ (Wu et al., 2019), which can be expressed as follows:

$$S = \frac{\ln(1-kb) - \ln(1-b)}{mb \ln 10} \tag{2}$$

where

$$k = \frac{1}{(1-b)\left(d_{max}/d_{max\,0}\right)^m + b} \tag{3}$$

Based on the above equation, the gradation curve area can be calculated by substituting the parameters m, b, and d_{max}/d_{max0} into the equation. If d_{max}/d_{max0} can be determined as a constant, the gradation curve area S can be determined by m and b, and the gradation structure is only related to m and b. Therefore, the gradation area can be used as a characteristic parameter to represent the change in gradation structure. Guo (2003) suggested using 5 mm as the criterion to distinguish sand and gravel. On this basis, this study sets d_{max} to 40 mm and d_{max0} to 5 mm when calculating the gradation area S. Therefore, only the same area S of the gradation curve before and after scaling is required. On this basis, the univariate method can be used to study the effect of the maximum particle diameter on the parameters of the D-C E-μ M for coarse-grained soils. In summary, using a single-variable method to set a series of initial conditions with the same maximum particle diameter and different gradation areas (as well as the same gradation areas and different particle maximum sizes) to study the relationship between the maximum particle diameter, gradation structure, and the deformation parameters of the D-C E-μ M is feasible.

In each consolidated-drained triaxial test, the main outputs included the axial stress–strain relationship, volumetric strain, and pore pressure response under different confining pressures. These measured curves were used to determine the Duncan–Chang E-μ model parameters (K, G, n, D, Rf, F) following the standard fitting procedure: (i) the initial tangent and secant moduli were obtained from the stress–strain curves to derive K and G; (ii) the ultimate stress and corresponding strain were used to calibrate Rf; (iii) parameters n and D were back-calculated by nonlinear fitting of the stress–strain curve; and (iv) parameter F was derived from the bulk modulus–strain relationship. Once the parameter set was determined for each specimen, the values were correlated with the gradation descriptors (d_{max} and S).

3 Analysis of test results

3.1 Relationship between D-C E-μ M parameters and gradation area

Based on the results of the large-scale triaxial consolidated-drained tests of soil materials S1-40 to S9-40, the relationship between the gradation area S and the variation of the D-C E-μ M parameters was compiled, which is shown in Figure 5. The results indicate the gradation area S has a large influence on the parameter properties of the D-C E-μ M when the maximum particle diameter is the same. The parameters G, K, F, and R_f in the model all decrease with increasing area S. Among them, F decreases faster before the gradation area reaches 0.4 and tends to flatten after that. The parameter K has a more significant curve pattern in logarithmic coordinates. Parameters n and D first increase and then decrease with increasing gradation area S. Parameter n reaches its peak value when the gradation area S is 0.5, and parameter D reaches its peak value when the gradation area S is 0.6. The parameter n shows a slightly decreasing tendency after reaching the peak value.

Equations 4–9 were obtained by performing nonlinear regression analyses on the experimental data of specimens S1-40 to S9-40. Based on the observed variation trends in Figure 5, logarithmic and exponential functions were selected as the most suitable forms to capture the relationships between the gradation curve area (S) and the Duncan–Chang E-μ model parameters (K, n, R_f, G, F, D). In these equations, the coefficients (a_i, b_i, c_i, d_i, e_i) are purely empirical fitting parameters without direct physical meaning, but they quantitatively describe the sensitivity of each model parameter to changes in S. The derivation assumes that the influence of gradation structure can be represented by a single descriptor (S), while the effect of maximum particle size (d_{max}) is considered separately in Section 3.2. This simplification ensures that the correlations reflect the isolated effect of S on model parameters. With the help of computer software to fit the experimental data, the equation for K, n, R_f, G, F, D, and S can be expressed as:

$$\lg K = \frac{a_1 + b_1 s}{c_1 + d_1 s^{e_1}} \tag{4}$$

$$n = \frac{a_2 + b_2 s}{c_2 + d_2 s^{e_2}} \tag{5}$$

$$R_f = \frac{a_3 + b_3 s}{c_3 + d_3 s^{e_3}} \tag{6}$$

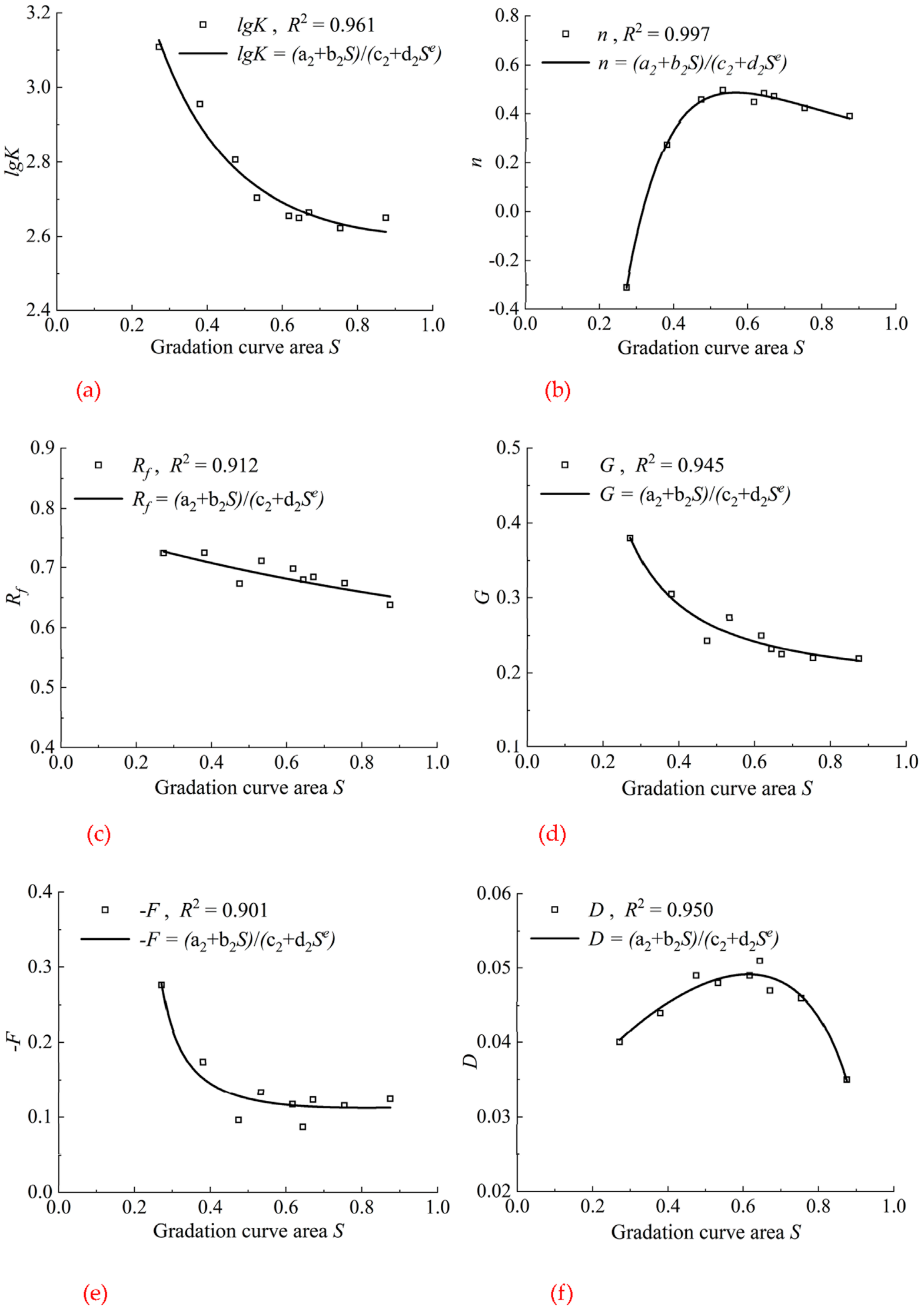

FIGURE 5
The relationship between gradation area S and Duncan-Chang *E*-μ model parameters. **(a)** The relationship between *lgK* and *S*. **(b)** The relationship between *n* and *S*. **(c)** The relationship between R_f and *S*. **(d)** The relationship between *G* and *S*. **(e)** The relationship between *-F* and *S*. **(f)** The relationship between *D* and *S*.

$$G = \frac{a_4 + b_4 s}{c_4 + d_4 s^{e_4}} \quad (7)$$

$$-F = \frac{a_5 + b_5 s}{c_5 + d_5 s^{e_5}} \quad (8)$$

TABLE 2 Fitting results of Equations 4–9 for sand and gravel materials.

lgK		N		R_f		G		-F		D	
a_1	1.520	a_2	−1.016	a_3	0.972	a_4	0.1841	a_5	0.081	a_6	0.007
b_1	1.125	b_2	3.214	b_3	−0.125	b_4	0.8077	b_5	0.321	b_6	−0.007
c_1	0.046	c_2	0.262	c_3	1.280	c_4	−0.566	c_5	−26.91	c_6	11.407
d_1	0.969	d_2	6.378	d_3	0.361	d_4	5.291	d_5	30.457	d_6	−11.397
e_1	0.452	e_2	2.676	e_3	14.274	e_4	0.904	e_5	0.077	e_6	−0.0081
R^2	0.961	R^2	0.997	R^2	0.921	R^2	0.945	R^2	0.901	R^2	0.950

$$D = \frac{a_6 + b_6 s}{c_6 + d_6 s^{e_6}} \tag{9}$$

where a_i, b_i, c_i, d_i, and e_i (values of i are from 1 to 6) are the equation parameters.

The test data for soil specimens S1-40 to S9-40 were fitted based on the above equation. The fitting results are shown in Table 2, and the fitting curve is plotted in Figure 5. It can be seen that the fitting curve for the deformation parameters of the D-C E-μ M agrees well with the test point, with an error basically less than 7.6% between them, a maximum error less than 9.12%, and the coefficient of determination above 0.901. Therefore, it can be assumed that the influence of the model parameter on the gradation area S in the scale effect can be quantitatively described by Equations 4–9.

3.2 Relationship between the D-C E-μ M parameters and maximum particle diameters

The result of the large-scale triaxial consolidated-drained test for soil materials D1-40 to D3-10 is summarized, and the change pattern of the D-C E-μ M parameters for coarse-grained soils with the same gradation area S and different maximum particle diameters $d_{\max}$ is shown in Figure 6. It can be seen that the D-C E-μ M parameters R_f, k, n, F, and D increase with the maximum particle diameter, and the parameter G decreases with the maximum particle diameter. Further analysis reveals that the maximum particle diameter $d_{\max}$ and the D-C E-μ M deformation parameters can be expressed as logarithmic functions. Therefore, the equation for the relationship between $d_{\max}$ of coarse-grained soils and model parameters can be expressed as:

$$\lg K = Z_1 \ln\left(d_{\max}/d_{\max 0}\right) + \lg K_0 \tag{10}$$

$$n = Z_2 \ln\left(d_{\max}/d_{\max 0}\right) + n_0 \tag{11}$$

$$R_f = Z_3 \ln\left(d_{\max}/d_{\max 0}\right) + R_{f0} \tag{12}$$

$$G = Z_4 \ln\left(d_{\max}/d_{\max 0}\right) + G_0 \tag{13}$$

$$-F = Z_5 \ln\left(d_{\max}/d_{\max 0}\right) + F_0 \tag{14}$$

$$D = Z_6 \ln\left(d_{\max}/d_{\max 0}\right) + D_0 \tag{15}$$

where $d_{\max 0}$ is the value distinguishing sand and gravel, which is taken as 5 mm; Z_1, Z_2, Z_3, Z_4, Z_5, Z_6, lgK_0, n_0, R_{f0}, G_0, F_0, and D_0 represent the parameters; lgK_0, n_0, R_{f0}, G_0, F_0, and D_0 represent the D-C E-μ M deformation parameters of the specimen when the maximum particle diameter $d_{\max}$ is 5 mm; Z_1, Z_2, Z_3, Z_4, Z_5, and Z_6 represent the change rate of the D-C E-μ M parameters when the maximum particle diameter is 5 mm.

The D1-40 to D3-10 test data were fitted according to Equations 10–15. The fitting results are shown in Tables 3–5, and the fitted curves are shown in Figure 6. It can be seen that the fitted curves are in good agreement with the test points, and the fitted curves of the deformation parameters of the D-C E-μ M are significantly consistent with the test points. The error between the fitted values of the parameters and the corresponding test points is basically less than 9.4%, the maximum error is less than 10.08%, and the coefficient of determination is basically greater than 0.913. According to mathematical statistics, all fitting data have strong correlations.

3.3 Establishment of the empirical formula of the D-C E-μ M based on the similar gradation method

The effects of maximum particle diameter and gradation area on the parameters of the D-C E-μ M can be quantitatively described by the above equations. However, the model parameters are meaningless quantities, and the above two parameters cannot be coupled to deduce the influence of the scale effect on the model parameters. Scaling by the similar gradation method provides a scaled specimen that has the same gradation area S as the soil material with the original gradation. Therefore, this study established an empirical formula for the influence of scale effect on model parameters based on the similar gradation method. The formula was fitted with the test results obtained from specimens with particle diameters of 10–40 mm, and the experimental data obtained from specimens with a maximum particle diameter of 60 mm were

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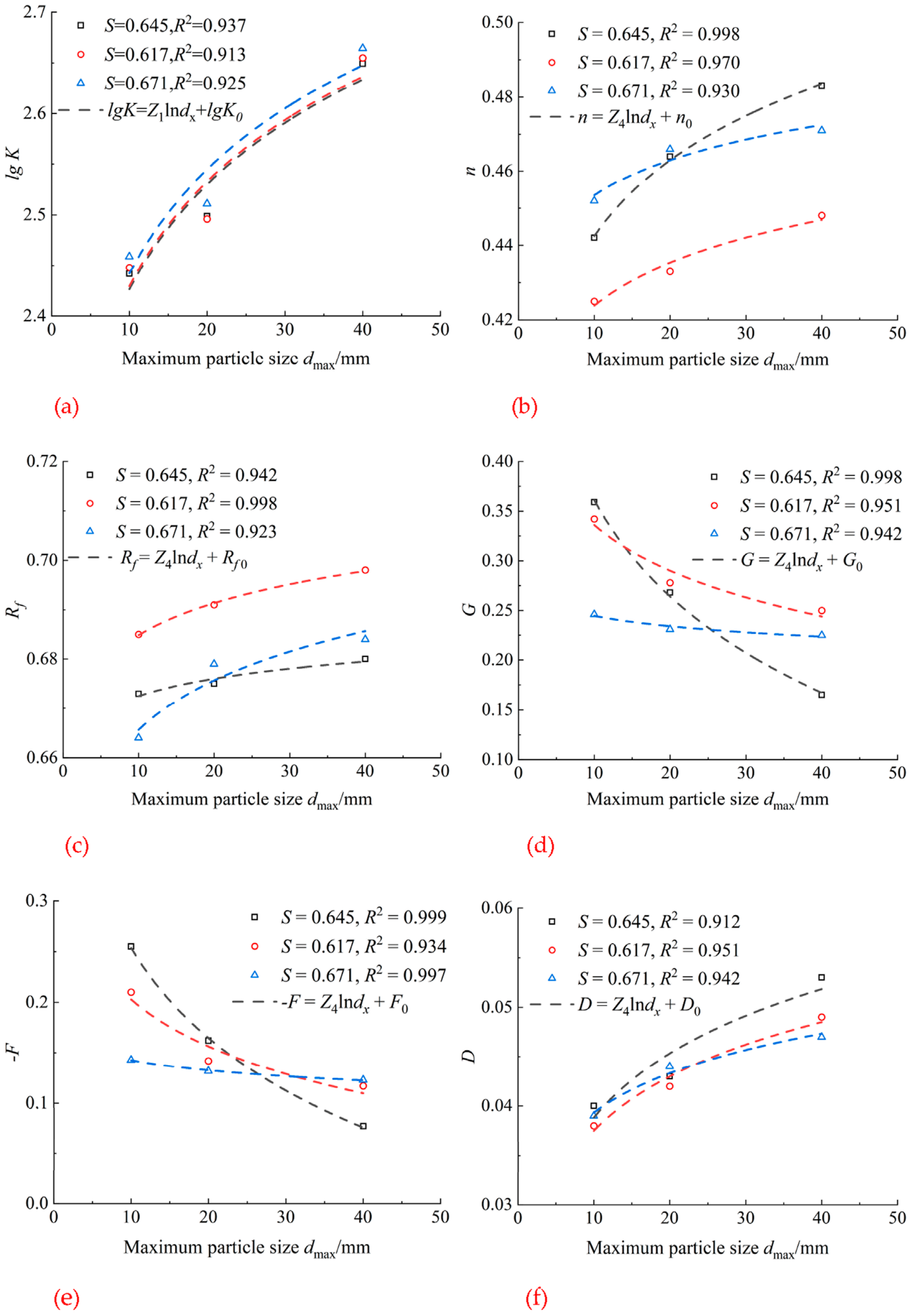

FIGURE 6
The relationship between maximum particle size d_{max} and Duncan-Chang E-μ model parameters. **(a)** The relationship between lgK and d_{max}. **(b)** The relationship between n and d_{max}. **(c)** The relationship between R_f and d_{max}. **(d)** The relationship between G and d_{max}. **(e)** The relationship between $-F$ and d_{max}. **(f)** The relationship between D and d_{max}.

TABLE 3 Fitting results of Equations 10–15 for sand and gravel materials (S = 0.617).

lgK		n		R_f		G		$-F$		D	
z_1	0.196	z_2	0.016	z_3	0.005	z_4	−0.066	z_5	−0.059	z_6	0.008
lgK_0	2.326	n_0	0.385	R_{f0}	0.676	G_0	0.512	$-F_0$	0.355	D_0	0.019
R^2	0.913	R^2	0.998	R^2	0.998	R^2	0.951	R^2	0.934	R^2	0.976

TABLE 4 Fitting results of Equations 10–15 for sand and gravel materials (S = 0.645).

lgK		n		R_f		G		$-F$		D	
z_1	0.184	z_2	0.029	z_3	0.009	z_4	−0.135	z_5	−0.120	z_6	0.009
lgK_0	2.083	n_0	0.374	R_{f0}	0.661	G_0	0.699	$-F_0$	0.565	D_0	0.017
R^2	0.937	R^2	0.970	R^2	0.942	R^2	0.998	R^2	0.999	R^2	0.912

TABLE 5 Fitting results of Equations 10–15 for sand and gravel materials (S = 0.671).

lgK		n		R_f		G		$-F$		D	
z_1	0.172	z_2	0.015	z_3	0.014	z_4	−0.013	z_5	−0.014	z_6	0.006
$\mathrm{lgK_0}$	2.220	n_0	0.425	R_{f0}	0.633	G_0	0.283	$-F_0$	0.178	D_0	0.027
R^2	0.925	R^2	0.930	R^2	0.923	R^2	0.942	R^2	0.997	R^2	0.980

TABLE 6 Summary table of comparison between experimental and calculated values of deformation parameters in Duncan Zhang E-μ model.

Test number	Type of numeric value	lgK	n	R_f	G	$-F$	D
D1-60	Measurement values	3.139	0.450	0.704	0.243	0.113	0.050
	Calculation values	3.130	0.450	0.699	0.241	0.113	0.052
D2-60	Measurement values	2.887	0.494	0.697	0.152	0.076	0.053
	Calculation values	2.836	0.492	0.698	0.146	0.074	0.054
D3-60	Measurement values	2.982	0.488	0.692	0.232	0.122	0.052
	Calculation values	2.924	0.486	0.690	0.230	0.120	0.051

verified. The validation results demonstrated the correctness of the empirical formula when using the similar gradation method for scaling.

Since the empirical formula of the D-C E-μ M based on the similar gradation method eliminates the effect of the gradation area, it can be expressed by Equations 10–15. Using the fitting results of Equations 10–15 and Tables 3–5, the deformation parameters of the D-C E-μ M for a specimen with a maximum particle diameter $d_{\max}$ of 60 mm were calculated, and the calculation values were compared with the measurement values. The calculation results are shown in Table 6, and the comparison is illustrated in Figure 7.

As can be seen from Figure 7, the error between the predicted and experimental values of gravel is not large. The error between the fitted values of the model parameters and the corresponding test points is basically less than 2.8%, the maximum error is less than 7.68%, and the coefficients of determination are all above 0.956, which is in an acceptable range. Therefore, the influence of scale effect on the deformation parameters of coarse-grained soils can be quantitatively described by Equations 10–15.

The use of the empirical formula to reflect the effect of the maximum particle diameter on the deformation parameters of the D-C E-μ M is of great application value. By determining the

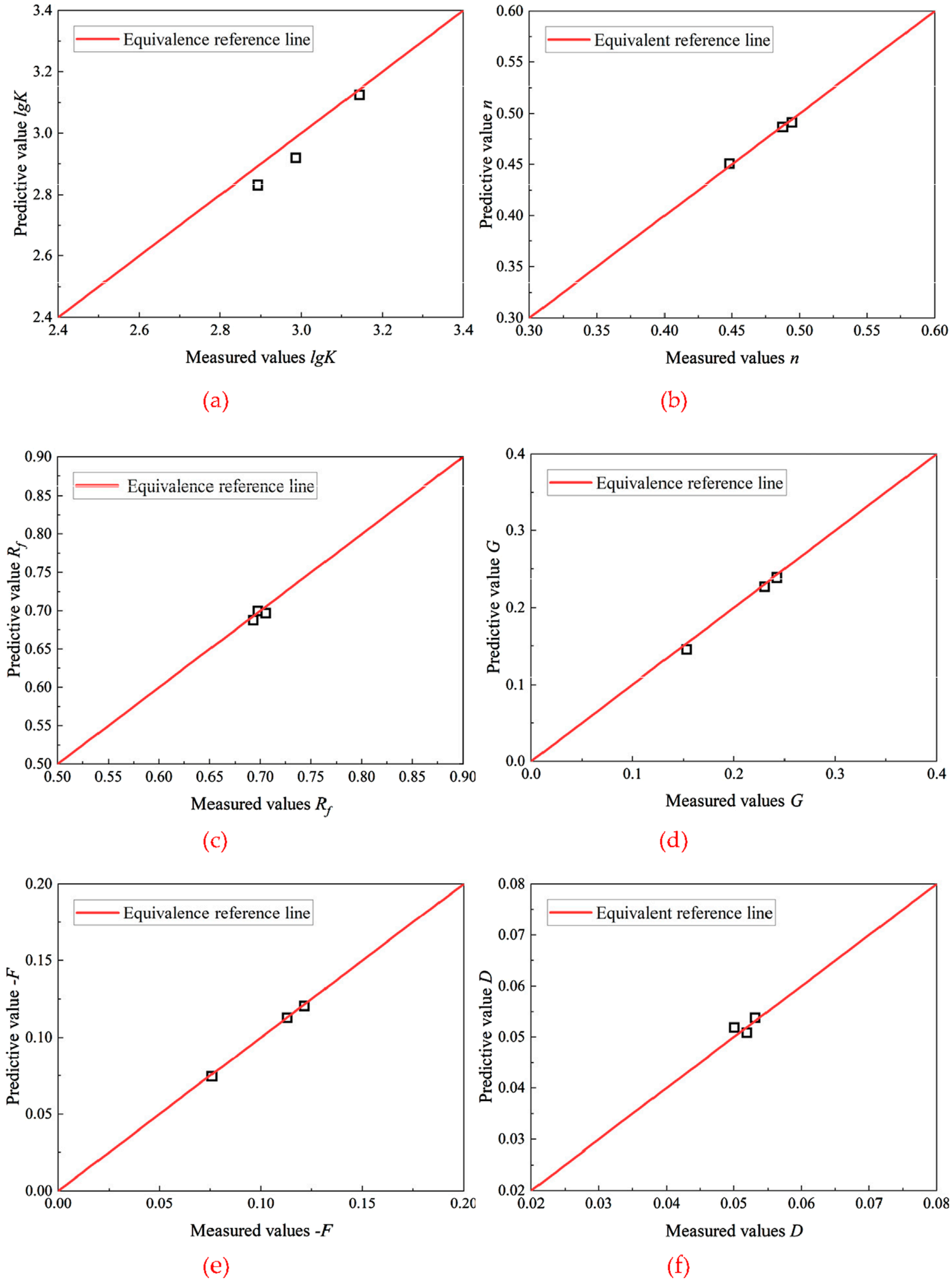

FIGURE 7
The measured and calculated values of deformation parameters for Duncan Zhang E-μ model for rockfill materials. **(a)** Parameter *lgK*. **(b)** Parameter *n*. **(c)** Parameter R_f . **(d)** Parameter. *G*. **(e)** Parameter *-F*. **(f)** Parameter *D*.

material parameters in Equations 10–15 from a series of large-scale triaxial consolidated-drained tests on scaled soils using the similar gradation method, the deformation parameters of the D-C E-μ M for the coarse-grained soils with the original gradation can be derived. In this way, the influence of scale effect on the deformation characteristics of coarse-grained soils can be eliminated, and

TABLE 7 Summary of Duncan Zhang E-μ model parameters for different materials.

Source		d_{max}/mm	lgK	n	R_f	G	-F	D
Purulia dam	Measurement values	25	2.787	0.452	0.642	0.569	0.291	−0.075
	Measurement values	50	2.687	0.513	0.663	0.580	0.249	−0.057
	Measurement values	80	2.642	0.538	0.690	0.586	0.235	−0.040
	Calculation values	80	2.635	0.550	0.678	0.586	0.221	−0.045
Parbati dam	Measurement values	20	2.707	0.778	0.643	0.615	0.115	−0.053
	Measurement values	40	2.603	0.823	0.658	0.622	0.110	−0.046
	Measurement values	80	2.518	0.856	0.662	0.642	0.081	−0.044
	Calculation values	80	2.504	0.866	0.672	0.630	0.102	−0.041
Kol dam	Measurement values	25	2.837	0.509	0.774	0.593	0.129	−0.026
	Measurement values	50	2.780	0.538	0.748	0.568	0.186	−0.028
	Measurement values	80	2.737	0.554	0.714	0.545	0.235	−0.028
	Calculation values	80	2.740	0.557	0.728	0.551	0.224	−0.028

the safety and reliability of geotechnical engineering design can be improved.

4 Validation of the empirical formula for the parameters of the D-C E-μ M based on the similar gradation methods

In this study, only large-scale triaxial consolidated-drained tests on sand and gravel were conducted. In order to verify the applicability of the proposed prediction formula to different types of coarse-grained soils, the above conclusions need to be supported with more experimental data. Varadarajan et al. (2006) conducted triaxial drainage tests using rockfill materials from the Purulia and Parbati dams and alluvial rockfill materials from the Kol dam, which had similar gradations. Table 7 summarizes the values of the E-μ model parameters for the three different materials, and the equation parameters are derived by fitting some of these experimental data using Equations 10–15 to derive the model parameters with original gradations. The comparison of the predicted values and measured values is shown in Figure 8.

It can be seen from Figure 8 that the data of predicted and measured values are in good agreement. The maximum error of 4.18% for the deformation parameters of the D-C E-μ M illustrates the reliability of the empirical formula for predicting model parameters and its applicability to different types of coarse-grained soils. Therefore, by performing triaxial tests on soils of similar gradation to obtain each parameter in Equations 10–15, the deformation parameters of the D-C E-μ M for coarse-grained soils of any gradation can be better predicted.

5 Conclusion

Based on the continuous gradation equation of soil, 21 groups of graded specimens with maximum particle diameters d_{max} of 60, 40, 20, and 10 mm were designed, and a series of consolidated-drained tests were performed on the coarse-grained soil using DJSZ-150 large-scale dynamic and static tri-axial testing machine. Furthermore, a quantitative study of the D-C E-μ M parameters for coarse-grained soils with different maximum particle diameters and gradation structures under the same relative compactness was performed. The conclusions of this study are as follows:

1. When the maximum grain size d_{max} is constant, the parameters G, lgK, F, and R_f in the D-C E-μ M decrease with increasing gradation area S. Parameters n and D increase and then decrease with the increasing gradation area S, followed by a decreasing trend after reaching a specific value. The equations between each model parameter and gradation area S are established.
2. When the gradation area S remains constant (i.e., the gradation structure is constant), the D-C E-μ M parameters R_f, lgK, n, F, and D increase with the maximum parameter diameter d_{max}. In contrast, the parameter G decreases as the maximum particle diameter d_{max} increases. All model parameters follow a logarithmic function with d_{max}.
3. An empirical prediction model of D-C E-μ parameters for coarse-grained soils was developed using the similar gradation method, explicitly incorporating the influence of maximum particle size.
4. The proposed empirical formula was validated against indoor test data and previous studies, confirming its capability to eliminate scale effects and its applicability to different types of coarse-grained soils.

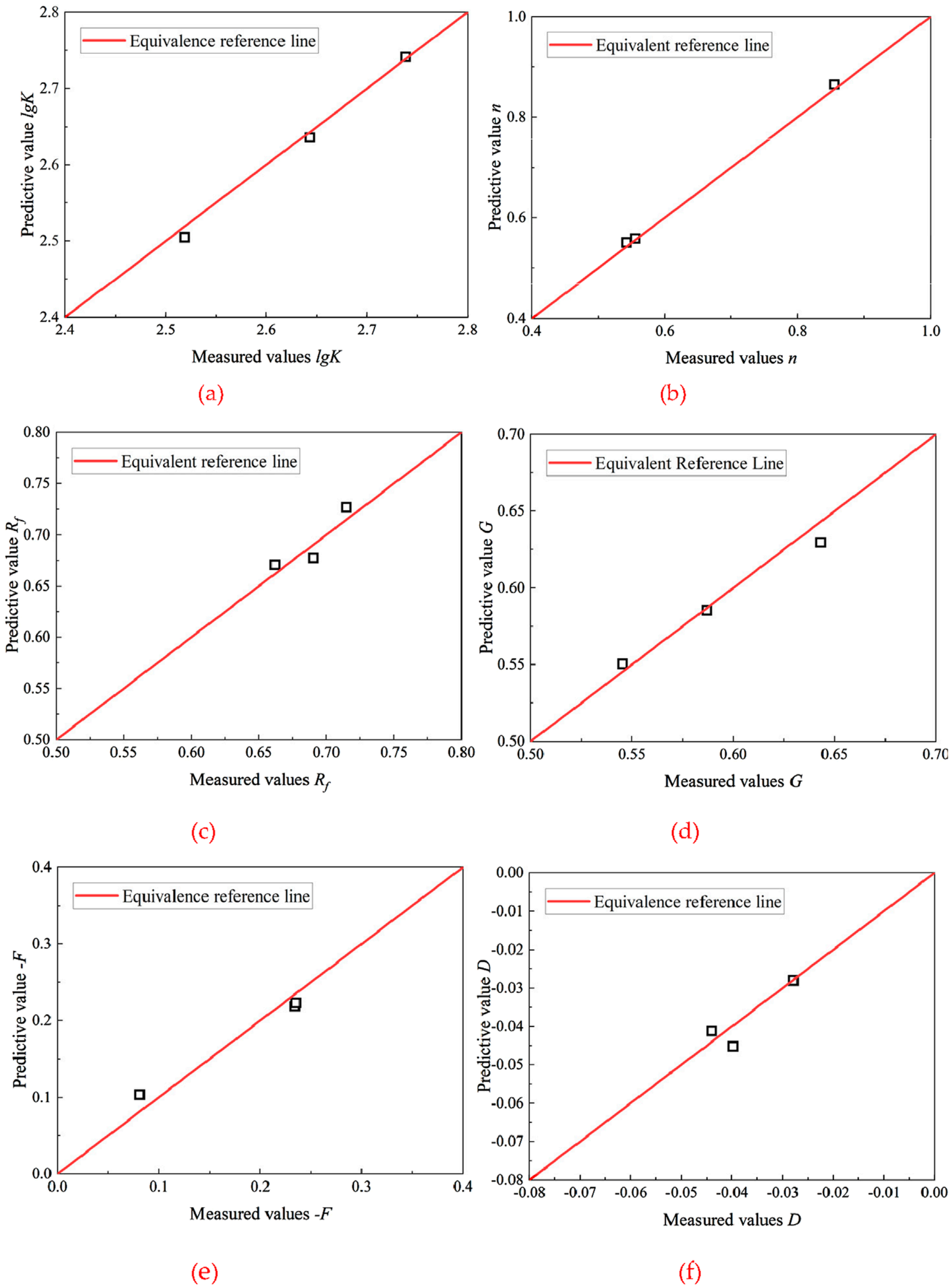

FIGURE 8
Measurement and calculation of deformation parameters of Duncan Zhang E-μ model for rockfill materials. **(a)** Parameter *lgK*. **(b)** Parameter *n*. **(c)** Parameter R_f. **(d)** Parameter *G*. **(e)** Parameter *-F*. **(f)** Parameter *D*.

Future research should extend the present work in several directions. First, the proposed empirical formula has only been validated for maximum particle sizes up to 60 mm and for weakly weathered granite; therefore, larger particle sizes and other lithologies, particularly soft or easily breakable rockfill, should be investigated to improve generality. Moreover, the current study mainly addresses geometric scale effects, while the role of particle breakage and its interaction with scale effects under high confining stresses remains to be clarified. In addition, although the gradation curve area (*S*) provides a useful single descriptor, future work should consider multiple indices such as uniformity coefficients or fractal dimensions to better capture complex gradation features. Finally, validation through DEM-based numerical simulations and field monitoring data is recommended to strengthen engineering applicability and to quantify improvements in deformation and stability predictions for earth–rock dams.

Data availability statement

The raw data supporting the conclusions of this article will be made available by the authors, without undue reservation.

Author contributions

XS: Funding acquisition, Project administration, Writing – review and editing, Methodology. XL: Funding acquisition, Resources, Writing – review and editing. ZX: Writing – review and editing, Supervision, Resources, Funding acquisition. JD: Investigation, Writing – review and editing, Resources, Funding acquisition. RS: Conceptualization, Methodology, Writing – original draft, Formal Analysis, Software. HJ: Validation, Supervision, Writing – review and editing. MJ: Visualization, Writing – review and editing, Validation, Methodology, Supervision. QL: Writing – review and editing. CW: Writing – review and editing.

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Conflict of interest

Author ZX was employed by Yunnan Communications Investment & Construction Group Co., LTD. Dali Management Office. Author HJ was employed by Inner Mongolia Traffic Design and Research Institute Limited Liability Company.

The remaining authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

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