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Structural Analysis of Panel Rockfill Dam Based on Finite Element Secondary Development

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ABSTRACT: To enhance the engineering applicability of deformation analysis for concrete-filled rockfill dams (CFRDs), this study conducts secondary development of MSC Marc through finite element modeling. The HYPELA2 is employed to implement the Duncan–Chang E-B constitutive model, while CRPLAW is utilized for the Burgers rheological model, establishing a three-dimensional partitioned finite element framework capable of simultaneously describing nonlinear and time-varying effects. Taking a specific CFRD as an example, a 3D model was constructed with stratified filling conditions, aligning grid nodes with monitoring points for result comparison. Results indicate that the maximum settlement occurs in the middle-lower dam body, concentrated in the downstream sub-rockfill zone, reaching approximately 36.41 cm upon completion. Horizontal displacement exhibits differential distribution between upstream and downstream sections. The study validates the feasibility of secondary development in MSC Marc for implementing E-B and Burgers models, providing a computational foundation for deformation analysis and monitoring interpretation during CFRD construction phases.

KEYWORDS: CFRD, MSC Marc, Duncan–Chang E-B,Burgers Rrheological Model..

I. INTRODUCTION

Concrete-faced rockfill dams (CFRDs), as one of the most widely used dam types in high dam engineering, exhibit significant nonlinearity, regional variability, and time-dependent deformation responses during their service life. Throughout the entire process of dam construction, initial impoundment, and long-term operation, dam displacement is often controlled by two mechanisms: one is the instantaneous deformation caused by self-weight and water pressure (dominated by nonlinear stress-strain relationships), and the other is the rheological deformation of the rockfill material under long-term loading (dominated by creep, delayed elasticity, and viscous flow). Especially for high dams, the proportion of rheological displacement in the total displacement during operation is not negligible. Relying solely on static constitutive models is insufficient to reproduce the true law of settlement development over time, thus affecting the reliability of panel stress assessment, regional deformation interpretation, and safety evaluation conclusions.

From a numerical simulation perspective, the three-dimensional finite element method can provide high engineering fidelity at the geometric, zoning, and load path levels. However, the key lies in whether the constitutive model can simultaneously account for both "instantaneous nonlinearity" and "long-term rheological behavior" and be stably implemented in a commercial finite element platform. Given the significant differences in applicability and generalization among different soil and rock material models, and the fact that many models, based on specific assumptions, only accurately characterize certain properties, engineering applications require model selection and implementation path design based on the stress characteristics of rockfill dams and available parameter conditions. Therefore, this paper adopts the Duncan-Chang EB model as the static constitutive model for rockfill materials to characterize their nonlinearity and compressive hardening characteristics, and selects the Burgers element rheological model to describe the deformation evolution of rockfill materials over time. Simultaneously, relying on the secondary development interfaces (HYPELA2 and CRPLAW) provided by MSC Marc, the above constitutive and rheological models are embedded

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in Fortran, thereby constructing a unified computational framework that can be used for layered filling.

At the engineering verification level, to improve the comparability and consistency of interpretation between "calculation results and monitoring response," this paper adopts a mesh strategy of nodalizing monitoring points during 3D finite element modeling. This ensures a one-to-one correspondence between the monitoring instrument locations and finite element nodes, facilitating the direct extraction of settlement and horizontal displacement for comparative analysis. Furthermore, calculations are conducted using typical engineering examples to systematically discuss the characteristics of dam bodies during layered filling. Based on this approach, the following sections will sequentially introduce the selection and theoretical basis of the dam body material constitutive model, the secondary development implementation method on the MSC Marc platform, and the process of engineering example modeling, working condition settings, and calculation result analysis.

II. Constitutive Model of Dam Material

In the finite element numerical simulation analysis of face rockfill dams, it is essential to simultaneously examine both transient displacement and rheological displacement of the dam body. Typically, static constitutive models are employed to calculate transient displacement, while rheological constitutive models are used for rheological displacement. Currently, mainstream soil constitutive models are primarily categorized into two types: static models and rheological models. Models proposed domestically and internationally each have their own emphases, but most still exhibit insufficient applicability and universality, with relatively limited models widely adopted in geotechnical engineering. Given the complexity of soil stress-strain relationships, researchers often introduce specific assumptions when establishing mathematical models, resulting in these models typically only accurately describing certain aspects of soil mechanical properties. Therefore, different material models must be selected based on specific engineering requirements. This study adopts the Duncan-Zhang E-B model as the static constitutive model and the Burgers model as the rheological constitutive model for the finite element numerical simulation of face rockfill dams.

2.1 A Static Constitutive Model for Rock Mass Materials

In static analysis, rockfill structures primarily bear self-weight and hydrostatic pressure loads. The two most widely used static constitutive models are the Duncan-Zhang E-B model and the E-v model, both of which are nonlinear. The E-v model was derived by Duncan and Zhang in 1970, later refined into the E-B model in 1980. The E-v hyperbolic model establishes a stress-strain hyperbolic relationship based on triaxial test results $(\sigma_1 - \sigma_3) \sim \epsilon_a$ and $\mathscr{E}_\Gamma \sim \mathscr{E}_a$, deriving the mathematical expressions for the tangential Young's modulus E and tangential Poisson's ratio v. The E-B model, building upon the E-v framework, replaces the tangential Poisson's ratio v with the tangential volume modulus Bt. Scholars like Zhu Junhao have systematically compared the two models from an engineering practice perspective, noting that while their structural elastic modulus calculations show minimal differences, both models have limitations in Poisson's ratio values. Compared to the E-v model, the E-B model more accurately characterizes the nonlinear and compressive-hardening properties of rockfill structures. With mature experimental methods, simple parameter acquisition, and years of engineering validation, this model has become widely adopted in hydraulic engineering for simulating rockfill deformation characteristics. Given these advantages, this study selects the Duncan-Zhang E-B model as the static constitutive model for rockfill structures.

The Duncan-Chang E-B model approximates the relationship between deviatoric stress and axial strain using a hyperbolic curve, from which the tangent elastic modulus E_t is derived as follows:

$$E_t = \frac{E_i}{1-s} \qquad (2\text{-}1)$$

where:

E_i is the initial tangent modulus,

R_f is the failure ratio,

S is the stress level.

Since the initial tangent modulus varies with different confining pressures, its value can be calculated as:

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$$E_i = KP_a\left(\frac{\sigma_3}{P_a}\right)^n \quad (2\text{-}2)$$

where:

K is the intercept of the fitted line,

n is the slope of the fitted line,

P_a is the atmospheric pressure,

σ_3 is the confining pressure.

The failure ratio R_f is given by:

$$R_f = \frac{\sigma_{1f}}{\sigma_1} \quad (2\text{-}3)$$

where σ_{1f} represents the asymptotic deviatoric stress, and σ_1 is the deviatoric stress at failure. Using the Mohr-Coulomb failure criterion, the deviatoric stress at failure can be expressed as:

$$\sigma_1 = 2c\cos\phi + 2\sigma_3\sin\phi \quad (2\text{-}4)$$

where:

c is the cohesion,

ϕ is the internal friction angle.

Given that the cohesion for rockfill materials is often assumed to be zero, the stress level S can be calculated as:

$$S = \frac{\sigma_1}{\sigma_{1f}} \quad (2\text{-}5)$$

By substituting these equations, the final tangent elastic modulus under loading conditions is given by:

$$E_t = \frac{KP_a\left(\frac{\sigma_3}{P_a}\right)^n}{1-\left(\frac{\sigma_1}{\sigma_{1f}}\right)} \quad (2\text{-}6)$$

During the construction and operation of rockfill dams, the loading and unloading process (such as during reservoir filling and drawdown) is cyclic. In these conditions, the initial tangent modulus cannot accurately describe the unloading and reloading process. Therefore, the rebound modulus E_r is used in place of the initial tangent modulus, and is calculated as:

$$E_r = K_rP_a\left(\frac{\sigma_3}{P_a}\right)^n \quad (2\text{-}7)$$

where K_r and n can be determined through experimental tests. The tangent bulk modulus B_t, related to the confining pressure, is expressed as: □

$$B_t = K_bP_a\left(\frac{\sigma_3}{P_a}\right)^m \quad (2\text{-}8)$$

where:

K_b is the bulk modulus coefficient,

m is the bulk modulus exponent.

In summary, the Duncan-Chang E-B model requires nine parameters for computation: $K, n, P_a, R_f, c, \phi, K_r, m$, and K_b. However, since the cohesion c is assumed to be zero for rockfill materials, only eight parameters are required for the static analysis of rockfill dams in this study.

2.2 Rheological Model of Rock Mass Material

Soil (including rockfill) is not a homogeneous material but a complex three-phase system comprising gas, liquid, and solid components. Rockfill deformation is influenced not only by static loads such as self-weight and hydraulic pressure, but also by time-dependent displacement effects known as rheological displacement. The rheological characteristics of rockfill typically manifest over extended periods, potentially persisting for decades in rockfill dams before gradually stabilizing. During operational phases, rheological displacement constitutes a significant portion of total dam displacement, rendering it non-negligible. Consequently, extensive research has been conducted on rheological displacement. Currently, the most widely used rheological calculation models fall into two categories: empirical models and element models. Empirical models are mathematical fitting equations established through extensive in-situ loading tests, using polynomial functions to approximate rockfill displacement variations. Since these models are derived from experimental data, scholars have developed various empirical models tailored to specific projects, such as Shen Zhujiang's three-parameter model, Li Guoying's seven-parameter model, and Zhu Sheng's seven-parameter model. This demonstrates that empirical models vary significantly across different engineering contexts and rockfill types, making them less applicable than element models. Element models integrate physical components—including Hookean elastic elements (springs),

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Newtonian viscous elements (viscous fluid), and Saint-Venant plastic elements (slips)—into series or parallel configurations to create diverse rheological models. Based on these considerations, this study adopts the Burgers model as the rheological calculation framework for rockfill dams.

In this study, the Burgers model is selected as the rheological model for rockfill materials. The schematic diagram of the Burgers model is shown in Figure1.

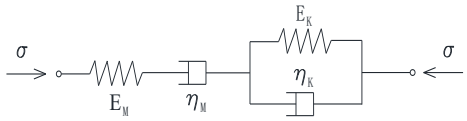

Figure 1 Burgers Model Schematic

The total strain in the Burgers model is the sum of the strains from the Maxwell model and the Kelvin-Voigt model. The rheological equation at time t is given by:

$$\varepsilon(t) = \frac{\sigma_0}{E_M} + \frac{\sigma_0}{\eta_M} t + \frac{\sigma_0}{E_K}\left(1 - e^{-\frac{t}{\tau}}\right) \quad (2\text{-}9)$$

where:

$\varepsilon(t)$ is the strain at any time t,

E_M and η_M are the elastic and viscous coefficients of the Maxwell model,

E_K and η_K are the elastic and viscous coefficients of the Kelvin-Voigt model,

$\tau = \frac{\eta_K}{E_K}$ is the relaxation time of the Kelvin-Voigt model.

The strain increment over a given time interval is calculated as:

$$\Delta\varepsilon = \frac{\Delta t}{\eta_M} + \frac{\sigma_0}{E_K}\left(1 - e^{-\frac{\Delta t}{\tau}}\right) \quad (2\text{-}10)$$

In summary, the Burgers rheological model has four parameters: E_M, η_M, E_K, and η_K. For the strain increment calculation, only three parameters are required: η_M, E_K, and η_K. These parameters are used in the rheological analysis of rockfill dams to account for time-dependent displacements.

III. FEA of Panel Rockfill Dam Based on MSC.Marc

3.1 Implementation of E-B Model in MSC.Marc Software

Among the multiple user subroutines in MSC.Marc software, HYPELA is specifically designed for defining material models through secondary development, particularly for customized development of the Duncan-Zhang E-B model. With continuous software version upgrades, HYPELA has been expanded into the more comprehensive HYPELA2 version. Compared to HYPELA, HYPELA2 provides more extensive functional support, which is why this study adopts this subroutine as its research foundation.。

The HYPELA2 system operates by having MSC.Marc provide initial stress, strain, strain increments, and other state variables for the subroutine at the start of each increment step. The user subroutine then defines the corresponding stress s, tangent stiffness matrix D, state variables, and other required parameters at the end of each increment step. This iterative process continues until all increment and analysis steps are completed.

The user subroutine HYPELA2 features a fixed structural framework. At the beginning of the subroutine file, the subroutine type must first be defined by declaring it as SUBROUTINE HYPELA2, followed by the declaration of primary variables. After completing the header file references, the user enters the custom module phase, where the variables declared in the header file must be explicitly defined.

3.2 The Implementation of the Rheological Model in MSC.Marc

Among the extensive user subroutines provided by MSC.Marc, CRPLAW is a specialized module for secondary development that defines non-viscous strain rates in viscoelastic models. As a key component for material constitutive modeling, it enables users to customize Burgers' rheological models. Through this interface, users can define material properties including stress-strain curves, plastic flow laws, and yield surfaces. During customization, users must write Fortran code to interact with the CRPLAW subroutine.

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IV. Engineering Example

4.1 Overview of Panel Rockfill Dam of a Hydropower Station Project

Located in the upper reaches of the Yellow River, this hydropower project features five 300-megawatt generating units with an annual output of 5.14 billion kilowatt-hours. The reservoir maintains a normal water level at 2,005.0 meters and a verified flood level at 2,008.0 meters, with a total storage capacity of 620 million cubic meters and an annual runoff of 22.6 billion cubic meters. The upstream reservoir controls a watershed area of 144,000 square kilometers. As a Class I (Type 1) major water conservancy project, it primarily serves power generation while also providing irrigation and water supply functions.

The hub project primarily comprises water-retaining structures, water diversion and power generation facilities, and drainage systems. The dam adopts a concrete-faced rockfill structure, with a crest elevation of 2,010.0 meters, a maximum height of 132 meters, a crest width of 10 meters, and a total length of 429 meters. A 5.8-meter-high concrete wave wall is installed on the upstream side of the dam crest. The upstream slope ratio of the dam body is 1:1.4, with an additional 10-meter-wide, 1:2 slope ratio earth-rock slope material added to the surface layer of the concrete face below the 1,940.0-meter elevation.

The concrete slab features a variable thickness design, with thickness calculated using the formula (0.3 + 0.003H) meters (where H denotes the slab height). The top layer measures 0.3 meters, while the bottom layer reaches a maximum thickness of 0.76 meters. Vertical joints are installed through stress control, dividing the slab into 38 sections (Figure 2). The slab is constructed with C25 concrete and reinforced with bidirectional steel mesh.

The downstream slope ratio of the dam ranges from 1:1.5 to 1:1.3, with a composite slope ratio of 1:1.8. The slope base is backfilled to an elevation of 1909.2 meters. A 10-meter-wide zigzag-shaped road is constructed upstream of the dam slope.

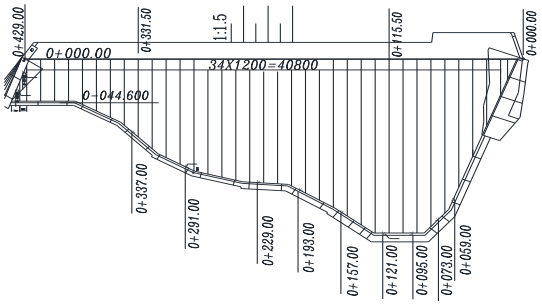

Figure 2 Concrete panel seam drawing (Unit: m)

The dam is divided into reinforced concrete slab, cushion layer (2A), transition layer (3A), main rockfill I (3BI), main rockfill II (3BII) and secondary rockfill (3C) from upstream to downstream.

4.2 Finite element meshing

Based on engineering data and topographical factors, a three-dimensional finite element model was established for a face rockfill dam, including concrete face panels, toe plates, transition zones, cushion zones, compression walls, and various dam body sections. Given the intact bedrock with good quality and minimal deformation, the modeling process excluded bedrock considerations and applied fixed boundary conditions. To facilitate data extraction, nodes were strategically placed at monitoring instrument locations during meshing, ensuring precise instrument-node correspondence. The model was divided into sections and positions with reasonable segmentation, prioritizing the dam body's characteristic lines as partition boundaries. The model employed 8-node isoparametric elements, totaling 10,378 elements and 10,966 nodes.

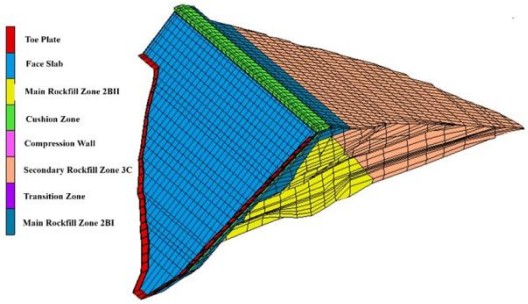

Figure 3 Finite element calculation model mesh

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4.3Analysis of calculation results

The finite element analysis of the three-dimensional panel rockfill dam is illustrated in Figure 4. The results are presented and analyzed using the maximum cross-sectional data at the 0+130 mark on the dam's left side. The dam's settlement value is defined as positive for upward movement and negative for downward movement, measured in meters (m). The horizontal displacement value is defined as positive for downstream (rightward) and negative for upstream (leftward), also measured in meters (m).

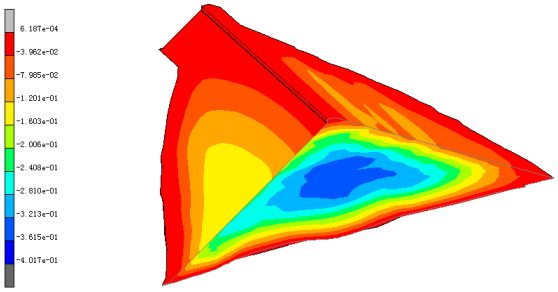

Figure 4 Three-dimensional display of the calculation results of the faceplate rockfill dam

The dam's layered filling process was analyzed in 25 stages. During calculations, only the gravitational force of each rockfill layer was considered. Upon completion, settlement values and horizontal displacements are shown in Figures 5 to 6. As the maximum settlement occurred at the ES2-14 monitoring point in the middle-lower dam section, measuring 36.41 cm. The 3C sub-rockfill zone exhibited greater settlement than the 2BII main rockfill zone due to inferior material properties, with the maximum settlement (dark blue area) occurring in the downstream 3C zone. Reveals upstream horizontal displacement of 11.94cm in the middle-lower 2BI main rockfill zone (upstream direction) and downstream displacement of 12.09cm in the middle-lower 3C sub-rockfill zone (downstream direction). Due to panel variations and material parameter differences upstream, the horizontal displacement there was lower in elevation and smaller in magnitude compared to downstream measurements.

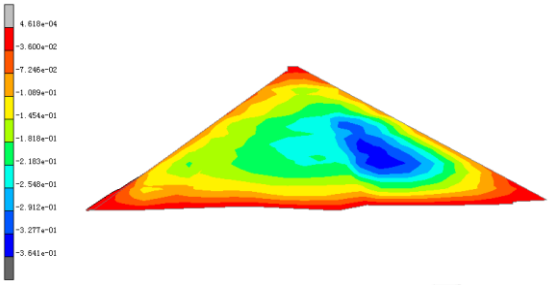

Figure 5 Sedimentation diagram of dam filling conditions completed (Unit: m)

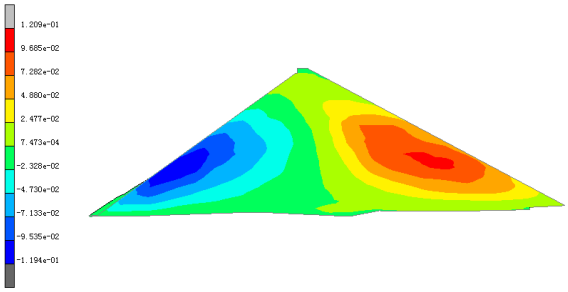

Figure 6 Horizontal displacement diagram of dam filling conditions completed (Unit: m)

V. CONCLUSION

Building upon existing research by domestic and international scholars, we selected a constitutive model for rockfill dam calculations. Using Fortran, we developed the computational code for this model through secondary development interfaces provided by MSC.Marc's HYPELA2 and CRPLAW. Additionally, we established a finite element model for a specific face rockfill dam in MSC.Marc, applied the secondary-developed constitutive model for computations, and validated both the feasibility and accuracy of the modified model.

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