

Research Article

Material Point Method-Based Simulation of Run-Out Characteristics for a Soil–Rock Mixed Landslide Induced by Surcharge

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Surcharge loading on slopes is a prevalent engineering practice that can precipitate landslides, posing significant risks to construction integrity and safety. This study elucidates the impact of surcharge loading on mixed soil–rock slopes and benchmarks their response against that of pure soil slopes under analogous loading conditions. Investigating damage manifestations, this research quantifies the distribution of plastic zones, the morphological alterations of slopes at characteristic stages, the dynamics of slip velocity at monitoring points, and the extent of landslide run-out. The material point method is adopted for its proficiency in simulating large deformation scenarios. Two-dimensional models of a representative soil–rock mixed slope and a pure soil slope are meticulously crafted using digital image processing techniques. The distinct damage profiles exhibited by the mixed and pure soil slopes are compared. The comparative assessment elucidates the distinct damage patterns of different slopes, enhancing the understanding of their behavior under variable surcharge intensities and contributing to the discourse on slope stability assessments.

1. Introduction

Human engineering activities, such as soil deposition on slopes and the construction of edifices upon them, have been identified as significant contributors to landslide occurrences in practical engineering scenarios [1-8]. This issue has garnered substantial attention from the academic community, with researchers from both national and international institutions dedicating efforts to investigate the phenomenon, thereby yielding a wealth of scholarly insights. Regarding laboratory experiments, a soil slope model was constructed by Ni et al. [9], through which the dynamics of landslide progression under various loading conditions were scrutinized. The findings from this study indicated that the phase of accelerated sliding in induced landslides is remarkably transient. The work of Liao et al. [10] encompassed the examination of strength-deformation characteristics of in situ loess under loading conditions. Utilizing both triaxial tests and numerical simulations, they explored stress and strength evolution on the most critical sliding surfaces during slope stacking. Employing the

discrete element method, Hu and Lu [11] simulated slope failure behavior consistent with observations from laboratory experiments. On the numerical front, Wu [12] applied the material point method (MPM) to model the kinematic features of mound-load-induced earthen landslides, with their parametric study delineating 29 distinct operational scenarios and culminating in the development of a predictive model for disaster causation in such landslides. Wu and Zhao [13] formulated a numerical model grounded in geological survey data and probed the contact behavior of elastic-plastic materials, thereby elucidating the deformation and damage mechanisms inherent to mound-load-induced landslides. Additionally, various scholars have delved into the deformation and damage mechanisms of artificially induced landslides through theoretical research, augmenting the understanding of the mechanisms governing this intricate occurrence [14-16].

Recent studies have predominantly concentrated on isotropic homogeneous soil slopes; however, nonhomogeneous slopes present a more common scenario in practical engineering applications [17–19]. This paper presents an examination

of the deformation and damage behaviors exhibited by nonhomogeneous slopes, specifically those comprising soil-stone mixtures subjected to heap loads. The soil-rock mixtures under consideration are primarily constituted of crushed stone and soil, which exhibit substantial strength disparities. This is largely attributed to the soil particle interconnectivity being considerably weaker than the cementation forces between the mineral constituents of the aggregate blocks [20]. The deformation and damage responses of the soil component are governed by the compaction of the interstitial voids and the relative displacement and slippage of particles. In contrast, the crushed stone is subject to microfracturing, pore space compression, and the initiation and propagation of cracks, culminating in material failure [21]. Owing to the heterogeneity in physical and mechanical properties among particles within the soil-rock mixture, the finer interactions between soil particles, as well as the macroscopic interactions between crushed stone and soil particles, manifest characteristics that diverge from those observed in typical soil or rock media [22, 23]. As a result, the stress-strain relationships and yield conditions pertinent to this class of mixtures are distinctly differentiated from those associated with homogeneous soil or rock.

The MPM is employed due to its effectiveness in handling large deformation simulations [24, 25]. Despite its advantages, MPM faces stability challenges, particularly when material points intersect computational grids, which can lead to errors or failure in the calculations [26]. Researchers have addressed these challenges by incorporating characteristic functions that stabilize the method when material points cross grids, drawing on ideas from the smoothed particle hydrodynamics (SPH) method [27]. Additionally, the use of higher order B spline basis functions has been investigated to improve the method's accuracy [2, 3, 28, 29]. The affine particle-in-cell (APIC) method has been proposed to enhance the mapping and projection processes within MPM, increasing computational stability [30]. A further refinement has been made with the PolyPIC format [31]. MPM's integration with other numerical methods such as the discrete element method (DEM) and the finite element method (FEM) has yielded a multiscale simulation approach that captures damage behavior at both macro and microlevels [32-34]. In practical applications, scholars have realized the MPM's superiority in several fields, including aerospace [35], explosion analysis [36], high-speed collision [37-39], biomechanics [40], and geotechnical engineering [41]. In the geotechnical field, the MPM has been specifically applied to the slope [6, 7, 42–45], dam [46], multifield coupling [47], and fracture analysis [48], yielding compelling results.

The study investigates the effects of surcharge loading on soil–rock mixed slopes, with a comparative analysis of pure soil slopes subjected to identical loading conditions. Emphasis is placed on delineating the response of these slopes by assessing various failure indicators such as the distribution of plastic deformation zones, alterations in slope morphology during critical phases, the evolution of slip velocities at specific observation points, and the maximum extent of debris dispersion postfailure. To capture the complexity of the damage



FIGURE 1: Material point method discrete format.

evolution, the research employs the MPM, a robust numerical approach tailored for simulations involving significant deformations. The slope models, including a standard soil–rock mixed slope and a pure soil counterpart, are obtained by digital image processing techniques. The entire process of landslide motion is recreated, and the material motion and deformation characteristics including slip velocity, slip distance, and plastic strain of the slope are recorded for analysis.

This paper is organized as follows: First, the MPM is described in Section 2. The model setup and the material parameters are then presented in Section 3. Section 4 outlines the simulation results given by MPM. Conclusion is drawn in the last section.

2. MPM

2.1. Methodology. The MPM evolved from the particle-in-cell (PIC) method [49–51]. Sulsky et al. [25] replaced the solution of the instanton equations with the solution on the mass points to overcome the challenge of considering event correlation when solving the instanton equations on the grid. Consequently, they proposed the MPM, which is more suitable for solid material simulation analysis. As illustrated in Figure 1, the MPM utilizes two description systems, namely, the material point and the grid, to characterize the simulated object's behavior during the simulation. The object is discretized into a series of Lagrangian material particles that move in the background grid. It is noteworthy that these two description methods have different significance in the MPM, with the material points occupying the primary position and the background grid occupying the secondary position. A new background grid is created at the start of each time step, which is then discarded at the end of the

computational step, ensuring that the grid does not store any information during the simulation. Thus, all information, including mass, momentum, energy, strain, and stress, is stored at the material point. After obtaining the information from the material point at each computational step, the momentum is updated on the grid, and boundary conditions are imposed.

In the present study, the classical MPM has been employed for the investigative analyses. Despite the superior stability and accuracy offered by the generalized interpolation MPM [26] and the B-sample MPM [29], these advanced techniques necessitate a heightened computational intensity. Considering the extensive computational demands of the research delineated in this paper, the utilization of piecewise linear shape functions alongside the updated stress first (USF) computational scheme [52] was deemed appropriate for the solution strategy. Thermal exchanges are not accounted for in this study; consequently, energy conservation is assured, and mass information is preserved within the material points, ensuring no loss of material information throughout the simulation. The weak form of the governing equations, along with the associated intrinsic equations, is expounded upon in the subsequent sections.

2.2. Discretization Formulations. As illustrated in Figure 1, employing a rectangular linear finite element background mesh to discretize the model, the density of the object can be approximated as follows:

$$\rho(\mathbf{x}_{\mathrm{i}}) = \sum_{p} m_{\mathrm{p}} \delta(\mathbf{x}_{\mathrm{i}} - \mathbf{x}_{\mathrm{ip}}), \qquad (1)$$

where m_p represents the mass of the material point, δ is the Dirichlet function, and x_{ip} denotes the coordinates of the material point. The equivalent integral weak form of the equation and the given surface force boundary condition are given by the following:

$$\int_{\Omega} \rho \,\ddot{u}_{i} \delta u_{i} dV + \int_{\Omega} \rho \sigma_{ij}^{s} \delta u_{i,j} dV - \int_{\Omega} \rho b_{i} \delta u_{i} dV - \int_{\Gamma_{t}} \rho \overline{t}_{i}^{s} \delta u_{i} dA = 0,$$

$$(2)$$

where δu_i denotes the imaginary displacement equal to 0 at the boundary Γ_{u} ; $\sigma_{ij}^s = \sigma_{ij}/\rho$ denotes the specific stress; $\overline{t}_i^s = \overline{t}_i/\rho$ denotes the boundary surface force and the imaginary displacement satisfies $\delta u_i \in \{\delta u_i | \delta u_i \in C^0, \delta u_i | \Gamma_u = 0\}$, in which C^0 denotes the set of continuous functions. According to Equation (2), we can transform the weak form of the control equation into the form of summation over the masses as follows:

$$\sum_{p} m_{p} \ddot{u}_{ip} \delta u_{ip} + \sum_{p} m_{p} \sigma^{s}_{ijp} \delta u_{ip} - \sum_{p} m_{p} b_{ip} \delta u_{ip} - \sum_{p} m_{p} \overline{t}^{s}_{ip} h^{-1} \delta u_{ip} = 0, \qquad (3)$$

where the subscript p denotes the physical quantity carried by the material point at the position x_{ip} , and h is the hypothetical boundary layer thickness, which is introduced to convert the last term at the left end of the weak form into a volume fraction.

Within each computational step of the MPM, the relationship between the material points and the background grid remains robust until the background grid is discarded. The transfer of information between these two components is accomplished via the interpolation of the shape function $N_{\rm I}(x_i)$, which is constructed on the background grid nodes. In this paper, the shape function is represented as a variable with subscript *I* for the variables on the background grid nodes. The specific form of the shape function $N_{\rm I}(x_i)$ is as follows:

$$N_{Ip} = \frac{1}{4} \left(1 + \xi_I \xi_p \right) \left(1 + \eta_I \eta_p \right), I = 1, 2, 3, 4, \tag{4}$$

where $\xi_p \in [-1, 1], \eta_p \in [-1, 1]$ denotes the natural coordinates of the material point *p*, and (ξ_p, η_p) is the natural coordinate value of the unit node, which takes the value $(\pm 1, \pm 1)$.

At the commencement of each computational step within the MPM framework, the weight of each material point with respect to the neighboring grid nodes is computed. Subsequently, the pertinent information from the material points is transferred onto the respective grid nodes in accordance with the calculated weights. Mathematically, the mass and momentum at a grid node are formulated as follows:

$$m_{\rm I} = \sum_{p} m_{\rm p} N_{I\rm p}, \ P_{Ii} = \sum_{p} m_{\rm p} v_{\rm pi} N_{I\rm p},$$
 (5)

where m_p and v_{pi} denote the mass and velocity of the material points, respectively.

In the context of the USF calculation format, the stresses at the material points are updated at the commencement of the calculation step. Following the acquisition of material point information, the grid is solved for the strain rate $\dot{\varepsilon}_{ijp}^{n-1/2}$ and the spin rate $\Omega_{ijp}^{n-1/2}$ of the material point based on the velocity gradient of the grid nodes, facilitating the determination of the objective stress rate, also known as the Jouman stress rate [53].

$$\dot{\varepsilon}_{ijp}^{n-1/2} = \sum_{I} \frac{1}{2} \left(S_{Ip,j}^{n} v_{iI}^{n-1/2} + S_{Ip,i}^{n-1/2} v_{jI}^{n-1/2} \right), \tag{6}$$

$$\Omega_{ijp}^{n-1/2} = \sum_{I} \frac{1}{2} \left(S_{Ip,j}^{n} v_{iI}^{n-1/2} - S_{Ip,i}^{n-1/2} v_{jI}^{n-1/2} \right),$$
(7)

$$\dot{\sigma}_{ij} = \sigma_{ij}^{\nabla} + \sigma_{ik}\Omega_{jk} + \sigma_{jk}\Omega_{ik}.$$
 (8)

In the above equations, $\sigma_{ij}^{\nabla} = C_{ijkl}^{\sigma J} \dot{e}_{kl}$ is the Jouran stress rate and $C_{ijkl}^{\sigma J}$ is the elastic stiffness tensor. Upon obtaining

the objective stress rate, we can already calculate the trial stress $\tilde{\sigma}_{ijp}^n = \sigma_{ijp}^{n-1} + \dot{\sigma}_{ijp}^{n-1/2} \Delta t$ based on the isotropic elastic intrinsic model. The return mapping algorithm [54] is then utilized to bring the trial stress to the Drucker–Prager criterion, followed by pulling any stress above the yield surface back to the yield surface to derive the true stress σ_{ijp}^n . The internal forces of the grid nodes can be calculated based on the obtained stress information, and external forces can be applied based on the boundary conditions. In this paper, only the influence of gravity is taken into account as an external force. After obtaining the nodal force information, the momentum equation can be solved on the grid nodes as follows:

$$f_{iI}^{\text{int}} = -\sum_{p} S_{Ip,j} \sigma_{ijp} \frac{m_{p}}{\rho_{p}}, \qquad (9)$$

$$f_{iI}^{\text{ext}} = \sum_{p} S_{Ip} m_{p} b_{ip} + \sum_{p} S_{Ip} \frac{m_{p}}{\rho_{p}} \overline{t}_{ip} h^{-1} \frac{m_{p}}{\rho_{p}}, \qquad (10)$$

where σ_{ijp} denotes the Cauchy stress of the material points, $N_{Ip,j}$ denotes the derivatives of shape functions, ρ_p denotes the density of the material points, b_{ip} denotes the body force such as gravity, $\overline{t}_i^s = \overline{t}_i / \rho$ represents the boundary surface force, and *h* represents the boundary layer thickness.

$$p_{iI}^{n+1/2} = p_{iI}^{n-1/2} + f_{iI}^n \Delta t, \qquad (11)$$

where $f_{il} = f_{il}^{\text{ext}} + f_{il}^{\text{int}}$, superscript represents the time step, and $p_{il}^{n+1/2}$ represents the resultant force of node force at the *n*th time step.

The terminal phase of the computational procedure encompasses the remapping of the updated momentum to the material point, followed by the revision of the material point's location. A hybrid momentum mapping approach, integrating features from both the PIC and FLIP techniques, is employed to map momentum information with the aim of enhancing stability. This hybridization serves to attenuate the instabilities typically associated with the FLIP method [55].

$$v_{\rm ip}^{n+1/2} = \alpha \sum_{I} P_{iI}^{n+1/2} N_{Ip}^{n} / m_{\rm I}^{n} + (1-\alpha) \left(v_{\rm ip}^{n-1/2} + \Delta t \sum_{I} \frac{f_{Ii} N_{Ip}}{m_{\rm I}} \right),$$
(12)

where the first term denotes PIC mapping, and the second term denotes FLIP mapping. $\alpha = 0.01$, which is known to ensure good stability based on prior research [30]. However, given the marked difference in strength between the soil and stone materials, we adopt a slightly larger value $\alpha = 0.02$ in this paper. Notably, it is important to emphasize that the momentum information associated with the PIC mapping pertains to the postupdate momentum of the grid nodes, whereas the momentum information associated with the FLIP mapping corresponds to the momentum increment over the time step.

The positions of the material points are updated as follows:

$$x_{\rm ip}^{n+1} = x_{\rm ip}^n + \Delta t v_{\rm ip}^{n+1},$$
(13)

where x_{ip}^n and x_{ip}^{n+1} denote the location of material points at *n* and n+1 time step, respectively.

3. Model Setup and Material Parameters

In this study, digital image processing methodologies were harnessed to develop a mixed soil–rock slope model. An image portraying the soil–rock mixed slope, derived from the work of Lianheng et al. [17], was digitized to capture the image information. The RGB color values were extracted and utilized to distinguish between stone and soil constituents, with pixels of distinct colors being designated as representative material points. To achieve the desired number of material points for the final model, the pixel count within the image was systematically reduced or augmented. Upon fulfillment of the model's criteria, the pixel coordinates were appropriated as the coordinates for the corresponding material points.

Figure 2 illustrates the finalized model of the slope, where distinct colors represent different materials: earth yellow for the mound carrier, blue for soil, and black for stone. The model is composed of 45,697 material points, with stone constituents forming approximately 20% of the total. More precisely, the model comprises 9,207 material points attributed to stone, while the soil is represented by 36,490 material points. The size distribution of the stones is aligned with the observations made by Lianheng et al. [17], where stones with diameters exceeding 0.8 m and those below 0.2 m constitute 6% and 5.5% of the total stone volume, respectively. The predominant diameter range for stones in the model spans from 0.4 to 0.7 m, adhering approximately to a normal distribution pattern.

Table 1 displays the material parameters for both the slope and the heap carrier. The mechanical properties of the soil are mirrored in the heap carrier, save for the density, where the heap carrier exhibits a marginally higher value when compared to that of the soil.

For the computational analysis conducted, the opensource code from the research by the Soga team was utilized as a foundation, which was then enhanced to suit the specific requirements of the present study [56]. The initialization of the slope's state was achieved through the adoption of a linear incremental gravity model prior to the commencement of the simulations. Boundary conditions were rigorously defined: Vertical displacements at the base were constrained; lateral movements were restricted on the left boundary; and the right boundary was set to limit rightward movements, specifically at the base of the slope within a vertical extent of 0–3.2 m. As illustrated in Figure 3, gravity was incrementally increased from zero to half the terminal value, T, before reaching and sustaining the target magnitude, thereby ensuring a steady state from t = T onward. This methodical escalation of gravity was strategically employed to circumvent the stress oscillations



FIGURE 2: Geometric parameters of soil-rock mixed slope model (45,697 material points in total; 36,490 MPs for soil; 9,207 MPs for stone).

Parameters	Soil	Stone	Stacked objects
Density, ρ (kg/m ³)	1,800	2,410	2,000
Young's modulus, E (MPa)	50	20,000	50
Poisson's ratio, ν	0.35	0.2	0.35
Internal friction angle, ϕ (°)	24	42	24
Cohesion, c (kPa)	30	900	30
Dilatancy angle, ψ (°)	24	42	24
Constitutive model	Drucker–Prager	Drucker–Prager	Elastic





FIGURE 3: Incrementation of gravity over the period T.

typically induced by its abrupt application. The initial state resolution was executed using the pure PIC momentum mapping method. Despite its known limitations, the method's numerical stability was leveraged to curtail oscillations effectively.

In the conducted simulation, a temporal resolution was achieved by adopting a time step of 0.01 ms for all computational increments. The background mesh was configured with a grid dimension of 0.1 m, and each cell within this grid was populated with four material points, uniformly positioned with an initial interpoint spacing of 0.05 m. The simulation spanned over a period of 25 s, culminating in a total of 2,000,000 discrete time steps. To ensure a consistent application of the pile load atop the slope, the pile carrier was constrained to behave in an elastic manner throughout the simulation duration. For the purpose of calibrating the soil and rock constituents within the slope, the Drucker–Prager (D–P) model was utilized to simulate elastic test stresses.

4. Simulation Results

4.1. Stability of Slopes. Figure 4 delineates the maximum displacement responses of the slope under varying top loads. It is observed that the soil–rock mixed slope retains superior stability relative to a uniform top load. At a top load of 352.8 kPa, the maximum displacement registered is approximately 0.08 m, indicative of the slope's natural settlement attributed to its own weight. This suggests that the external load does not exert a detrimental effect on the stability of the slope. In contrast, a top load of 313.6 kPa results in a maximum displacement of roughly 1.2 m, signifying substantial deformation and the onset of complete destabilization damage. Moreover, as the top load is increased to 352.8 kPa, the



FIGURE 4: The maximum displacement of the slope at different top loads.



FIGURE 5: Kinetic energy curves of the pure soil slope under different stack loadings: (a) kinetic energy curves for various PPC scenarios; (b) kinetic energy curves for various dx scenarios.

displacement escalates to a peak value of about 5.1 m. These observations underscore the considerable inaccuracies that can arise from approximating the actual heterogeneous slope to an isotropic pure soil model for the purpose of stability analysis.

Figure 5 displays the time-dependent curves of the slope body's kinetic energy for top pile loads of 313.6 and 352.8 kPa. Several groups of examples under the same conditions are made to demonstrate how mesh size and the spacing of the material points influence the simulation results. In the figures, PPC represents the number of particles per cell, with a larger PPC indicating a smaller spacing between particles. When studying the effect of PPC variation on simulation results, the mesh size is fixed at 0.1 m. As can be seen from Figure 5(a), the simulation results are almost identical when PPC changes, indicating that particle spacing

does not affect the simulation results at the current mesh resolution. When investigating the impact of mesh size on simulation results, the particle spacing is fixed at 0.05 m. From Figure 5(b), it is observed that, as the mesh size increases, the simulated kinetic energy curve decreases slightly, suggesting that a reduction in mesh resolution does affect the calculation results, although the impact is minimal.

The analysis of kinetic energy variations reveals several key observations. Initially, it is noted that the peak kinetic energy of the pure soil slope demonstrates a linear escalation in response to an increasing top load. Subsequently, upon exceeding the bearing capacity of the slope, catastrophic failure ensues, and a rapid surge to peak kinetic energy is observed in the system. Following this surge, the slope is seen to reattain stability under a reduced top load, with the duration of landslide activity lasting in the vicinity of 5 s. In



FIGURE 6: Plastic zone as well as displacement distribution at the top load equals to 313.6 kPa. ((a: a_1 and a_2)) show the displacement distribution and plastic zone distribution of pure soil slope; (b: (b_1 and b_2)) show the displacement distribution and plastic zone distribution of the mixed soil–rock slope.

the case of a larger top load, however, the landslide duration prolongs to approximately 16 s, featuring a pronounced stable sliding phase within the decay process.

Figure 6 offers a critical comparison between the behavioral responses of two types of slopes subjected to identical mounding loads. When the top pile load imposed on the pure soil slope reaches its threshold, a uniform and maximum displacement occurs across the slope body, which translates as a whole. The sliding region is demarcated by a sliding zone characterized by the utmost plastic deformation. Below this zone, the slope body maintains stability, exhibiting no signs of plastic deformation. The distribution of plastic regions within the pure soil slope is predominantly confined to the sliding area and the leading edge of the mound carrier. In contrast, when subjected to an equivalent load, the soil-rock mixed slope exhibits a markedly higher degree of stability, with a maximum displacement not exceeding 0.1 m. The plastic deformation is mainly localized at the lower portion of the mound carrier, demonstrating minimal plastic deformation elsewhere.

4.2. Dynamics of Soil–Rock Mixed Slopes under Ultimate Mounding Load. Figure 7 illustrates the deformation response of the soil–rock mixed slope to an incrementally increased top pile load. It is observed that as the loading



FIGURE 7: Maximum displacement curve of soil-rock mixed slope.

magnitude escalates, the slope undergoes pronounced deformation. The maximum displacement of the slope rapidly reaches a peak prior to the application of a 513.5 kPa load, after which no further change is detected, indicating a swift



FIGURE 8: Displacement curve of soil-rock mixed slope at the top stack load equals to 513.5 kPa and kinetic energy change curve. (a): (a₁) displacement curves for various PPC scenarios; (a₂) displacement curves for various dx scenarios. (b): (b₁) kinetic energy curves for various PPC scenarios; (b₂) kinetic energy curves for various dx scenarios.

restabilization without subsequent notable damage. At the critical load of 513.5 kPa, the displacement recorded during the initial 0–10 s interval mirrors the displacement at 509.6 kPa. Post the 10-s mark, however, the slope is subjected to significant displacement, and by the 22-s time point, no indications of impending stabilization are discernible.

To rigorously examine the behavior of a soil–rock mixed slope under a load of 513.5 kPa, the duration of the simulation was extended to 32 s, inclusive of the initial 2 s allocated for establishing the baseline state. This facilitated the generation of curves depicting the evolution of displacement and kinetic energy within the slope, as delineated in Figure 8. Comparative simulations were performed under consistent conditions to assess the effects of mesh granularity and material point distribution on the fidelity of the simulation results. The ensuing data suggest a conclusion parallel to that observed in the pure soil slope analyses: The particle count per cell (PPC) exerts minimal impact on the outcome of the simulations, whereas an increase in mesh spacing has a marginally detrimental effect on the precision of the simulated results.

Figures 4 and 8 elucidate the differential deformation and damage responses exhibited by the soil–rock mixed slope as opposed to the pure soil slope when subjected to the ultimate load. It is observed that the displacement curve for the pure soil slope exhibits a continuous rise over time until it reaches a plateau under the ultimate load condition. By contrast, the soil–rock mixed slope displays an initial phase of displacement increase, succeeded by a period of stability lasting approximately 10 s. Subsequent to this phase, a pronounced



FIGURE 9: The rate distribution clouds of the soil—rock mixed slope during the damage process: (a–f) indicate the rate clouds at 1, 8, 10, 14, 18, and 25 s, respectively.

escalation in the maximum displacement of the slope is noted, which then proceeds to a gradual state of equilibrium post the 25-s mark. It is noteworthy that the kinetic energy curve's trajectory shows a close correlation with that of the displacement curve.

Figure 9 presents the velocity distribution contours for the soil–rock mixed slope at various critical junctures throughout the damage progression. In particular, Figure 9(a)) corresponds to the initial acceleration stage, Figure 9(b) to the stabilization stage, while Figures 9(c) and 9(d) correspond to the secondary acceleration stage. Figures 9(e) and 9(f) represent the deceleration and subsequent stabilization stages, respectively. It is observed that the soil–rock mixed slope attains its maximum velocity during the secondary acceleration phase, with velocities reaching an approximate magnitude of 1.8 m/s.

4.3. Final Configuration of Soil–Rock Mixed Slope under Ultimate Mounding Load. Subsequent to the stabilization of the slope, figures were generated to delineate the distribution of plastic zones, the pattern of displacement, and the slope's ultimate morphology. These illustrative figures are provided in Figures 10 and 11.

The analysis of Figure 10(a) reveals that the soil–rock mixed slope, upon initial stabilization, developed a continuous sliding zone and multiple plastic zones with a crossdistributed pattern, characteristic of significant rock-binding effects. It is important to note that the continuous sliding zone emerged not at the slope's base but rather at an elevated position proximal to the foot of the slope. When juxtaposed with the preliminary model depicted in Figure 2, it becomes evident that the dispersal of stones at the base of the slope served as a barrier, impeding the downward progression of the plastic zone into this region.

In the poststabilization phase of the soil-rock mixed slope, as depicted in Figure 10(b), plastic deformation was observed to have reached its full extent. The slope mass above the continuous sliding zone, along with the mound carrier, underwent substantial displacement, excluding the slope mass at the mound carrier's leading edge, which



FIGURE 10: Distribution of plastic zone of soil-rock mixed slope at ultimate stacking load: (a) distribution of plastic zone at t = 10 s (first stabilization stage); (b) distribution of plastic zone at t = 30 s.



FIGURE 11: The final stage of the soil-rock mixed slope at the ultimate stack load: (a) displacement distribution at t = 30 s; (b) slope configuration at t = 30 s.

retained its structural integrity. In contrast, the remainder of the sliding mass displayed pronounced fragmentation. Subsequent to stabilization, as illustrated in Figure 11, the maximum displacement was recorded at approximately 6.0 m, predominantly concentrated at the slope's forefront adjacent to the pile carrier. The displacement observed in the pile carrier and its underlying slope was comparatively minor. The results underscore the influence of stone distribution at the base of the slope on the stability and deformational response of the soil-rock mixed slope. The sliding and plastic zones, which were initially identified during the early stabilization stage, were instrumental in defining the ultimate deformation and fragmentation patterns observed upon final stabilization. These insights carry significant implications for the design and maintenance of soil-rock mixed slopes in engineering applications.

5. Conclusion

In this study, the deformation response of soil-rock mixed slopes subjected to stacking loads is investigated utilizing the MPM. To enhance the fidelity of the soil-rock mixed model simulation, digital image processing techniques were employed to construct a model that aligns more closely with the actual heterogeneous block distribution. For the sake of computational efficiency, the classical MPM was selected, and a mixed mapping scheme with small time increments was implemented to maintain stability throughout the computational process. The dynamic response of the slope, along with its deformation and damage characteristics, was effectively captured by simulating various operational conditions under differing top pile loads. The research culminated in a set of conclusions based on the empirical data gathered.

(1) The comparative analysis indicates enhanced stability for soil–rock mixed slopes under top-loading conditions as opposed to pure soil slopes. It was determined that, upon reaching the loading capacity limit, pure soil slopes demonstrate immediate sliding and a rapid ascent to peak kinetic energy, with the peak magnitude displaying a linear dependency on the imposed load. Postpeak kinetic energy decay varies with the size of the stacking load, with larger loads precipitating a decay characteristic that encompasses a phase of stabilized sliding. Contrarily, the soil–rock mixed slopes show a markedly reduced sensitivity to the ultimate mounding load, suggesting an inherent robustness in their stability profile.

- (2) When the soil-rock mixed slope is subjected to its threshold top pile load, it is not subjected to instantaneous landslide damage. Instead, it undergoes a period of stability following the initial displacement increase, which precedes a severe landslide event. It is during the second phase of sliding that the maximum landslide velocity is observed.
- (3) The spatial distribution of boulders within the soil-rock mixed slope critically dictates the emergence of the primary sliding zone when damage occurs. Furthermore, the shear zones manifest in a staggered pattern rather than a uniform distribution. The resultant damage is characterized not by a cohesive sliding motion but by a pronounced fragmentation of the slope material.

Data Availability

You can obtain the data included in the article by contacting the corresponding author.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

References

- G. Lv, Y. He, and B. Wei, "Dynamic stability analysis of slope subjected to surcharge load considering tensile strength cut-off," *Mathematical Problems in Engineering*, vol. 2020, Article ID 5196303, 17 pages, 2020.
- [2] K. Zhang, S.-L. Shen, A. Zhou, and D. Balzani, "Truncated hierarchical B-spline material point method for large deformation geotechnical problems," *Computers and Geotechnics*, vol. 134, Article ID 104097, 2021.
- [3] L. Zhang, X. Wang, T. Xia, B. Yang, and B. Yu, "Deformation characteristics of Tianjiaba landslide induced by surcharge," *ISPRS International Journal of Geo-Information*, vol. 10, no. 4, Article ID 221, 2021.
- [4] T. S. Nguyen, K.-H. Yang, Y.-K. Wu, F. Teng, W.-A. Chao, and W.-L. Lee, "Post-failure process and kinematic behavior of two landslides: case study and material point analyses," *Computers and Geotechnics*, vol. 148, Article ID 104797, 2022.
- [5] T. S. Nguyen, K.-H. Yang, C.-C. Ho, and F.-C. Huang, "Postfailure characterization of shallow landslides using the material point method," *Geofluids*, vol. 2021, Article ID 8860517, 25 pages, 2021.
- [6] A. Troncone, L. Pugliese, and E. Conte, "Analysis of an excavation-induced landslide in stiff clay using the material point method," *Engineering Geology*, vol. 296, Article ID 106479, 2022.
- [7] A. Troncone, L. Pugliese, A. Parise, and E. Conte, "A simple method to reduce mesh dependency in modelling landslides involving brittle soils," *Géotechnique Letters*, vol. 12, no. 3, pp. 167–173, 2022.
- [8] E. Conte, L. Pugliese, and A. Troncone, "A simple method for predicting rainfall-induced shallow landslides," *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 148, no. 10, 2022.

- [9] P. Ni, S. Wang, S. Zhang, and L. Mei, "Response of heterogeneous slopes to increased surcharge load," *Computers* and Geotechnics, vol. 78, pp. 99–109, 2016.
- [10] H. J. Liao, L. J. Su, Z. D. Li, Y. B. Pan, and H. Fukuoka, "Testing study on the strength and deformation characteristics of soil in loess landslides," in *Landslides and Engineered Slopes. From the Past to the Future*, Z. Chen, J.-M. Zhang, K. Ho, F.-Q. Wu, and Z.-K. Li, Eds., vol. 2, CRC Press, 1st edition, 2008.
- [11] Y. Hu and Y. Lu, "Study on soil–rock slope instability at mesoscopic scale using discrete element method," *Computers* and Geotechnics, vol. 157, Article ID 105268, 2023.
- [12] F. Wu, "Run-out characteristic simulation of a surcharge-induced soil landslide using the material point method," *Hydrogeology & Engineering Geology*, vol. 44, no. 6, Article ID 126, 2017.
- [13] W. Wu and J. Zhao, "A dynamic-induced direct-shear model for dynamic triggering of frictional slip on simulated granular gouges," *Experimental Mechanics*, vol. 54, no. 4, pp. 605–613, 2014.
- [14] E. X. Song, *Elasto-plastic consolidation under steady and cyclic loads*, Ph.D. thesisDelft University of Technology, 1990.
- [15] N. Coli, P. Berry, and D. Boldini, "In situ non-conventional shear tests for the mechanical characterisation of a bimrock," *International Journal of Rock Mechanics and Mining Sciences*, vol. 48, no. 1, pp. 95–102, 2011.
- [16] H. Liu, B. Xiong, G. Yang, and Y. Zhou, "Field instrumentations of a two-tiered geogrid-reinforced soil wall backfilled with soil–rock mixture," 2013.
- [17] Z. Lianheng, H. Dongliang, Z. Shuaihao, C. Xiao, L. Yibo, and D. Min, "A new method for constructing finite difference model of soil–rock mixture slope and its stability analysis," *International Journal of Rock Mechanics and Mining Sciences*, vol. 138, Article ID 104605, 2021.
- [18] E. W. Medley and P. F. S. Rehermann, "Characterization of bimrocks (rock/soil mixtures) with application to slope stability problems," in *Eurock 2004 & 53rd Geomechanics Colloquium Salzburg, Austria*, 2004.
- [19] W. Xu, "Conception, classification and significations of soil–rock mixture," *Hydrogeology & Engineering Geology*, vol. 36, no. 4, pp. 50–56, 2009.
- [20] W.-J. Xu, Y. J. Wang, Z. Y. Chen, and H. U. Rui-Lin, "Stability analysis of soil–rock mixed slope based on digital image technology," *Rock and Soil Mechanics*, vol. 29, no. S1, pp. 341– 346, 2008.
- [21] X. Wen-Jie, X. Qiang, and H. Rui-Lin, "Study on the shear strength of soil–rock mixture by large scale direct shear test," *International Journal of Rock Mechanics and Mining Sciences*, vol. 48, no. 8, pp. 1235–1247, 2011.
- [22] Y. Hu and M. F. Randolph, "A practical numerical approach for large deformation problems in soil," *International Journal for Numerical and Analytical Methods in Geomechanics*, vol. 22, pp. 327–350, 1998.
- [23] L. Yu, J. Liu, X. J. Kong, and Y. Hu, "Three-dimensional RITSS large displacement finite element method for penetration of foundations into soil," *Computers and Geotechnics*, vol. 35, no. 3, pp. 372–382, 2008.
- [24] D. Sulsky, S.-J. Zhou, and H. L. Schreyer, "Application of a particle-in-cell method to solid mechanics," *Computer Physics Communications*, vol. 87, no. 1-2, pp. 236–252, 1995.
- [25] D. Sulsky, Z. Chenb, and H. L. Schreyer, "A particle method for history-dependent materials," *Computer Methods in Applied Mechanics and Engineering*, vol. 118, no. 1-2, pp. 179–196, 1994.
- [26] S. G. Bardenhagen and E. M. Kober, "The generalized interpolation material point method," *Computer Modeling in Engineering & Sciences*, vol. 5, no. 6, pp. 477–495, 2004.

- [27] H. H. Bui, R. Fukagawa, K. Sako, and S. Ohno, "Lagrangian meshfree particles method (SPH) for large deformation and failure flows of geomaterial using elastic–plastic soil constitutive model," *International Journal for Numerical and Analytical Methods in Geomechanics*, vol. 32, no. 12, pp. 1537–1570, 2008.
- [28] Y. Gan, Z. Sun, Z. Chen, X. Zhang, and Y. Liu, "Enhancement of the material point method using B-spline basis functions," *International Journal for Numerical Methods in Engineering*, vol. 113, no. 3, pp. 411–431, 2018.
- [29] R. Tielen, E. Wobbes, M. Möller, and L. Beuth, "A high order material point method," *Procedia Engineering*, vol. 175, pp. 265–272, 2017.
- [30] C. Jiang, C. Schroeder, A. Selle, and J. Teran, "The affine particle-in-cell method," ACM Transactions on Graphics, vol. 34, no. 4, pp. 1–10, 2015.
- [31] C. Fu, Q. Guo, T. Gast, C. Jiang, and J. Teran, "A polynomial particle-in-cell method," ACM Transactions on Graphics, vol. 36, no. 6, pp. 1–12, 2017.
- [32] W. Liang and J. Zhao, "Multiscale modeling of large deformation in geomechanics," *International Journal for Numerical and Analytical Methods in Geomechanics*, vol. 43, no. 5, pp. 1080– 1114, 2019.
- [33] Y. P. Lian, X. Zhang, and Y. Liu, "Coupling of finite element method with material point method by local multi-mesh contact method," *Computer Methods in Applied Mechanics* and Engineering, vol. 200, no. 47-48, pp. 3482–3494, 2011.
- [34] Y. P. Lian, X. Zhang, and Y. Liu, "An adaptive finite element material point method and its application in extreme deformation problems," *Computer Methods in Applied Mechanics and Engineering*, vol. 241–244, pp. 275–285, 2012.
- [35] X. X. Cui, X. Zhang, X. Zhou, Y. Liu, and F. Zhang, "A coupled finite difference material point method and its application in explosion simulation," *Computer Modeling in Engineering & Sciences*, vol. 98, no. 6, pp. 565–599, 2014.
- [36] W. Hu and Z. Chen, "Model-based simulation of the synergistic effects of blast and fragmentation on a concrete wall using the MPM," *International Journal of Impact Engineering*, vol. 32, no. 12, pp. 2066–2096, 2006.
- [37] P. Liu, Y. Liu, and X. Zhang, "Improved shielding structure with double honeycomb cores for hyper-velocity impact," *Mechanics Research Communications*, vol. 69, pp. 34–39, 2015.
- [38] Z. Ye, X. Zhang, G. Zheng, and G. Jia, "A material point method model and ballistic limit equation for hyper velocity impact of multi-layer fabric coated aluminum plate," *International Journal of Mechanics and Materials in Design*, vol. 14, no. 4, pp. 511–526, 2018.
- [39] X. Zhang, K. Y. Sze, and S. Ma, "An explicit material point finite element method for hyper-velocity impact," *International Journal for Numerical Methods in Engineering*, vol. 66, no. 4, pp. 689–706, 2006.
- [40] A. De Vaucorbeil, V. P. Nguyen, S. Sinaie, and J. Y. Wu, "Material point method after 25 years: theory, implementation, and applications," in *Advances in Applied Mechanics*, vol. 53, pp. 185–398, Elsevier, 2020.
- [41] J. Fern, A. Rohe, K. Soga, and E. Alonso, "The material point method for geotechnical engineering: a practical guide," in *The Material Point Method for Geotechnical Engineering*, J. Fern, A. Rohe, K. Soga, and E. Alonso, Eds., CRC Press, 2019.
- [42] K. Soga, E. Alonso, A. Yerro et al., "Trends in large-deformation analysis of landslide mass movements with particular emphasis on the material point method," *Géotechnique*, vol. 68, no. 5, pp. 457-458, 2018.

- [43] A. Troncone, L. Pugliese, A. Parise, and E. Conte, "A practical approach for predicting landslide retrogression and run-out distances in sensitive clays," *Engineering Geology*, vol. 326, Article ID 107313, 2023.
- [44] X. Li, P.-F. Xie, Y. Zheng et al., "Modeling of soil–rock mixture landslides with the generalized interpolation material point method," *Frontiers in Earth Science*, vol. 10, 2022.
- [45] L. Zhao, N. Qiao, D. Huang, S. Zuo, and Z. Zhang, "Numerical investigation of the failure mechanisms of soil–rock mixture slopes by material point method," *Computers and Geotechnics*, vol. 150, Article ID 104898, 2022.
- [46] X. Zhao, D. Liang, and M. Martinelli, "MPM simulations of dam-break floods," *Journal of Hydrodynamics*, vol. 29, no. 3, pp. 397–404, 2017.
- [47] G. Wang, F. Sun, R. Wang, L. Zhang, T. Cao, and B. Li, "Simulation of heat and fluid flow in porous medium and fractures by material point method," *International Journal of Numerical Methods for Heat & Fluid Flow*, vol. 32, no. 10, pp. 3328–3359, 2022.
- [48] F. Sun, G. Wang, L. Zhang, R. Wang, T. Cao, and X. Ouyang, "Material point method for the propagation of multiple branched cracks based on classical fracture mechanics," *Computer Methods in Applied Mechanics and Engineering*, vol. 386, Article ID 114116, 2021.
- [49] J. U. Brackbill, D. B. Kothe, and H. M. Ruppel, "Flip: a lowdissipation, particle-in-cell method for fluid flow," *Computer Physics Communications*, vol. 48, no. 1, pp. 25–38, 1988.
- [50] J. U. Brackbill and H. M. Ruppel, "FLIP: a method for adaptively zoned, particle-in-cell calculations of fluid flows in two dimensions," *Journal of Computational Physics*, vol. 65, no. 2, pp. 314–343, 1986.
- [51] F. H. Harlow, "The particle-in-cell computing method for fluid dynamics," M.S. thesis, , The University of California, 1964.
- [52] S. G. Bardenhagen, "Energy conservation error in the material point method for solid mechanics," *Journal of Computational Physics*, vol. 180, no. 1, pp. 383–403, 2002.
- [53] X. Zhang, Z. Chen, and Y. Liu, The Material Point Method: A Continuum-Based Particle Method for Extreme Loading Cases, Academic Press, 2016.
- [54] O. Gonzalez and A. M. Stuart, A First Course in Continuum Mechanics, Cambridge University Press, 2008.
- [55] Y. Zhu and R. Bridson, "Animating sand as a fluid," ACM Transactions on Graphics, vol. 24, no. 3, pp. 965–972, 2005.
- [56] K. Kumar, J. Salmond, S. Kularathna et al., "Scalable and modular material point method for large-scale simulations," in 2nd International Conference on the Material Point Method for Modelling Soil–Water–Structure Interaction, MPM Research Community, September, 2019.