Zonation and Stability Analysis of Toppling-Deformed Slope Based on Discrete Element Method

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Abstract. This study presents a toppling-deformed slope located at the entrance of the Tonghua 1# tunnel of Wenchuan-Maerkang highway in southwestern Sichuan, China. The characteristics, zonation, and failure mechanism of this toppling-deformed shape were explored based on field investigations, geological mapping, and numerical simulation. The results suggested that the prerequisites for toppling deformation were the typical rock structure and special lithologic composition. The studied slope can be zoned into three areas: falling area, strongly toppling-deformed area, and slightly toppling-deformed area. Numerical modeling based on the discrete element method indicated that strongly toppling-deformed area exhibits the largest displacement on the bottom, with a total displacement of 5.74 cm in the natural scenario, 9.07 cm in rainfall scenario, and 1.44 m in earthquake scenario. The slope is unstable and damaged under earthquake conditions. The slope’s failure mode can be summarized as follows: the earthquake caused the slope toe to collapse→the retrogressive deformation occurred→the slope surface collapsed and the sliding surface gradually penetrated→the deformed slope accelerated deformation→the deformed slope caused the overall failure.”. The results also showed that this toppling slope has not yet advanced to the later stages of progressivity failure and is currently limited to collapsing at shallow levels. This study can provide an insight into toppling-deformed slope failure in the construction areas associated with the combination of seismic activities.

1. Introduction
Rock slope toppling is a particular geological hazard characterized by the rotation of columns or rock blocks about a fixed end[1]2. Toppling failures are ubiquitous in natural or excavated slopes with stratified rock masses. The common causing mechanisms of toppling failures mainly comprise tectonic activity, heavy rainfall, and anthropologic disturbance. Many studies focused on the characteristics, slope stability, and mechanisms of rock slope toppling[3-5]. Toppling failure causes vast deep-seated landslide slopes to form more easily than other slope failure modes and should be carefully considered. These slope failures cause severe casualties and significant economic losses. Therefore, examining the slope stability is extremely important to study and prevent such geological disasters[6][7].

The identification of toppling failure has dominated the engineering practice of tunnel entrance and exit slopes in southwest China for decades. Several methods, including the limit equilibrium method[8][9], finite element method[10], and discrete element method (DEM)[11][12] have been developed
and tested for studying toppling slopes. DEM is used to analyze the deformation of discrete media, motion trend, and the issue of rigid or deformable block separation. This is a more suitable method for discontinuous rock stability analysis and has been widely used in the study of mining, high slope, highway, earthquake, and blasting\textsuperscript{[13]}. In addition, physical modeling, such as tilting tables and centrifuge testing, was applied to reveal the mechanism of this type of disaster. Numerical modeling techniques, among others, have significantly improved rock slope analysis, especially for structurally controlled slope toppling. Many researchers have used the universal distinct element code (UDEC), which can analyze deformation tendency by considering both continuous and discrete approaches\textsuperscript{[14]}, to model the rock slope toppling.

This study analyzes the toppling-deformed slope stability in a tunnel’s entrance and exit using detailed investigations and DEM modeling, with a focus on the characteristics and earthquake controls of this type of slope that may be expected at a specific site in the Wenchuan-Maerkang (Wen-Ma) highway. The entrance slope of Tonghua 1\# tunnel is situated in the Tonghua Village, Li County, Sichuan Province, China. This study employs a field investigation to identify the slope’s zonation. UDEC software was implemented under natural, rainfall, and earthquake scenarios to evaluate the stability of this toppling slope. The stability of toppling-deformed slopes is a major engineering geological problem in highway construction and operation. Analysis of its deformation mechanism and potential failure mode, as well as adequate evaluation of its stability, is significant for remedial works and hazard mitigation of rock slopes around the tunnel.

2. Study area
The Tonghua 1\# tunnel is located at the K71 + 100 section of Wen-Ma highway, belonging to the Tonghua Village, Li County, Sichuan Province in China (Figure 1). The deformed slope is located on the right side of the tunnel entrance, and the left bank of the Zagunao River. The study area lies in the southern segment of the mountain, with an elevation of 1508–1640 m. Intense and frequent earthquakes occurred in this region due to strong tectonic movements. The slope has an aspect of 155°, a slope gradient of 35°–60°, characterized by being gentle in the upper part and steep in the lower part of the terrain. The strata are composed of Silurian and Quaternary lithology. The rocky slope consists of phyllite belonging to Maoxian formation (S\textsubscript{ma}), with well-developed cleavage and an initial occurrence of 355° \v 77°. The slope is fractured by two joint sets (J1 and J2), which cut the rock mass into blocks. J1 strikes southeast and dips to the southwest at 68°. J2 strikes southeast and dips to the southwest at 45°. The unconsolidated strata include gravel and cobble soil of colluvium, alluvium, proluvium, and eluvium.
3. Characteristics and zonation of the toppling slope

3.1 Overview of the toppling slope
The toppling-deformed slope had a longitudinal length of 130–140 m, a height difference of 120–130 m, and a width of 120–140 m (Figure 2). It covered an area of $1.8 \times 10^4 \text{ m}^2$, with an average thickness of 25 m and a volume of $45 \times 10^4 \text{ m}^3$. The rear edge was located in the slope break zone, with an elevation of about 1630 m. Rock blocks and boulders were accumulated at the slope toe, with an elevation of about 1510 m. Lateral edges were bounded by two gullies. A mass of rock boulders and blocks accumulated in the slope toe and gullies, with a rock block ratio of 50%, gravel ratio of 10%, and size of boulders larger than 1 m of 40%, with the maximum size of 4.0 m.

3.2 Zonation of slope
Field investigations indicated that rock masses in slope breaks and mountain ridges are strongly toppled and deformed, resulting in fractured rock masses, with frequent rockfalls occurring in these areas (Figure 3). However, rock masses are slightly deformed in negative terrain sections such as lateral gullies and gentle slopes. The bedrock’s normal occurrence is $355^\circ \pm 77^\circ$. Based on the field investigation, we determine the rock layer dip angle less than 17° as the falling rock mass, the dip angle between 17°–45° as strongly toppling rock mass, and the dip angle between 45°–70° as slightly toppling rock mass. The toppling-deformed slope can be divided into the falling area (Area A), the strongly toppling-deformed area (Area B), and the slightly toppling-deformed area (Area C) based on the deformation degree and inclination angle of the toppling rock layer as a quantitative indicator.
3.2.1 Falling area (Area A)

The falling area was located in the upper and middle section of the deformed slope, with a longitudinal length of about 5 m, a width of about 10 m, and a depth of about 6.5 m. The inclination angle of the deformed rock masses was less than 17°, with an angle difference of approximately 60° with the original bedrock. Dangerous rocks were well developed, presenting high weathering and low stability. Moreover, rockfalls and talus were observed in the foot of the slope, with rock blocks sizes of 1–3 m. Area A mainly consisted of the residual masses of an early failure event. Dangerous rock mass rockfalls may occur under heavy rain or earthquake conditions. The instability of dangerous rock masses in this area will cause direct damage to the Wen-Ma Expressway’s construction and operation.

3.2.2 Strongly toppling-deformed area (Area B)

The strongly toppling-deformed area was located in the middle and lower section of the deformed slope, with a length of about 25–50 m, a width of about 70 m, and a depth of about 10–20 m (Figures 2 and 3). This area had low vegetation coverage, and the slope gradient was larger than 45°. The deformed rock mass inclination angle was about 17°–45°, which is about 32°–60° different from the original bedrock. The rock masses exhibited poor integrity and high weathering, influenced by unloading, local looseness occurrence. Compared with the deformed slope’s left side, the toppling deformation on the right side was higher and the stability was relatively lower. Locally protruding rock masses in this area may collapse and fall under heavy rain or earthquake conditions. The rock masses are severely disintegrated, interlocked, and heavily broken due to a high degree of deformation at the ridges. Particularly, the disturbance to the slope toe during the tunnel construction process may aggravate the slope deformation, resulting in an overall slide along the fracture surface.

3.2.3 Slightly toppling-deformed area (Area C)

The slightly toppling-deformed area is distributed in the upper left part of the deformed slope with a length of about 15–30 m, a width of about 70 m, and a depth of about 20–45 m, and the vegetation in
this area is sporadic. This area has a gentler slope than area B, with a value of 35°–45°. The deformed rock layer inclination is 45°–70°, which is about 7°–32° different from the normal bedrock inclination. The fracture surface exhibits a step-like shape, developing from the bottom to the top. The rock mass integrity is relatively blocky. The local relaxation and highly weathering phenomenon are pervasive under the unloading action.

The colluvial gravel and soil were distributed in the slope’s lower part and the gullies indicated that this area was the residual part of an earlier failed slope. The slopes of the middle and lower slopes were relatively steep from the topographical perspective, allowing for the toppling slope to fail. The fracture surface of the slightly toppling masses was step-shaped, illustrating the slope’s retrogressive failure.

4. Stability analysis using UDEC

A numerical simulation is an effective tool for the quantitative analysis of slope stability. The dynamic deformation law of jointed rock masses can be simulated using DEM. UDEC software[18] was used for the slope at the entrance of the Tonghua tunnel to clarify the stability of the toppling deformation.

4.1 Model setup

A 2D model was constructed based on the geological profile 1–1’ in Fig. 3, in which eight history points monitored the slope’s displacement and strain (Figure 4). Three simulation scenarios were selected: natural, rainfall, and earthquake scenarios. For the first two scenarios, the slope’s left and right sides were fixed in a horizontal direction and its bottom side was fixed in a vertical direction. A viscous boundary condition was imposed on the bottom side of the earthquake analysis, whereas the free-field boundary conditions were specified in both lateral sides to minimize wave reflections. The rock and soil physical and mechanical parameters were obtained experimentally (Table 1). The materials adopted the Mohr-Coulomb strength properties. The seismic load in the simulation is simplified as a sinusoidal stress wave that can be transmitted upwards from the bottom of the model. The duration of the seismic action is 5 seconds. Considering that the frequency of the seismic wave is small, 3 Hz was used in this simulation, and the peak acceleration was 0.25 g.

![Image](image-url)

**Figure 4.** a) Geological material model of profile 1–1’, b) Mesh generation, c) Monitoring points

**Table 1.** Physical and mechanical parameters of rocks
Table 2. Physical and mechanical parameters of discontinuities

| Discontinuity | Normal stiffness (GPa) | Shear stiffness (GPa) | Internal friction angle(°) |
|---------------|-----------------------|----------------------|--------------------------|
| Bedded plane  | 50                    | 30                   | 25                       |
| Joint         | 50                    | 30                   | 23                       |

4.2 Results of simulation

4.2.1 Natural scenario

The slope’s maximum total displacement was 5.74 cm, which was located at the bottom and middle of area B. The X-displacement of area B was 2.0–5.0 cm, and the maximum displacement was observed at the bottom. The Y-displacement was 1.5–4.0 cm, and the maximum displacement was observed at the top. The X-displacement of the slightly toppling-deformed area was 1.0–3.0 cm, and the maximum displacement was observed at the bottom. The Y-displacement was 0.5–3.5 cm, and the maximum displacement was observed at the top. The deformed slope’s total displacement (Figure 5) revealed that the total displacement of area C was 1.72–4.59 cm under the natural scenario, whereas the total displacement of area B was 5.16–5.74 cm. The X-displacement of the monitored history points (Fig. 5d) shows that area B has the largest deformation (history points #1 and #3 in Fig. 4c). The X- and Y-displacements of monitoring point #1 are 3.0 cm and 4.0 cm, respectively. The X- and Y-displacements of monitoring point #3 are 4.6 cm and 2.3 cm, respectively. The history points #5 in area C has an X- and Y-displacement of 1.6 cm and 1.8 cm, respectively. The undeformed bedrock is stable, of which points #8 has an X- and Y-displacement of 0.3 cm and 0.3 cm, respectively.
4.2.2 Rainfall scenario

The slope’s maximum total displacement was 9.07 cm, which was located at the bottom of area B. The X-displacement of this area was 4.0–8.0 cm, and the maximum displacement was identified at the bottom. The Y-displacement was 2.0–6.0 cm, and the maximum displacement was identified at the top. The X-displacement of area C was 2.0–5.0 cm, with the maximum value located at the bottom. The Y-displacement was 1.0–5.0 cm, with the maximum value located at the top. The deformed slope’s total displacement (Figure 6) shows that the total displacement of area C is 2.72–7.26 cm under rainstorm conditions, with the displacement in the central section being the smallest, at only 2.72–4.54 cm; whereas the total displacement of area B is 8.17–9.07 cm.

The X-displacement of the monitored history points (Figure 6d) shows that area B exhibits the largest deformation (history points #1 and #3 in Figure 4c). The X- and Y-displacements of monitoring point #1 are 4.0 cm and 6.0 cm, respectively. The X- and Y-displacements of monitoring point #3 are 7.8 cm and 3.6 cm, respectively. The history points 5# in area C have an X- and Y-displacement of 2.2 cm and 2.4 cm, respectively. The undeformed bedrock is stable, of which points 8# has an X- and Y-displacement of 0.3 cm and 0.3 cm, respectively.
4.2.3 Earthquake scenario

Iterations consisted of 5 s time steps and the results were recorded at different steps (Figure 7). The maximum total displacement of the slope was 24.7 cm after a 1 s time step, and it was concentrated on the bottom of area B. After 2 s time steps, the displacement was observed at the bottom of each area. Total displacement increased from 24.7 to 45.1 cm. The maximum block displacement increased from 41.2 to 113 cm. The total displacement showed that area B may slide along its bottom interface under the 2.0 s sinusoidal excitation, and the dangerous rock mass on the slope surface has been separated from the parent rock masses. Further, the maximum displacement significantly changed after the 5 s time step. The displacement was concentrated at the bottom of each area. The maximum total and block displacement increased to about 1.44 m and 4.79 m. In addition, flexural toppling was temporarily observed at the top of the slope.
4.3 Analysis of the slope failure mechanism

Based on the simulation results, the deformation concentrated at the toe of the strongly toppling area, with a maximum value of 24.7 cm after 1 s time step under earthquake scenario, and the displacement decreased with an increase in depth. The maximum displacement and the deformation extended in the vertical direction after the 2 s time step were also located at the toe of the strongly toppling area. The maximum displacement significantly increased after the 5 s time step, with a value of 1.44 m. The displacement at the slope’s crown increased to 56.1 cm, indicating the occurrence of retrogressive failure. A comprehensive analysis of numerical simulation under earthquake conditions shows that the deformation of the deformed slope is primarily located in the strongly toppling-deformed area, whereas the failure of the slightly toppling-deformed area is relatively insignificant. The strongly toppling-deformed area is dominated by shear failure due to the slope toe’s stress concentration. Retrogressive deformation occurred due to the earthquake process. Thus, the slope’s failure mode can be summarized as follows: the earthquake caused the slope toe to collapse→the retrogressive deformation occurred→the slope surface collapsed and the sliding surface gradually penetrated→the deformed slope accelerated deformation→the deformed slope caused the overall failure.”.

5. Conclusion

In this study, we discussed the toppling-deformed slope at the Tonghua 1# Tunnel Entrance of Weng-Ma Expressway in southwestern China. The zonation, deformation behavior, and failure mechanism were investigated based on field investigations and numerical simulation. The main conclusions can be summarized as follows:

(1) The original bedrock occurrence of the slope is $355^\circ \angle 77^\circ$. The deformed slope with the inclination angle of the rock layer less than $17^\circ$ with fractured rock mass is classified as the falling area, according to the strength of the toppling deformation and its potential failure mode. The deformed slope with a rock inclination angle of $17^\circ$–$45^\circ$ is classified as a strongly toppling-deformed area, whereas the deformed slope with a rock inclination angle of $45^\circ$–$70^\circ$ is classified as a slightly toppling-deformed area.

(2) Using a numerical simulation method, the distribution of displacement indicators showed that the strongly toppling-deformed area exhibits the largest displacement at the bottom, with a total displacement of 5.74 cm in a natural scenario, 9.07 cm in rainfall scenario, and 1.44 m in earthquake scenario.

(3) Based on numerical simulation results and comprehensive analysis, the slope is unstable and damaged under earthquake conditions. The slope’s failure mode can be summarized as follows: the earthquake caused the slope toe to collapse→the retrogressive deformation occurred→the slope surface collapsed and the sliding surface gradually penetrated→the deformed slope accelerated deformation→the deformed slope caused the overall failure.”.
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References

[1] Hoek E, Bray JW (1981) Rock slope engineering. Institute of Mining and Metallurgy, London, UK
[2] Hungr O, Leroueil S, Picarelli L (2014) The Varnez classification of landslide types, an update. Landslides 11(2) 167-194
[3] Liu CH, Jaksa MB, Meyers AG (2008) Improved analytical solution for toppling stability analysis of rock slopes. International Journal of Rock Mechanics and Mining Sciences 45(8) 1361-1372
[4] Huang R (2015) Understanding the Mechanism of Large-Scale Landslides. Engineering Geology for Society and Territory-Volume 2, Springer. 13-32
[5] Zhang Z, Liu G, Wu S, et al. (2015a) Rock slope deformation mechanism in the Cihaxia Hydropower Station, Northwest China. Bulletin of Engineering Geology and the Environment 74(3) 943-958
[6] He K, Ma G, Hu X, Liu B (2021) Failure mechanism and stability analysis of a reactivated landslide occurrence in Yanyuan City, China. Landslides 18(3) 1097-1114
[7] Liu B, He K, Han M, Hu XW, Wu TW, Wu MY, Ma GT (2021) Dynamic process simulation of the Xiaogangjian rockslide occurred in shattered mountain based on 3DEC and DFN, Computers and Geotechnics 134 104122
[8] Goodman RE, Bray JW (1976) Toppling of rock slopes, in Rock Engineering for Foundations & Slopes, ASCE pp 201-234
[9] Amini M, Majdi A, Aydan O (2009) Stability analysis, the stabilisation of flexural toppling failure. Rock Mech Rock Eng 42 751–782
[10] Adhikary DP, Dyskin AV, Jewell RJ (1996) Numerical modelling of the flexural deformation of foliated rock slopes. Int J Rock Mech Min Sci 33(6) 595–606
[11] Nichol SL, Hungr O, Evans SG (2002) Large-scale brittle and ductile toppling of rock slopes. Can Geotech J 39(4) 773–788
[12] Liu B, Hu XW, He K, He SH, Shi HB, Liu DY (2020) The starting mechanism and movement process of the co-seismic rockslide: a case study of the Laoyingyan rockslide induced by the "5.12" Wenchuan earthquake. Journal of Mountain Science 17(5) 1188-1205
[13] Ning Y, Zhang G, Tang H, Shen W, Shen P (2019) Process analysis of toppling failure on antidip rock slopes under seismic load in southwest China. Rock Mechanics and Rock Engineering 52(11) 4439-4455
[14] Alejano Leandro R, Gomez-Marquez van, Martinez-Alegra Roberto (2010) Analysis of a complex toppling-circular slope failure. Eng Geol 114 93–104
[15] Pal S, Kaynia AM, Bhasin RK, Paul DK (2012) Earthquake stability analysis of rock slopes: a case study. Rock Mechanics and Rock Engineering 45(2) 205-215
[16] Zhang L, Pei X, Lin H, Li S (2015b) Evolution of landslide based on growth characteristics of trees on the landslide - a case study of Erguxi landslide. Mountain Research 33(4) 503-510 (In Chinese)
[17] Liu M, Liu FZ, Huang RQ, Pei XJ (2016) Deep-seated large-scale toppling failure in metamorphic rocks: a case study of the Erguxi slope in southwest china. Journal of Mountain Science 13(012) 2094-2110
[18] Itasca Consulting Group Inc. (2011) UDEC (Universal Distinct Element Code), Version 4.1. Minneapolis Itasca