Assessment of interaction between a waste storage dam and instability in downstream right-side slope by 3D numerical analyses

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Abstract
Waste dams (tailings) are used for storing the byproducts of mining operations and are very common in the mining industry. As the material they are storing can be harmful to the environment and the people in the vicinity, the construction of these dams is of high importance. During the construction phase, the topography of the site and geological–geotechnical parameters of the soils around the dam are as important as the stability and the leakage issues of the main dam body. In this paper, a nickel-ore waste dam located in Manisa-Gördes, Turkey is investigated in detail by 3-dimensional (3D) finite-element analyses. At the downstream side of the dam, there was slope instability at the right-hand side slopes due to improper loading on the top of these slopes. Whether this slope failure will affect the stability of the main dam body or not is the main question of this study. Within these confines, 3D finite-element analyses that cover a large area, including the dam, upstream and downstream sides, and the waste have been performed before and after the planned downstream slope rehabilitation. The main rehabilitation methodology employed in this study is the removal of the displaced material and re-shaping of right-hand side slopes. The results show that after the rehabilitation of the slopes, the deformations decreased considerably, and the waste dam became safer. Consequently, the study presents an interesting case study including the interaction between waste storage dam and side-slope instability with 3D finite-element analysis. At present, the planned slope excavations and the dam body construction were completed successfully.

Keywords Waste dam · Slope stability · 3D numerical analysis · Deformation · Flysch zone

Introduction
A waste dam (tailings dam) is generally an earth-fill embankment dam used to store byproducts of mining operations after separating the ore from its gangue. Tailings stored in these dams can be liquid, solid, or a slurry of fine particles, and are usually highly toxic and potentially radioactive which are hazardous to the environment. The main difference between these dams from any other earth-fill dam is that these are made permanently to store the waste from the ores. Currently, thousands of tailings dams worldwide contain billions of tons of waste material from mineral processing activity at mine sites (Rico et al. 2008a). The leakage of these dams can be very dangerous for the environment and the people in the vicinity. The leakage is prevented using geosynthetics, clay cores and checked continuously. However, the construction steps of these dams are very similar to earth-fill dams that are built for other purposes such as hydropower and irrigation.

As mentioned above, because the stored ore wastes can be very hazardous to the environment, the stability of a mine waste dam (tailings dam) is considered to be very important in recent years, especially after failures of some of them. A chronological list of failures of mine waste dams can be seen in https://www.wise-uranium.org/mdaf.html (reached on March 30th, 2021). According to this list, the rate of tailings dam failures is increasing: about 65% of the total failures in the last 60 years occurred between 1990 and 2021.
The failures are mainly due to heavy rain (Rico et al. 2008b; Azam and Li 2010), poor drainage conditions, poor detailing in the construction phases, overloading the dam with excessive amount of waste (higher than the designed level), static and seismic liquefaction (Ormann et al. 2013), etc. The dam failures not only cause casualties but also the downstream side of the dam is covered with waste mud, which is toxic and environmentally hazardous. These failures are a big threat in the cities, forests, lakes, etc. in the vicinity of the dam (Rico et al. 2008a, b).

Azam and Li (2010) have reviewed the tailing dam failures for the last 100 years. According to them, the most important cause of failure is unusual weather. Foundation failures were common in the past, but in recent years, failures due to the foundation have been reduced. Zandarin et al. (2009) has performed a detailed numerical analysis on a tailings dam from a nickel industry in Cuba and concluded that the guidelines and codes in practice are valid and the pore pressures are very much important in the stability of tailings and must be operated carefully.

In addition to the causes of failures above, the topography of the site is also important in the stability of the dams. The slopes in the upstream and downstream sides of the dam can create various problems for the dam’s safety. However, most of the studies in the literature (Ozcan et al. 2013; Cho and Song 2014; Das and Hedge 2020; Jin et al. 2020) deal with the stability of the dam body instead of the side slopes in the downstream and upstream. The main problem for the case in this study was the instability in the right slope at the downstream side of the main dam body. A slope failure has occurred on the side slopes and there was a probability of this failure to affect the main dam body. For this reason, the purpose of this study is to assess the effects of instability in the downstream right slopes of a tailings dam of the main dam body. The 3D numerical analyses were performed in that unstable area to understand if those instabilities affect the main dam body or not. These 3D numerical analyses were performed using Midas GTX NX 3D. The whole area was covered in the analyses including the dam body, upstream and downstream sides so that the effects of the instabilities can be estimated accurately.

The deformations in the dam body are calculated as a result of these numerical analyses. According to the FEMA2005 (Federal Guidelines for Dam Safety Earthquake Analysis and Design of Dams), the expected performance of the dam is judged according to the severity of the deformation, such as loss of freeboard, the potential of cracking leading to failure of the embankment or foundation. Similarly, the performance of the dam following a movement can be measured by considering the (i) the use of the reservoir and (ii) the ability or lack thereof to quickly repair a damaged structure. It suggests a limiting deformation of 2 ft (~0.6 m). Similarly, Cetin (2014) has summarized the allowable deformations of a dam body from various standards and says that 0 to 1.5 m permanent deformations are acceptable if the settlement of the dam is less than one-tenth of the dam height according to Hawaiian Dam Safety Guide and Division of Safety of Dams (DSOD) California, however, according to FHW-0-97-076, the acceptable deformation is 0.3 m.

According to U.S. Department of the Interior Bureau of Reclamation Design Standards No.13 Chapter 9 (Static Deformation Analyses), the magnitudes of horizontal deformations (into and down the valley) are relatively small compared to the vertical settlement and this depends on the geometry, material and dam zone properties. For the dams that are built properly, the settlements at the crest after the construction are generally ranging between 0.2 and 0.4 percent and rarely exceed 0.5 percent of the embankment height. Keeping this in mind, the “1 percent rule” is used to make a conservative design. According to Chapter 13 (Seismic Analyses and Design), a predicted deformation of less than 1 foot (~0.3 m) would not be a threat to the dam (unless a critical part of the dam is damaged). Similarly, it is also mentioned that, for any embankment dam, estimated deformations exceeding 3 feet would raise a concern about cracking and loss of freeboard.

General characteristics of the study area and dam

The site which is the subject of this paper is located 18 km from the city Manisa-Gördes of Turkey. It is located at 789 to 980 m altitudes where the topography is steep and rugged due to land sliding. The location of the site is shown in Fig. 1.

Geological and geotechnical characteristics of the area

The study area is located in the northwest part of Gordes Basin and the Bornova Flysche Zone (Fig. 2). Several Tethyan suture belts are included in the eastern Mediterranean region around Turkey (Sengör and Yilmaz 1981; Robertson 2004). The Neotethyan Izmir–Ankara Suture Belt (IASB) is one of these suture belts within these (Sengör and Yilmaz, 1981). The IASB in northwestern Anatolia locates between the Sakarya Composite Terrane in the north (Göncüoglu et al. 1996) and the Tauride–Anatolide Platform in the south (Fig. 2, inset map; Tekin and Göncüoglu 2007; Tekin et al. 2012). Stack of S-vergent nappes or tectonic slices of the intact or dismembered ophiolites, assemblages of the subduction–accretion prism with or without high pressure–low temperature metamorphism (ophiolitic mélanges),
olistostromes of the fold-and-thrust belt and finally the continental margin units of the Tauride–Anatolide Platform form the Izmir–Ankara Suture Belt (Goncuoglu et al. 1992; Okay et al. 1996; Goncuoglu 2000; Goncuoglu et al. 2000; Tekin and Gönçüoglu 2007; Tekin et al. 2012). The Bornova Flysch Zone described by Okay and Siyako (1993) is a part of the Izmir–Ankara Suture Belt including several imbricated tectonic units. Sandwiched between them are olistostromes that were formed in front of the ophiolitic nappes during their emplacement onto the Tauride–Anatolide Platform margin (Goncuoglu et al. 1996, 2003; Tekin and Gönçüoglu 2007). Main lithologies forming the Bornova Flysch Zone are pillowed and massive volcanic rocks, radiolarian cherts and mudstones, pelagic limestones, blueschists, gabbros, serpentinites, recrystallized limestones (Tekin and Gönçüoglu 2007; Tekin et al. 2012). The age of the Bornova Flysch Zone is reported by Kaya (1972) and Konuk (1977) as Campanian to Danian. The wastewater storage dam is located in the Bornova Flysch Zone (Fig. 2).

An extensive site and laboratory investigation scheme have been performed in the area before the beginning of construction. After some progress, when there was a slope instability problem at the right-side slope, additional site tests whose locations are shown in Fig. 3 were performed. As can be seen in Fig. 3, the number of additional boreholes is 15. The total length of the boreholes is 701 m. In these
boreholes, hydraulic pressure tests have been performed for every 2 m as well as permeability tests and geological monitoring. The groundwater level is measured to be variable in between 0 and 16.0 m. To determine the unit weight, uniaxial compressive strength, modulus of elasticity, and Poisson’s ratio of the soils and rocks, samples have also been collected at the site and tested. One of the cross-sections indicated in Fig. 3 is presented in Fig. 4. According to this soil profile which depends on six boreholes that coincides with this section, the soil profile is erratic at the surface for about 50 m. According to these analyses, the generalized lithological profile and their parameters summarized in the following section have been developed.

Two detailed engineering geological and slope stability studies were performed by Tuncay et al. (2018) and Topal and Nalçakan (2019). Tuncay et al. (2018) performed 2D static and dynamic limit equilibrium analyses for the dam body, and they found that there is not a slope stability problem for the dam body. Field observations were carried out by Topal and Nalçakan (2019), and then 7 geotechnical drillings, each 50 m deep, were performed in the landslide area, and inclinometers were installed in the completed boreholes.

Fig. 2 Simplified geological map of the Bornova Flysch Zone (after Tekin and Göncüoğlu 2007) (the inset map published by Göncüoğlu et al. (1996) portrays the distribution of Alpine terranes)
In 4 of the boreholes, pressure meter tests were performed to obtain the deformation modulus of the soils and rocks. 14 uniaxial and 42 point load tests on rocks and 2 triaxial tests on soil samples were performed. In addition to these, Atterberg limit tests, water content tests and sieve analyses have also been conducted. According to the data obtained from the drillings, mudstone, radiolarite, siltstone, sandstone, limestone, and diabase belonging to the Bornova flysch zone were detected in the field. However, old waste material and clays were observed in the upper levels in the landslide zone. The mechanism of the landslide was described by Topal and Nalçakan (2019) (Fig. 4). The safe slope stability conditions were obtained as shown in Fig. 5. In the present study, the existing geometry of the landslide, and the safe slope geometry proposed by Topal and Nalçakan (2019) are considered for investigation of the effect of landslide and the excavation of the displaced material.

### Features of the dam

Kalsnes et al. (2017) classified the tailings dams such as upstream method, downstream method, and centerline method. According to Kalsnes et al. (2017), recent waste dam failures involved dams constructed by the upstream method, where the new embankments are founded on tailings. The waste dam investigated in this study was constructed by the downstream method because the downstream method is relatively safer when compared to the others. The crest height of the first stage is planned to be at 851 m from sea level and the height of the dam was about 60 m at this stage. The second stage is planned to be 14 m higher than the first one (865 m) in about 4 years and the last stage is to be at 886 m in about 9 years. The final storage capacity of the dam is 25 mm³. The wastewater contains nickel and sulfate as pollutants. In this study, the effects of slope instability on the existing dam (the second stage) are investigated. At the end of construction (final case, the third stage) the dam height reaches up to 100 m. There were no problems after the construction of the first stage. However, after construction of the second stage, there was a slope stability problem at the downstream right-side slope (Fig. 3) which is due to overloading of the excavated material on the slope.

The waste dam is a fill dam that consists of rock- and earth-fills. The upstream part of the dam is filled with earth-fill material (1B) whereas the downstream part is rock-fill (1C) for the first stage. In Table 1, the minimum strength...
and deformability parameters of the materials to be used in construction are given. During the construction process, the earth (1B) and rock (1C) fill must provide these minimum parameters. The sand filters are used in the dam body and drain the water till the drainage material beneath the dam body. The upstream surface of the dam is covered with geosynthetics so that the waste cannot leak through the dam body. These geosynthetics are GCL (geosynthetic clay liners), and HDPE (geoshield high-density polyethylene). These are used together over the clay surface so that the leakage will be avoided, and the waste will settle faster. Mendes et al. (2010) revealed that GCL can be installed as a passive barrier, where it is associated with geomembranes that compose the active barrier. Using these together, in case of damage in HDPE, the GCL beneath it will serve as a barrier and limit the rate of flow. Similarly, Ozhan and Güler (2018) concluded that using a geomembrane-laminated GCL is the appropriate solution for a mine tailing dam. As geomembranes are thin and rigid materials in general, they can fracture easily under high hydraulic pressures and when they contact with angular soil particles (Koerner 2005; Rowe 2005; Lupo and Morrison 2007). This is why GCL and HDPE are used together in this tailings dam. The minimum factor of safety is selected as 1.3, 1.5, and 1.0 for static empty reservoir, static case when the reservoir is full and seismic cases, respectively.

The safe upstream slope is designed to be 1 V:2.9H for the earth-fill part (i.e., for the first 35 m part from the bottom). At every 10 m, there is a berm of 5 m. For the upper parts of the upstream where there is rock-fill, the slope is 1 V:2H. For the downstream part, a slope of 1 V:2.5H is found to be safe with a larger (6 m) berm at every 20 m. A typical cross-section of the dam, including the materials used can be seen in Fig. 6 at this stage.

### Numerical analyses

Hamade and Mitri (2013) performed a reliability-based approach to the geotechnical design of tailings dams while Wei et al. (2016) presented a case study on a geotechnical investigation of drainage methods for heightening a tailings dam based on 2-dimensional stability analyses. Sitharam and Hegde (2017) investigated stability of a rock-fill tailing

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**Table 1** Properties of the dam material

| Zone                                  | Unit weight, $\gamma$ (kN/m$^3$) | Internal friction angle, $\phi$ (°) | Cohesion, $c$ (kPa) | Modulus of elasticity (kPa) |
|---------------------------------------|-----------------------------------|------------------------------------|---------------------|-----------------------------|
| 1A—Compacted clay                     | 17                                | 25                                 | –                   | –                           |
| 1B—Selected earth fill                | 18                                | 26                                 | 20                  | $1 \times 10^5$             |
| 1C—Selected rock fill                 | 18                                | 32                                 | –                   | $2 \times 10^5$             |
| 2A—Sand filter                        | 19                                | 34                                 | –                   | $1.5 \times 10^5$           |
| 2B—Filter/drainage                    | 19                                | 34                                 | –                   | –                           |
| Colluvium                             | 18                                | 26                                 | 20                  | –                           |
| Colluvium shear zone foundation        | 18                                | 18.5                               | 0                   | –                           |
| Waste                                 | 23                                | 20                                 | 0                   | –                           |

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![Fig. 6](image)

Fig. 5 The safe slope geometry proposed by Topal and Nalçakan (2019) (Section 3 in Fig. 3)
dam employing 2-dimensional numerical analysis. However, 3-dimensional (3D) numerical analyses are extremely useful tools to predict deformations and to consider the interaction of two or more events and/or structures (Gokceoglu et al. 2016 and 2021; Komu et al. 2020; Aygar and Gokceoglu 2021). For this reason, in the present study, to understand the effects of the landslide and the excavations to obtain safe slope geometry, 3D numerical analyses are performed. Midas GTS NX 3D (MIDAS 2021) software has been used in the analyses. The analyses were performed to assess the effects of the instability at the slope in resting between the dam body and the unstable portion on the dam body. The calculation stages and the equation used by the Midas GTS NX 3D can be found in the user manual published by MIDAS (2021). The analyses consist of: (i) current state, and (ii) the safe slope geometry (after removing the sliding mass, making a 2H/1 V slope at the rock slopes). The positions of rock masses are shown on the GSI chart (Fig. 7) proposed by Marinos and Hoek (2000). GSI considers rock mass structure and discontinuity characteristics. These properties were obtained from borehole cores and outcrops. Each rock mass was described and the GSI values are obtained. The highest uniaxial stress used in the numerical analyses is 40 MPa for diabase. However, except for diabase, the uniaxial stress of the rocks has an average value of about 20 MPa. These parameters were obtained from the detailed site investigation performed at the site before and during the construction process. In addition, RocLab software was also used to determine the parameters related to rock mass. The generalized Hoek–Brown failure criterion proposed by Hoek and Brown (1997) and Hoek et al. (2002) has a non-linear failure envelope. This criterion is needed GSI, m parameter, disturbance factor, elasticity modulus and uniaxial compressive strength of intact rock. In the Hoek–Brown failure criterion, the relationship between the principal stresses at failure

Fig. 6 Typical cross-section of the dam

Fig. 7 The positions of the rock masses in the study area on GSI chart proposed by Marinos and Hoek (2000)
for a given rock is defined by two constants such as the uni-
axial compressive strength of intact rock and \( m_i \) constant
(Hoek and Brown 1997). The uniaxial compressive strength
values of intact rocks are obtained from the tests while the
\( m_i \) values are selected from the estimation table proposed
by Hoek and Brown (1997). Employing the Hoek–Brown
criterion, the shear strength parameters and deformation
modulus of rock masses are obtained. The generalized soil
and rock parameters used in the numerical analyses are pre-
sented in Table 2. Table 2 summarizes that the lithological
profile consists of soils with low cohesion and high internal
friction angles. An example for a non-linear Hoek–Brown
failure envelope is shown in Fig. 8 for siltstone–mudstone
alternation.

### Current state

The 3D model of the current state of the area is analyzed
using Midas GTS NX software as stated above. Figure 9
presents the generated model. In this model, the green part
is the downstream side of the dam. The unstable slope zone
is shown with a circle (Fig. 9). The different colors in this
model are not different lithologies but they were used for
constructing the geometry of the model. The static boundary
conditions were used in the model, i.e., fixed in all directions
at the bottom. The sides can move freely in z-direction but
fixed in x- and y-directions to describe boundary conditions.
In these analyses, the parameters presented in Table 2 have
been used in the corresponding zones and the maximum
waste level, which is +863 m, is modeled to be on the safe
side.

After performing the analyses, the obtained deformations
in the x-direction are presented in Fig. 10a. The maximum
deformation is calculated at about 54 cm in -x-direction in
the whole area. This is the point, where the slope instability
occurred. However, in the mean dam body, the maximum
deformation in the x-direction is calculated to be about
9.3 cm on the left-hand side. Although this 9.3 cm is accept-
able in design, the important aspect in this analysis is the
54 cm displacement in the downstream side slopes. Those
slopes should be remediated so that no slope failure occurs
in that area. Similarly, the deformations in the y-direction
are presented in Fig. 10b. As can be seen from this figure,
the maximum deformations in this direction are calculated
in between 3 and 5.75 cm. Deformations in the z-direction,
i.e., settlements, are presented in Fig. 10c. The maximum
deformations are expected in the z-direction and as expected
a value of 34 cm is calculated in this direction.

### Removing the sliding mass with 2H/1 V slope
at the rock slopes (safe slope geometry)

Many factors can affect the stability of a dam. These can
be (i) foundation failure, (ii) internal erosion and piping,
(iii) overtopping, (iv) seepage, (v) seismicity, and (vi) slope
instability, etc. (Clarkson and Williams 2021). However, for
the case of this study, the stability problem is not encoun-
tered in the dam body, but at the side slopes that are at the
downstream side. The main question answered in the study
was, if this downstream side slope fails, will it create a prob-
lem for the main dam body. The answer to this question is
complicated as, after the movement of that side slope, the
mass will exert extra pressure on the dam body which may
cause slope instability. Some methods such as deep piles or
ground improvement by any means can be offered to prevent
slope failure. However, these solutions are not applicable for
this case practically. The most convenient way to prevent this
movement is to re-design the slope without an extra support

### Table 2  Geotechnical parameters used in the analyses

| Lithological unit                  | Unit weight (kN/m³) | Cohesion (kPa) | Internal friction angle (°) | Uniaxial compressive strength (MPa) | GSI | \( m_i \) | Poisson’s ratio | Deformation modulus (MPa) |
|------------------------------------|---------------------|----------------|----------------------------|--------------------------------------|-----|-----------|-----------------|----------------------------|
| Clayey-sandy fill                  | 19/20               | 1              | 35                         |                                      |     |           |                 |                            |
| Siltstone–mudstone alternation     | 24/25               |                |                            | 21.04                                | 17  | 6         | 0.27            | 197.10                     |
| Siltstone                          | 27/28               |                |                            | 22.80                                | 20  | 7         | 0.25            | 364.46                     |
| Mudstone                           | 27/28               |                |                            | 19.28                                | 15  | 4         | 0.29            | 210.82                     |
| Slope wash                         | 19/20               | 5              | 35                         |                                      |     |           |                 |                            |
| Diabase                            | 25/26               |                |                            | 40.00                                | 35  | 10        | 0.19            | 1360.88                    |
| Serpentine                         | 23/24               |                |                            | 15.00                                | 20  | 4         | 0.23            | 205.52                     |
| Limestone                          | 25.7/26             |                |                            | 23.50                                | 34  | 9         | 0.28            | 996.29                     |
| Waste                              | 23                  |                |                            |                                      |     |           |                 |                            |
| Rockfill                           | 18/19               | 0              | 32                         |                                      |     |           |                 |                            |
| Selected soil fill                 | 18/19               | 20             | 26                         |                                      |     |           |                 |                            |
Analysis of Rock Strength using RocLab

Hooke-Brown Classification
- intact uniaxial comp. strength (σic) = 21.04 MPa
- σ3 = 17, m3 = 6, Disturbance factor (D) = 0.7
- intact modulus (Ei) = 7890 MPa
- modulus ratio (MR) = 370

Hooke-Brown Criterion
- mz = 0.003, s = 5.97e-6, a = 0.553

Mohr-Coulomb Fit
- cohesion = 0.206 MPa, friction angle = 8.27 deg

Rock Mass Parameters
- tensile strength = -0.002 MPa
- uniaxial compressive strength = 0.027 MPa
- global strength = 0.476 MPa
- deformation modulus = 197.10 MPa

Fig. 8. The non-linear failure envelope was obtained from RocLab for siltstone–mudstone alternation (Rock Science 2007)

Fig. 9. 3D model of the site (the circle contains unstable slopes)
system. In addition to this removal of this extra mass, drainage of water is mandatory for the stability of the side slopes and overall dam stability. In this study, the finite-element analyses were performed with the removal of the extra mass over the side slopes so that they cannot become harmful to the dam body.

For removing the sliding mass at the downstream side, a slope was designed with 2H/1 V slopes at the rock slopes as shown in Fig. 6. The area is now modeled with this geometry, using similar soil and rock properties (Fig. 11). Similar to the above, the waste was modeled to be at its maximum height for the operational stage. The yellow circle shows the location of the final slopes proposed. These slopes may seem very close to the boundary. However, the appropriate boundary conditions were selected and the effect of these boundaries in the results is tried to

**Fig. 10** Deformations in a x-direction, b y-direction, and c z-direction at the current state

**Fig. 11.** 3D model of the site with the proposed rock slopes (2H/1 V)
minimize. Because the field model restricts the normally semi-infinite analysis area to the dam body surroundings, the analysis boundary is defined at a position where there is nearly no change in stress or displacement (MIDAS 2021) due to the slope excavations and the existing dam body. In addition, as the effects of these slopes on the dam body are the main target of investigation of this study, and consequently, these boundaries do not have an impact at those points.

After the cut in the unstable slopes, the maximum deformations observed in x-direction reduce to about 17 cm which is shown in Fig. 12a. The deformations at the dam body are negligible after cutting these slopes. Similarly, when observing the deformations in the y-direction, it can be seen that the maximum deformation is about 9 cm in the slopes and 4.2 cm in the dam body which is shown in Fig. 12b. Figure 12c shows the settlements after excavation of the slopes. As can be expected, there is not much difference between the settlement values with the previous case, as the settlement of the dam body is mainly due to its weight.

To summarize the findings of the numerical analyses, the deformation values are presented in Tables 3 and 4 for the side slopes and the dam body, respectively. According to these tables, it can be seen that after the construction of the side slopes, the deformation reduces.

According to “Fill Dams Design Standards” a settlement of about 2–4‰ of the total dam height is acceptable. This allows a settlement of 20 to 40 cm for a dam of 100 m, which is the case in this study. Considering the excavation plan for providing stabilization of downstream side slopes is completed successfully (Fig. 13). During the excavations, the deformations on the dam body are observed continuously, and good accordance between the results obtained from 3D numerical analyses and in situ measurements are found.

| Stage                                    | Deformation in x-direction (cm) | Deformation in y-direction (cm) |
|------------------------------------------|---------------------------------|---------------------------------|
| Current stage, static (2nd phase)        | 54.45                           | 21.70                           |
| Current stage with side slopes at downstream, static (2nd phase) | 16.97                           | 9.15                            |

Fig. 12  Deformations in a x-direction, b y-direction, and c z-direction after excavation of the slopes
Conclusions

As stated by Dong et al. (2020), a tailings dam is an indispensable part of mining safety production and environmental sustainability. In addition, according to literature surveys performed by Shahriari and Aydin (2018) and Williams (2021), the main causes of tailing dam failure are poor geotechnical design, improper site, irresponsibility, seismicity, and lack of control. Considering these issues, in this study, a critical case is studied, and the results are discussed. Consequently, the conclusions obtained from the study can be drawn as follows:

In this study, 3D numerical analyses of a waste dam have been performed, covering all the surrounding areas. The aim was to see the effects of the unstable downstream side slopes on the dam body. The downstream right-side slopes became unstable after some of the excavated material has been loaded on that area. Although no problem was foreseen for the second stage of the dam, that unstable slope may have created a problem for the last stage as the toe of the downstream side of the dam reaches the toe of the unstable part. For this reason, the stability of the right-side slopes should be provided before the construction of the last stage.

For the analyses, the engineering parameters were obtained from the detailed site survey performed before and during the construction. As the safety of the dam is extremely important, extensive site investigations were performed. The analyses were performed in two stages such as current state and current state with safe slope geometry.

In the analyses of the current state, the deformations reached up to 55 cm in the sliding area. Hence, a slope design (with 2H/1 V slope) is proposed, and the slope-side which reduced these deformations to about 17 cm which is acceptable.

During the excavation of the proposed side slopes, there may be a water flow through the rock discontinuities as the groundwater level is near the ground surface. Therefore, it is strongly recommended to take the measures to drain water for long-term stability. Similarly, water resting in the impermeable rocks can create additional water pressure on the slopes which may result in slope failures. Hence, the drainage of the water must be assured including both the surface and ground waters.

In cases where two structures that affect each other or a failure are likely to affect another structure, 3-dimensional analyzes must be performed. The results of this study, which is carried out as an example of such a situation, express both the current state of a slope instability that may affect a dam and the deformations that will occur in the case of excavation of the displaced material. Consequently, it is recommended to use the methodology presented in this study to understand the safety of the waste dams, which are extremely critical structures, to determine the deformations that will occur beforehand, and to take the necessary engineering measures.

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Declarations

Conflict of interest As the authors, we declare that we have no conflict of interest with any person or institution.
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