Assessment of overburden dump and highwall slope stability for Jambad open cast coal mine, West Bengal, India, using *in situ* and laboratory testing

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In this study, *in situ* multichannel analysis of surface waves was performed to characterize the overburden (OB) layers for Jambad open cast coal mine, West Bengal, India. OB dump samples were also collected and laboratory tests were carried out to evaluate the compaction and strength characteristics. Stability analyses of the OB dump slope and highwall were carried out using the finite element-based software Optum G2 considering the configurations suggested by the Eastern Coalfield Limited, India. The stability was also assessed for seismic loading conditions considering pseudo-static loading. This study concludes with recommendations for geometric configurations of the OB dump and highwall slope.

Keywords: Coal mine overburden, dump slope, highwall, laboratory tests, surface waves.

INDIA is one of the largest producers of coal worldwide. The production of coal has increased manifold in the country in the last few decades. Surface coal mining has mainly contributed to this increased coal production. Presently, more than 88% of coal production is from surface mines[1]. Due to the significant increase in production, overburden (OB) and inter-burden material quantity has increased considerably. A significant increase in the share of the opencast output in the Indian coal industry has resulted in considerable amounts of waste dump with a greater dump height constructed over a minimum area and increasing the danger of dump failures. It is worth mentioning that coal mine OB particles are often heterogeneous, with sizes ranging from clay to boulder, and may contain cavities[2,3]. Considering the heterogeneity of the OB dump materials, it is hazardous to evaluate the stability of the OB dump slope without a detailed study of the materials. This is reflected by the increasing trend of dump slope failures reported in India in recent years[4]. Internal dumping is a cost-effective and environmentally beneficial waste disposal option. However, it has several drawbacks, including dangerous breakdowns and operational risks[5]. Verma *et al.*[5] conducted finite element numerical analyses of internal dumps in an opencast coal mine of Wardha Valley Coal Field, Maharashtra, India, to assess their stability with variation in the bench slopes height. The factor of safety (FOS) was shown to decrease drastically with an increase in the height of the dump slope. A critical evaluation of stability analysis and design of pit slopes in Indian opencast coal mines was performed by Satyanarayana and Sinha[6], focusing on the effects of geology, groundwater and slope angle on stability assessment and failure mechanisms. Also, different analytical and numerical methods such as limit equilibrium method, finite element method and finite difference method were used to study the factors affecting the pit slope stability[7]. Zou *et al.*[8] explained some techniques and approaches used for waste dump design analysis and waste dump optimization based on an open-pit mine waste dump in Tibet. It was reported that prolonged rainfall in the mining area caused dump failure and loss of valuable life and property. A dump failure in 2013 at Basundhara mines of Mahanadi Coalfields Limited, Odisha, India, claimed 14 lives[9]. Along similar lines, Behera *et al.*[9] conducted a case study on the stability analysis of dump materials of an opencast coal mine at Talcher coalfield, Odisha, and found that the increase in pore water pressure as a result of rainfall infiltration is a major cause of failure. Gupte[10] recently used a numerical technique for dump stability analysis in one of the coal mines of Western Coalfield Limited, India. The study results recommended a 22% increase in the existing dump capacity by satisfying the required safety factor criteria.

In addition to the dump slope, evaluating the highwall slope stability further poses significant concerns. Any failure or accident may lead to human casualties and damage to equipment/machines. Satyanarayana *et al.*[11] performed a stability evaluation of a 170 m highwall slope for the Medapalli open cast project (MOCP) of Singareni Collieries Company Ltd (SCCL) using FLAC/SLOPE. Their study focused on the effect of groundwater on the stability of the slope[11]. It was reported that slope stability

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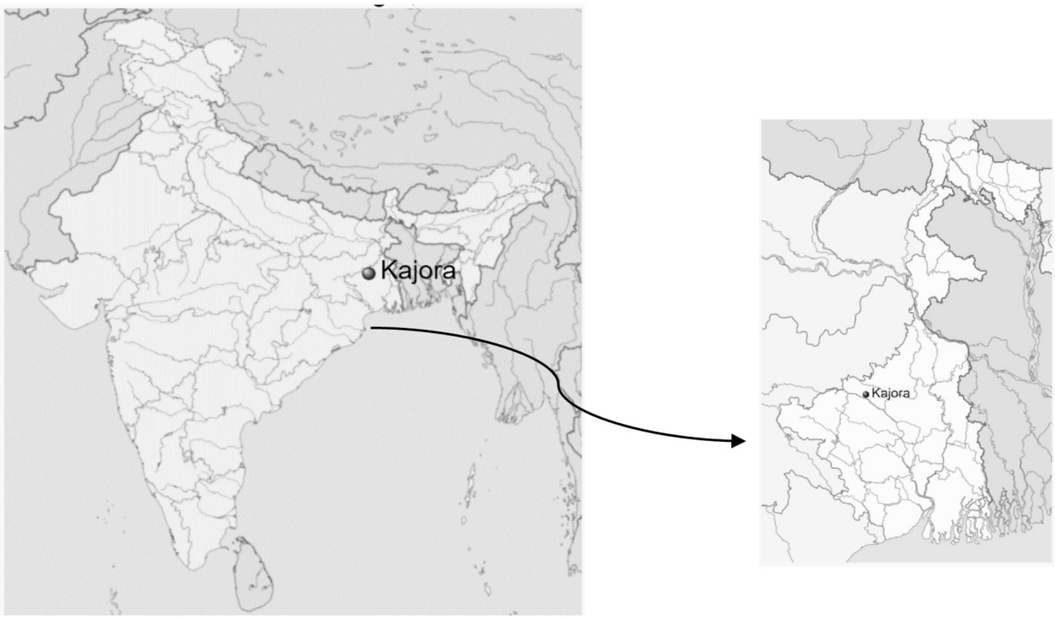

Figure 1. Map showing the location of the Kajora Area open cast coal mine, West Bengal, India.

is critical if it is undrained. Hence slope drainage methods were recommended. The numerical simulation of a deep highwall slope of Ramagundam open cast mine was done by Satyanarayana *et al.*[12], using FLAC/SLOPE to determine the impact of various parameters like density, angle of internal friction, cohesion, overall pit slope angle and depth on the stability of the highwall slope. All of the variables had a clear and significant association with FOS.

Faulting can cause slope failures when encountered in surface mining excavations. These slope failures can have significant cost implications for the mining project. Faces almost parallel to faults are more likely to fail, whereas those nearly perpendicular to faults are more likely to remain stable[13]. Hughes and Clarke[13] discuss the several forms of slope instabilities that might occur due to normal faulting considering some cases of North East England. Various empirical classification approaches are available to anticipate rock mass behaviour and/or slope performance. McQuillan *et al.*[14] have developed a new slope stability rating system for excavated coal mine slopes using a database of 119 intact and failed case studies gathered from open-cut coal mines in Australia. Based on this, visual observation of the excavated slope face was used to develop an empirical–statistical slope stability assessment approach for coal mine excavated slopes.

In the present study, dump and mine pit slope stability analysis was performed for Jambad open cast coal mine, Kajora Area, Eastern Coalfield Limited (ECL), located in the Burdwan district of West Bengal, India (Figure 1). The Kajora Area is located at 23.63°N 87.17°E. Considering the Directorate General of Mine Safety (DGMS, Indian Government Regulatory agency for safety in mines and oil fields) requirements for slope stability, ECL, India, committed the aforementioned scientific study to a team of geotechnical experts from the Indian Institute of Technology (Indian School of Mines, IIT (ISM)), Dhanbad. In this regard, finite element analysis was performed for dump and pit slope. The required material properties were taken based on laboratory, field study, literature and the relevant data provided by ECL. Further, the effect of saturation and earthquake on slope stability was also examined.

Material properties

Overburden dump

During the first visit to the site, three samples from different locations of the dump were collected for visual examination and laboratory testing. Due to the boulder particles present in the samples, it was not feasible to perform grain size analysis. However, by visual observation, the OB material was considered cohesionless, and further, tests like standard proctor compaction and direct shear test were performed. As particle size in the samples was greater than 4.75 mm, a compaction test was conducted in the mould of 1500 cm^3 capacity and application of 56 blows according to the energy corresponding to standard compaction. The compaction test results are presented in Figure 2 *a* and Table 1 respectively. They suggest that the first and second samples are similar in behaviour. Hence, their (samples 1 and 2) compaction characteristics, maximum dry unit weight and optimum moisture are similar. However, the compaction curve for sample 3 is below those corresponding to samples 1 and 2. This may be attributed due to the presence of significant fines in sample 3.

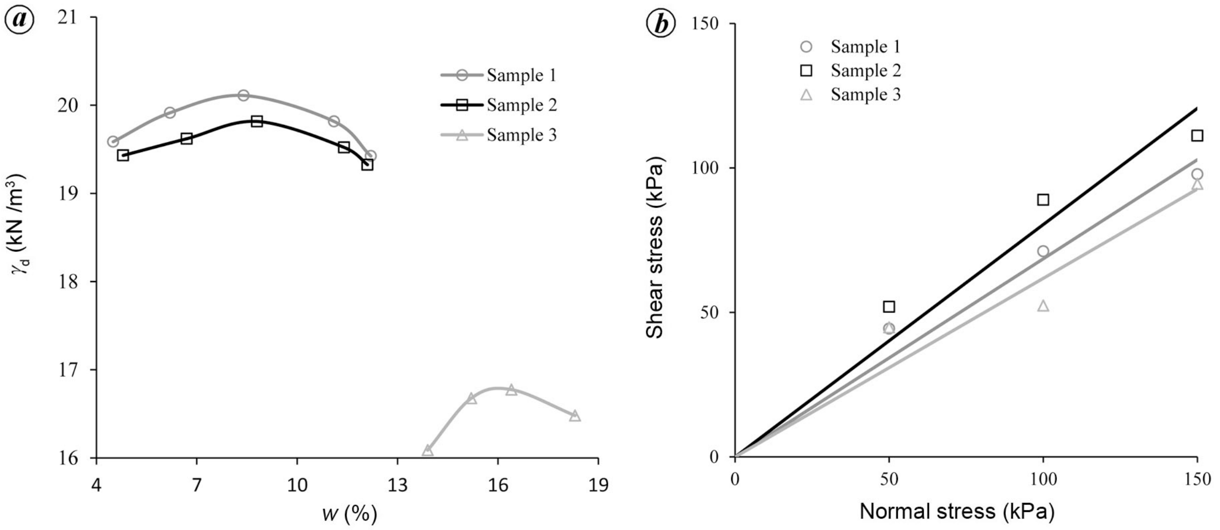

Figure 2. (***a***) Compaction and (***b***) shear strength characteristics of dump materials.

Table 1. Proctor compaction test results for the collected dump samples

Sample no.	Optimum moisture content (w_{opt}; %)	Maximum dry unit weight (γ_{dmax}; kN/m^3)	Maximum bulk unit weight (γ_{bmax}; kN/m^3)
1	8.4	20.11	21.8
2	8.8	19.82	21.56
3	16.4	16.78	19.53
Average	11.2	18.90	20.96

The maximum dry unit weight (γ_{dmax}) for samples 1, 2, and 3 was 20.11, 19.82 and 16.78 kN/m^3 respectively, whereas the optimum moisture content (w_{opt}) was 8.4%, 8.8% and 16.4% respectively. Table 1 shows that though γ_{dmax} values of the collected samples may vary, their associated bulk densities are comparable.

In line with the compaction test, a large direct shear test was performed on the compacted samples of dump material. This test was performed according to IS 2720 (Part-39/Section-1, 1977, reaffirmed-2007) provisions[15]. The large direct shear mould of 300 mm × 300 mm was filled with a given sample in three layers. The weight of a sample in a given layer corresponding to γ_{dmax} and w_{opt} was obtained by multiplying the bulk density with the volume of the layer. The sample poured in a given layer was compacted by tamping and levelled before preparing the second layer. After preparing the sample, fastening the loading cap and applying normal stress, the test was performed at a strain rate of 0.24 mm/min and normal stress of 50, 100 and 150 kPa. The peak shear stress corresponding to given normal stress was estimated from the shear stress versus shear strain variation (not reported here). Figure 2 *b* is a plot of this peak shear stress versus normal stress. From this plot, the friction angle for samples 1, 2 and 3 can be determined as 34.5°, 38.8° and 31.7° respectively. The average friction angle for dump material was determined to be 35°.

Following the laboratory test results, a geophysical survey on the site was conducted to evaluate their properties. In this regard, a multichannel analysis of surface waves (MASW) was conducted at two locations on the existing dump slope material. The test was conducted with a trigger distance of 1, 1.5 and 2 m at location-1, and 1 and 1.5 m at location-2. During data processing, the raw wave field was groomed to create a dispersion image of high resolution. Figure 3 *a* shows a typical analysed result of the MASW method. Table 2 shows the developed shear wave velocity profile up to a depth of exploration, i.e. up to 50 m. The velocity profile at a given location was found to vary with the change in trigger distance.

The velocity profile at both locations was comparable. The shear wave velocity in the dump material was found to vary from 148 to 650 m/s. Figure 3 *b* shows the variation of shear wave velocity with depth obtained from MASW at different locations. This plot represents the heterogeneous characteristics of the OB dump materials, as obtained from the MASW method. Considering the inconsistency in the observed value of shear wave velocity at a given depth, the following average linear variation up to 50 m depth was considered for further analyses.

$$V_s = 5.9 * z + 203.2, \quad (1)$$

where z is the depth (m) and V_s is the shear wave velocity (m/s).

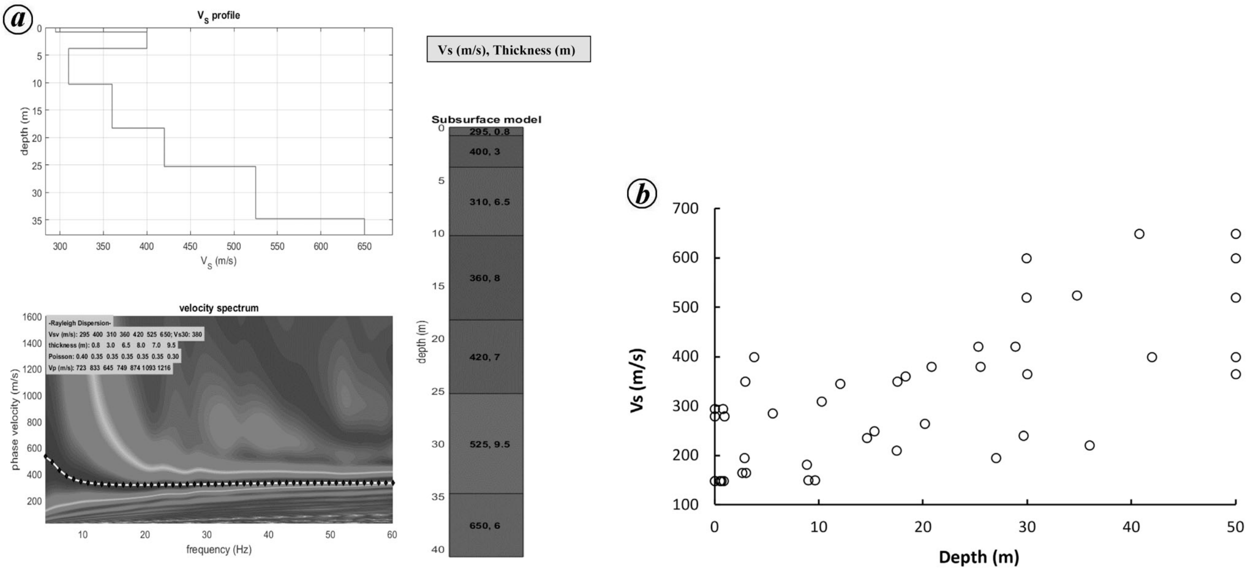

Figure 3. ***a***, Multichannel analysis of surface waves (MASW) for location-2 of Jambad opencast project (OCP), Kajora Area (trigger distance = 1.0 m). ***b***, Variation of shear wave velocity with depth.

Table 2. Variation of shear wave velocity with depth

Location-1						Location-2			
Trigger distance = 1 m		Trigger distance = 1.5 m		Trigger distance = 2 m		Trigger distance = 1 m		Trigger distance = 1.5 m	
Depth (m)	V_s (m/s)	Depth (m)	V_s (m/s)	Depth (m)	V_s (m/s)	Depth (m)	V_s (m/s)	Depth (m)	V_s (m/s)
0	148	0	280	0	125	0	295	0	148
0.5	148	0.95	280	0.65	125	0.8	295	0.85	148
3	165	2.95	350	2.65	165	3.8	400	2.85	195
9	150	5.55	285	9.65	150	10.3	310	8.85	182
17.5	210	12.05	345	14.65	235	18.3	360	15.35	250
27	195	17.55	350	20.15	265	25.3	420	20.85	380
36	220	25.55	380	29.65	240	34.8	525	28.85	420
42	400	29.95	520	30	365	40.8	650	29.95	600
50	400	50	520	50	365	50	650	50	600

Pit slope

Though it was possible to collect the sample for dump material, direct access to the pit was restricted due to legal bindings. Hence soil stratification for the pit slope could not be established. Further, no samples were provided by the authority to estimate the mine pit properties. However, based on previous lithological data, it was found that the pit slope was composed mainly of sandstone; the parameters of the same were obtained from the appropriate literature[5,11,12].

Domain and mesh details

In the present study, finite element analysis was performed using the Optum G2 software[16]. The usefulness of this software for a few specific geotechnical stability problems has been documented by Khatri *et al.*[17,18]. Typical features of this software include inputting particular variations of soil properties in the domain and adaptive meshes to improve solution accuracy. The chosen domain for the analysis of dump and highwall slope is described below.

Dump slope

Figure 4 *a* shows the trial domain selected for the analysis of dump slope; according to the requirements of ECL, the dump height of 50 m is divided into two benches. Further, it was required that the bench slope angle (α_d) should not be greater than 35° and the bench width should be ≥10 m. With these considerations, a bench width of 15 m was taken for analysis while the bench slope angle was optimized to satisfy safety norms laid down by DGMS, which has also established guidelines for scientific studies under the Coal Mines Regulations 2017. It indicates that the minimum safety factor for a pit and dump slope design should be higher than 1.5 for permanent slopes and 1.3 for others. Following these guidelines, considering the extent of detailed numerical analysis performed and the importance of slope along with the consequence of failure, a

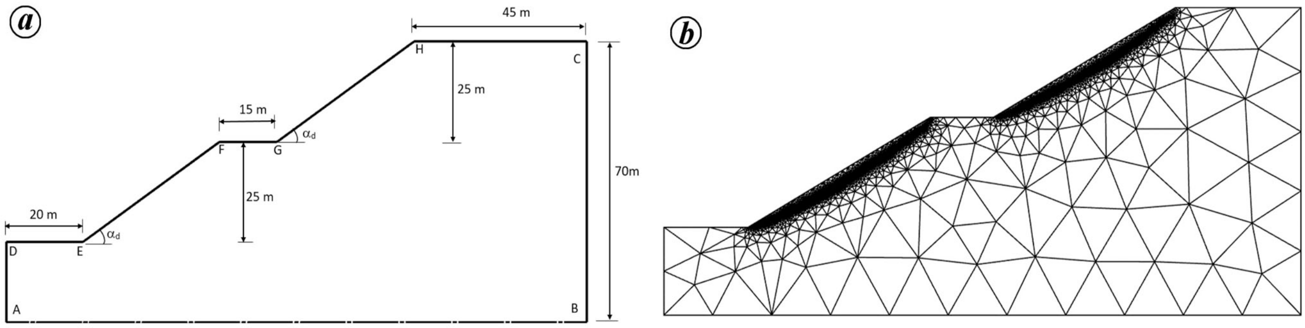

Figure 4. Details of dump slope: (***a***) soil domain and (***b***) mesh details for the analysis of dump slope.

limiting FOS of 1.3 was taken for the dump slope/bench slope and 1.5 for the highwall slope in dry/moist condition. It was further ensured that the safety factor for either submerged or seismic conditions should remain greater than 1. The foundation soil details were not available; hence additional 20 m of soil below the dump (line AD in Figure 4 *a*) was considered. Consistent with the finite element analysis requirement, the lateral displacement along lines BC and AD of the domain was assumed to be zero.

In contrast, the lateral and vertical displacement along the bottom boundary AB was substituted zero. The selected domain was meshed using six noded triangular elements. Figure 4 *b* shows the developed mesh for the dump slope. In the analysis, about 10,000 elements were considered to provide an accurate safety factor for a slope. A close observation of this figure suggests that the mesh becomes finer in the regions of shear failure. This task is automatically performed in Optum G2 while computing the solution.

Highwall slope

Figure 5 *a* presents a typical layer profile of the highwall. Figure 5 *b* shows the selected domain for the highwall, consistent with the ECL requirement. Figure 5 *c* reveals that the total height of the mine slope/highwall slope is about 106 m, including a coal seam of 14 m. The top 6 m of the slope was covered with soil. Further, it was required that the overall slope angle (corresponding to the dotted line UT; Figure 5 *b*) should not be more than 60° with respect to horizontal. Additionally, the width of each bench should be greater than 3 m. With these considerations, a bench width of 5 m and a bench angle (α_m) of 50° was provided. The total height of 106 m was divided into ten benches, each of 10.6 m, following the constraints imposed by ECL in the scope of the study. Note that the overall slope angle for the selected geometry is 38.37°. It may be mentioned here that the mine is quite old, about 20–30 years, and is nearing closure. Detailed documents/mine plans were not well maintained and not accessible to the present authors. However, visual inspection indicated that the foundation material mostly consisted of sandstone. This was also confirmed by the ECL authority. Hence based on the past literature, the required material properties were adopted in the present study. Similar to the dump slope, the chosen domain has meshed with six noded triangular elements with about 10,000 elements. Figure 5 *c* shows the developed mesh for the mine slope.

Methodology

As mentioned before, finite element analysis was performed in the present study using Optum G2. The soil was assumed to obey Mohr–Coulomb's failure criteria together with an associated flow rule. The strength-reduction analysis was invoked to analyse dump and mine slope, wherein the soil properties are reduced until failure occurs.

For the dump slope analysis, based on a laboratory study, simulations were performed using bench angles (α_d) of 30° and 35°. The study was conducted corresponding to the material properties of samples 1, 2 and 3 respectively. In this regard, the friction angle (ϕ) and bulk unit weight (γ_{bulk}) of the given sample were considered. Additionally, the analysis was also performed by taking the average properties of all the samples. Further, a study also considered the worst-case scenario involving an excess of material-3 by assigning weights of 4, 1 and 1 to the material properties of samples 3, 1 and 2 respectively to arrive at the weighted average properties. These analyses used bulk unit weight instead of dry unit weight to offer a lower safety factor. It is pertinent to note that the density and friction angle achieved in the field are indeterminate. Further, details of *in situ* moisture content and haul load magnitude were unavailable in the present study. Hence FOS was underestimated intentionally considering these uncertainties.

A tentative slope configuration that satisfies the DGMS guidelines was selected based on the above analysis results. This chosen slope geometry was further analysed based on the available field data, i.e. averaged shear wave velocity profile with depth. This was converted into standard penetration number (N) and friction angle profiles following

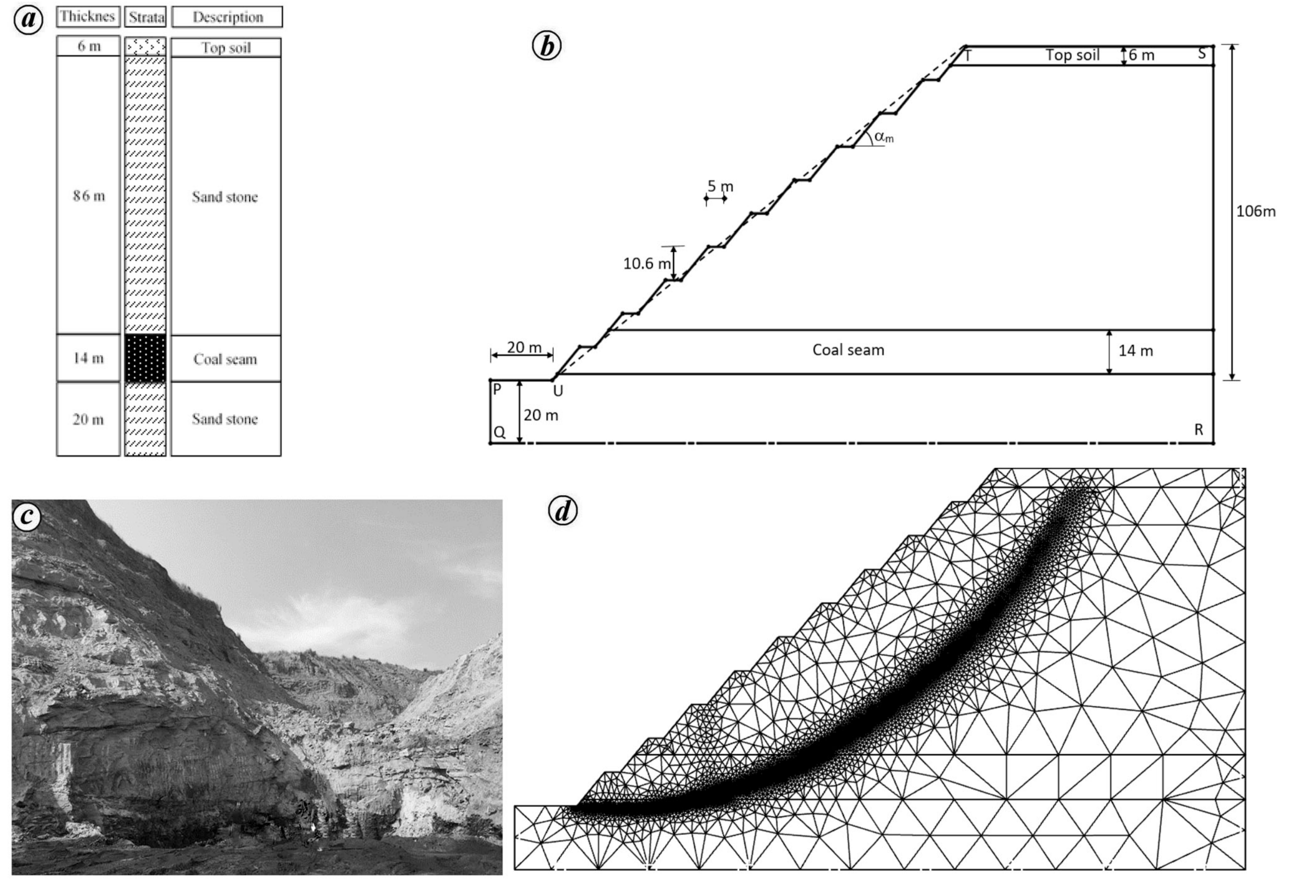

Figure 5. Details of highwall: (***a***) typical layer profile, (***b***) soil domain, (***c***) photograph of a highwall slope and (***d***) mesh details for the analysis of highwall slope.

Kumar *et al.*[19], who provided the relationship between shear wave velocity and *N* value for loose and dense sand and given as

$$V_s = 130 + 7.5N \text{ (loose sand)}, \quad (2)$$

$$V_s = 60 + 7N \text{ (dense sand)}. \quad (3)$$

In the present study, eq. (2) corresponding to loose sand was employed. Using the *N* value, the friction angle at a given depth was calculated as[11]

$$\phi = 27.12 + 0.2857N, \text{ for } N = 4\text{–}50. \quad (4)$$

After obtaining the friction angle profile, it was required to determine the bulk density at different depths. In this regard, a plot was drawn between friction angle and bulk unit weight based on the laboratory samples. In this respect, a linear trend line fitting provided the relationship between bulk unit weight and friction angle as $\gamma_{\text{bulk}} = 0.5965*\phi$. In this manner, the friction angle and bulk unit weight variation was developed. Below this depth, the soil properties were considered to be constant. Table 3 depicts the generated material profile. This profile was imported into Optum G2 for analysis of the chosen slope. To compare the results of this analysis with probabilistic analysis, the mean friction angle and unit weight were determined from Table 3 as 36.41° and 21.72 kN/m^3 respectively. The coefficient of variation for these parameters was estimated as 11% after statistical analysis of the data presented in Table 3. While inputting these properties in Optum G2, a log–normal distribution with a correlation length of 2 m was taken in both the horizontal and vertical directions respectively (based on the guidelines provided[20] in the literature)[16,20].

Following the laboratory and field study results, an overall slope configuration was selected. The effect of groundwater and earthquake was evaluated on this chosen slope. The analysis was performed only using the averaged bulk unit weight and shear strength determined in the laboratory. It is important to note here that ECL did not provide groundwater information. As a result, an approximate analysis was performed using the completely saturated soil mass to determine its effect on slope stability. In this regard, taking the average dry unit weight of 18.90 kN/m^3 and moisture content of 20%, the saturated unit weight for analysis was estimated as 22.68 kN/m^3 ≈ 23 kN/m^3.

After the saturated case analysis, computations were performed to evaluate the earthquake effect on the selected slope. In this regard, the pseudo-static analysis was performed wherein the equivalent body forces due to seismic forces were considered. This method computes the minimum safety factor by including the static horizontal and vertical forces representing the inertial effects of seismic vibrations due to an earthquake. These equivalent static forces are usually expressed as a product of horizontal or vertical seismic coefficients and the potential weight of the sliding mass. The horizontal equivalent-static force decreases the safety factor by reducing the resisting force and increasing the driving force. The vertical equivalent-static force typically has less influence on FOS. As a result, it is often ignored.

Since the actual slopes are not rigid and peak acceleration exists only for a short time, the coefficients used in practice generally correspond to acceleration values well below the peak ground acceleration (PGA; a_{max})[21]. Terzaghi[22] originally suggested using $k_h = 0.1$ for 'severe' earthquakes and $k_h = 0.2$ for 'violent, destructive' earthquakes. Marcuson[23] indicated that appropriate pseudo-static coefficients should correspond to one-third to one-half of the maximum acceleration, including amplification or de-amplification effects.

In the absence of site-specific estimates of design PGA, the design seismic inertia forces for equivalent-static slope stability assessment are taken as[24].

$$F_H = \frac{1}{3} * Z * I * S * W, \tag{5}$$

where F_H is the horizontal inertial force, Z the zone factor according to IS:1893-Part 1 (ref. 25), I the importance factor, S the empirical coefficient to account for the amplification of ground motion between bedrock and elevation of the toe of the slope and W is the weight of the sliding mass.

Following the IS:1893-Part 1 (ref. 25) stipulations and location of Kajora area in Zone-III, value of a $Z = 0.16$ was assigned. Further considering the slope necessary, an essential factor of 1.5 was selected. The empirical coefficient $S = 1$ was taken considering hard rock, soft rock and hard soil. With these considerations, the lateral earthquake coefficient $k_h = 1/3 * Z * I * S$ was determined as 0.08.

Note that if the estimate of design PGA at the elevation of the toe of the slope is available, the design seismic inertia forces for equivalent-static slope stability assessment may be taken as

$$F_H = \frac{1}{3} * a_{max} * W, \tag{6}$$

where a_{max} is the design PGA at the elevation of the toe of the slope. Considering a_{max} to be equal to the zone factor (=0.16 g), the coefficient is 0.053. However, an equivalent horizontal seismic coefficient of 0.08 was adopted in the present study considering the uncertainty over various considerations/assumptions and conservative design.

In the case of mine slope or highwall slope, access was not provided for geophysical studies. Further, no samples were supplied for laboratory testing. Hence the properties of topsoil, sandstone and coal seam (as indicated in Figure 5) were taken from Satyanarayana *et al.*[11,12]. The topsoil, sandstone and coal seam unit weights were 18.6, 23.2 and 15 kN/m^3 respectively. Further, the cohesion and friction angle of these materials were taken as 22 kPa and 15°, 165 kPa and 28°, and 260 kPa and 25° respectively. It is worth mentioning that in the present study, no joints/faults in OB strata or highwall were observed. The layers were mostly horizontal and the same was also confirmed by the ECL authority. Therefore, in this study, the influence of fault was not effectively considered.

Analysis for the dry case was performed by assigning these material properties in the Optum G2. Since groundwater was not observed within the slope, an examination of the same was not performed. Further similar to dump slope, the seismic condition analysis was performed by taking $k_h = 0.08$. The results of the finite element study on the dump and mine slope are described below.

Results and discussion

As mentioned earlier, in the dump case, slope stability analysis was performed based on the laboratory and field material properties. In contrast, the study was conducted for highwall slopes based on material properties reported in the literature. The results of various simulations are described here.

Table 3. Variation of material properties with depth based on multichannel analysis of surface waves test

Depth (m)	Shear wave velocity (V_s; m/s)	N	ϕ°	γ_{bulk} (kN/m^3)
0	203.16	10.45	30.11	17.96
2.5	217.85	12.55	30.71	18.32
5	232.66	14.67	31.31	18.68
7.5	247.59	16.80	31.92	19.04
10	262.63	18.95	32.53	19.41
12.5	277.80	21.11	33.15	19.78
15	293.08	23.30	33.78	20.15
17.5	308.48	25.50	34.40	20.52
20	324.00	27.71	35.04	20.90
22.5	339.64	29.95	35.68	21.28
25	355.40	32.20	36.32	21.66
27.5	371.28	34.47	36.97	22.05
30	387.28	36.75	37.62	22.44
32.5	403.39	39.06	38.28	22.83
35	419.62	41.37	38.94	23.23
37.5	435.98	43.71	39.61	23.63
40	452.45	46.06	40.28	24.03
42.5	469.04	48.43	40.96	24.43
45	485.75	50.82	41.64	24.84
47.5	502.57	53.22	42.33	25.25
50	519.52	55.65	43.02	25.66
70	519.52	55.65	43.02	25.66

N, Standard penetration number; ϕ, Soil friction angle; γ_{bulk}, Bulk unit weight.

Dump slope

Considering the requirement of ECL, stability computations were performed with a bench slope (α_d) of 35° and 30° by taking the material properties corresponding to samples 1, 2 and 3, average and a weighted average of the three samples. Table 4 indicates the variation of FOS. From Table 4, it can be observed that the bench slope of 35° is feasible only if the dump consists of a material with properties corresponding to sample 2. Otherwise, there is the possibility of slope failure. The bench slope of 30° is deemed safe considering individual, averaged properties. Using weighted averaged properties gives FOS = 1.22, which is less than the limiting value (1.3). Figure 6 shows a typical failure pattern after the analysis corresponding to averaged properties. As desired, a failure of individual bench slope was noticed, which further fulfils the purpose of bench provision. Note that bulk unit weights were considered rather than dry unit weights in the analysis to arrive at the lower values of the safety factor. Based on the above study results, a trial configuration with a bench slope of 30° was selected.

Furthermore, to ascertain the confidence in the selected slope configuration, the stability calculations based on field study were conducted. In this regard, a profile of material properties, shear strength and unit weight were generated based on MASW data. Figure 7 indicates the generated material profile. Using this figure, friction angle in the vertical direction varied from 30.1° (top) to 43.01°, while the unit weight ranged between 18.4 and 26.3 kN/m^3. Here again, bulk unit weight was considered for analysis. The simulations were performed for a bench slope of 30° considering this material profile. In this case, the safety factor was assessed as 1.217, similar to the value obtained using weighted average properties.

Table 4. Factor of safety variation for dump slope based on laboratory samples

	Factor of safety	
Sample no.	α_d = 35°	α_d = 30°
1	1.054	1.265
2	1.231	1.480
3	0.884	1.153
Average	1.001	1.298
Weighted average	1.02	1.223

α_d, Soil domain.

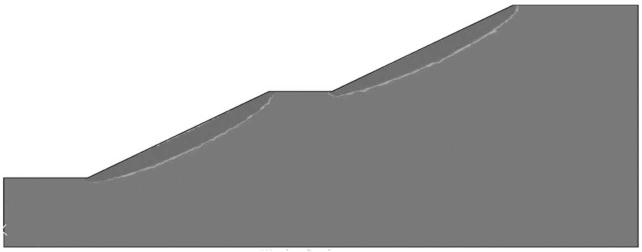

Figure 6. Typical failure pattern obtained from dump slope analysis based on a laboratory study (α_d = 30°).

In the above analysis, variation of material properties in the horizontal direction was neglected. Hence, a lower FOS was anticipated. Hence, an additional probabilistic analysis was performed by taking a mean friction angle and unit weight of 36.41° and 21.72 kN/m^3 respectively. The coefficient of variation for both was assumed at 11%, while correlation length in the vertical and horizontal directions was taken at 2 m each. About 1000 Monte–Carlo simulations were performed to evaluate the slope stability. Figure 8 depicts the variation of unit weight and friction angle in the domain after 1000 cycles, whereas Figure 9 displays the interpretation of a safety factor over 1000 cycles. Figure 9 reveals that the safety factor for the chosen slope ranges from 1.16 to 1.4, with a mean of 1.298 and a standard deviation of 3.75%.

Due to the use of averaged shear wave velocity profile with depth, on which the dump properties are dependent, such a low standard deviation in FOS value is justified. Notably, the mean safety factor obtained from the probabilistic analysis is close to that based on averaged properties reported from the laboratory tests. This implies that the selected averaged properties for the study are appropriate.

Further, the mean safety factor from this analysis is greater than that based on the material properties profile mentioned above. This is justified since, in the material profile-based analysis, the material properties increase in the vertically downward direction. In contrast, in the probabilistic analysis, the material properties vary in horizontal and vertical directions. As shown in Figure 10, the failure pattern produced in this study after 1000 cycles is similar to that obtained from the laboratory tests. The analysis to evaluate the effect of groundwater based on the use of the saturated unit weight of 23 kN/m^3 and averaged friction angle of 35° provided a safety factor of 1.290. Further, seismic analysis for this slope taking unit weight of 20.96 kN/m^3, friction angle of 35° and a horizontal acceleration coefficient of 0.08 resulted in a safety factor of 1.09.

The simulations described above suggest that the selected bench slope of 30° will be safe for dry, saturated and seismic conditions. The use of bulk unit weight instead of dry unit weight will provide additional safety.

Mine slope

The analysis for mine slope configuration, consistent with ECL requirements, was performed by selecting the properties from the literature. The FOS with an individual bench slope (α_m) of 50° and an overall slope of 38.37° was 1.495 for the static case and 1.316 for the seismic case. FOS for the static case was found to be greater than 1.4 and following the guidelines of DGMS for permanent

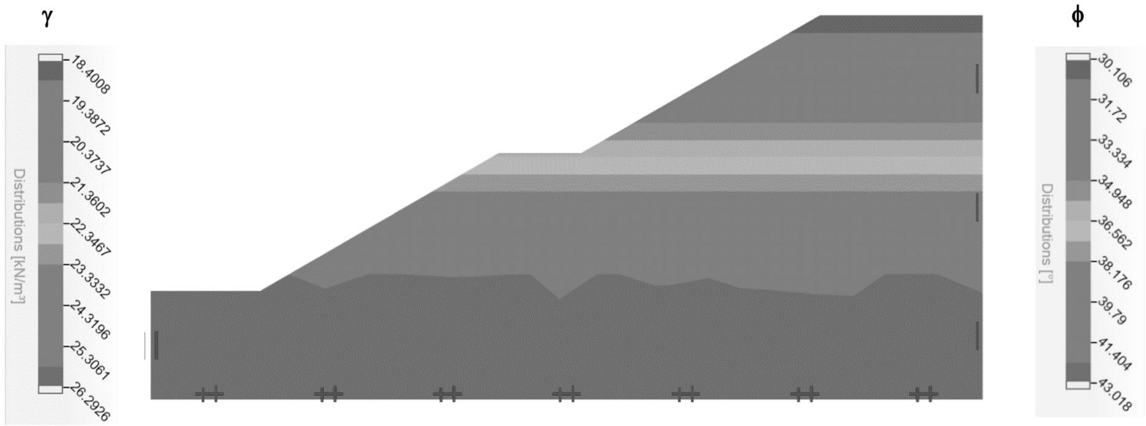

Figure 7. Variation of friction angle and unit weight in the soil domain based on shear wave velocity profile.

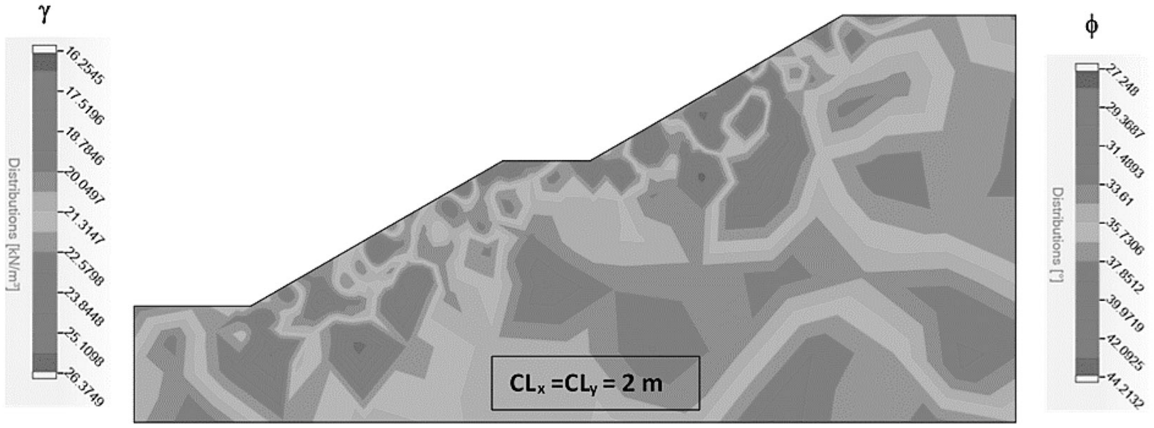

Figure 8. Variation of unit weight and friction angle in the domain for probabilistic analysis.

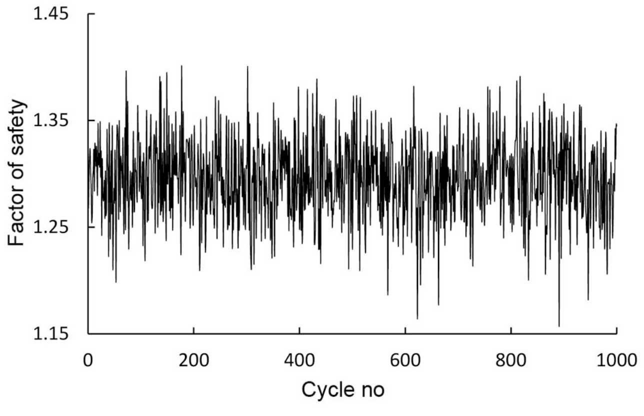

Figure 9. Variation of factor of safety over 1000 cycles.

slope. No attempt was made to optimize the bench slope angle further, considering the importance of the highwall slope and material properties from the literature. Figure 11 shows the failure pattern obtained from this analysis. The figure depicts the global failure of the slope as opposed to the failure of individual bench slopes (as observed in the analysis of the dump slope). A fault location within the slope was unknown; hence the plane failure or wedge failure analysis could not be performed.

Conclusion

The dump and mine slope stability was evaluated in the present study by performing a two-dimensional finite element analysis. This analysis was based on limited laboratory and extensive field tests. The outcome of this study brings forth the following conclusions: (i) For a given site, a dump with a bench slope of 30° was found to satisfy the stability criteria. (ii) For static conditions, the FOS values for the dump based on the limited laboratory and rigorous field tests were comparable. (iii) The dump slope can also be considered stable for a completely saturated case. At the same time, it is just stable for a seismic case. (iv) The selected mine slope configuration was stable for a static and seismic case. However, since the *in situ* material properties were not determined, their FOS values should be viewed with caution.

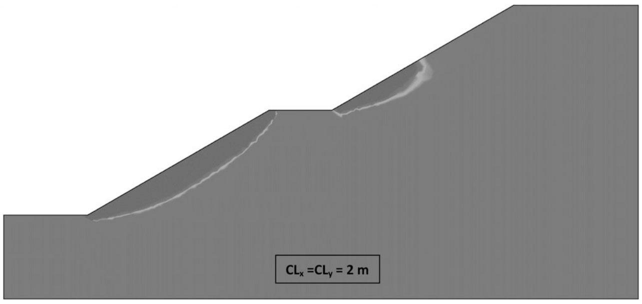

Figure 10. Failure pattern obtained at the end of 1000 cycles for dump slope.

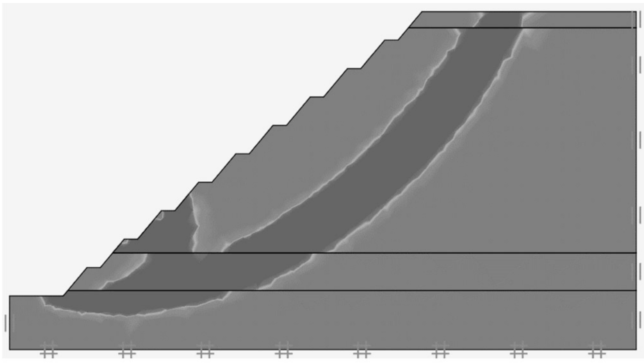

Figure 11. Failure pattern obtained at the end of 1000 cycles for highwall slope.

The dump and highwall slope constructed using the suggested configuration at the given site were observed to be stable, justifying the use of the selected material parameters and analysis. Further, the study report submitted to the ECL and DGMS has been duly accepted.

Conflict of interest: None.

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