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ORIGINAL PAPER

Analysis and Design of an Underground Portal in Lateritic Soils

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Abstract This paper presents a case study of the analysis and design of slopes for the portal of an underground crude oil storage cavern site. The site selected for the slope study is characterized by residual soils and granitic rock formations, located in the southwestern part of India. It is observed that in tropical residual soils, most hillslope failures are caused by rainfall and thus it is important to consider hydrological conditions when attempting to analyze the stability of slopes in such material. Combinations of shallow slopes with lower overburden and high steep hillslope with large overburden were considered in the present study along with varying combinations of lateritic soils and weathered rock formations. The paper discusses the various investigations carried out to define the geotechnical properties of lateritic soil and weathered rock, followed by numerical modelling and remedial measures adopted to ensure the stability of slopes during design and construction phase. Since analysis and design procedures for such residual soils are not well established, comprehensive geological and geotechnical investigations were carried out prior to numerical model development for carrying out finite element studies in order to ascertain long term stability of slopes under differing ground conditions. The results of the stability analysis indicated that slope under existing condition were potentially unstable under rainy conditions and specific supporting measures were planned to ensure stability. Several alternatives were examined for improving the stability of slope taking into consideration existing facilities, space available for mobilization of equipments and environmental conditions in reference to specific project requirements. The convergence pattern obtained from geotechnical monitoring using optical targets along the slopes did not showed any alarming movement for over a year.

Keywords Portal · Laterites · Slopes · Stability · Residual soil

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1 Introduction

Slopes stabilization associated with the portal of a crude oil storage project are of utmost importance considering specific purposes of these storages providing economic security to a country. Underground crude oil storages are built on the principle of hydraulic confinement, where crude oil is stored inside large unlined rock cavities which is confined from outside through water pressure which is made to flow towards the cavities at a pressure greater than the inside pressure. Portal of an underground storage project is the main access point to any underground scheme both during the construction and operation phases as shown in Fig. 1. In the present study the portal for a crude oil storage facility consist of two parallel tunnels; main access tunnel and water curtain access tunnel starting from main portal entrance as shown in Fig. 1. These tunnels serve as the main access for construction of main storage galleries situated deep inside for the storage of crude oil. During the operation of this facility all these tunnels are completely filled with water which helps in maintaining required pressure for the confinement of crude oil in the storage galleries beneath.

The construction of tunnel portals often involves solving problems that are closely connected with the morphology of the slope to be excavated, the existence of nearby constructions, the geometry of the structures to be constructed and the type of material involved. The construction of a stable portal site and of the surrounding slope, very frequently requires substantial cuttings which can be very problematic when working in residual soils like lateritic soil under heavy rainy conditions.

Tropical residual soils have unique characteristics related to their composition and the environment under which they develop. The soil type in the tropics is generally called laterite, or, residual soil. This soil was first described by Buchanan (1807) as "reddish in color, vesicular and unstratified in structure; a mantle of ferruginous (red to brown color) rock covering large areas in southern India. In natural state the material is soft enough to be cut into blocks with iron instruments but could rapidly harden on exposure to air to be fairly resistant to the weathering effect of climate". Most classical concepts related to soil properties and soil behavior have been developed for soils in temperate regions, and there has been difficulty in accurately modeling procedures and conditions to which residual soils are subjected (Fredlund and Rahardjo 1985, 1993).

The portal zone in most of the underground projects is frequently overlooked and often turns out be difficult area in terms of ground control. Failure of rock/weathered rock or soil regularly occurs in high angle approach cuts and initial subsurface portion of the portal. These problems are further aggravated by the weathered, anisotropic and stress relieved nature of the rock mass near the surface. It is often observed that if decomposition, caused by excavation in ground with little cohesion is not adequately confined, there is a risk that it will easily and rapidly propagate into the medium with serious effects on the whole slope. It is also noted that temperate region soil classification and testing methods often fail in predicting field performance of laterite or lateritic soils. This is because index tests upon which the classifications are based are not always reproducible for lateritic soils (Tuncer and Lohnes 1977). Lohnes and Demirel (1973) also pointed out that engineering classification system used for temperate soils regions tends to underestimate the strength of lateritic soils.

It is well known that force of gravity is the primary force causing stability problems in an unconfined loose soil state, which gets further destabilized by

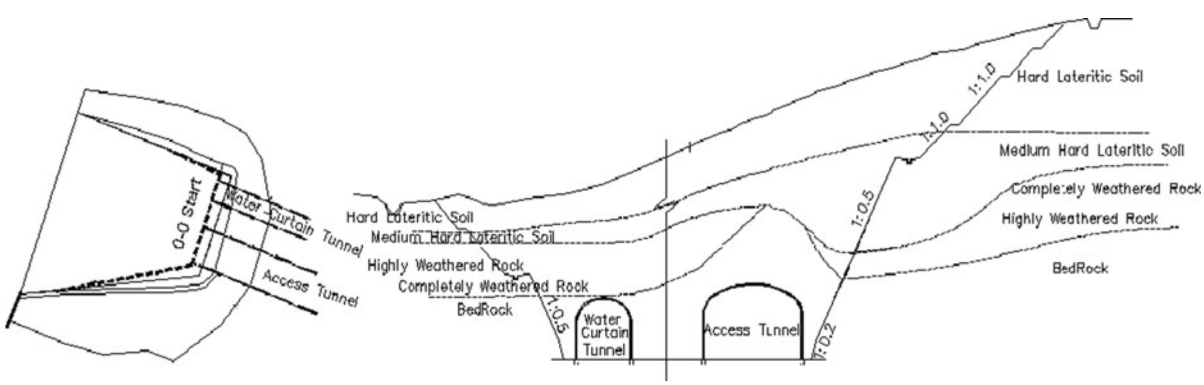

Fig. 1 Plan and section of portal along with access tunnel and water curtain tunnel

factors like water, erosion, earthquakes, surcharge loads, overburden pressure in residual soils. Under such conditions, the need for realistic engineering design and effective slope stabilization in residual soils subjected to large variations of water fluctuations is vital. Various studies conducted by Tohari and Sarah (2006), Shou (2000), Gogo-Abite (2005) and Gasmo et al. (2000) have reported significant decrease in factor of safety of slopes in residual soils under saturated conditions as compared to dry conditions.

In the present study, the site considered is characterized by presence of residual soils with an average rainfall of above 4,000 mm. The primary objective of this study was to investigate the effect of variability in strata of lateritic soils and weathered rock formations on slope stability of portal structure for the worst rainfall conditions. The paper also provides a realistic design of slopes in such conditions while highlighting role of proper investigations in stability determination and lays down construction guidelines necessary to fulfill during actual execution of works for ensuring long term stability requirements.

2 Site Investigations and Soil Characterization

Past field and laboratory studies have shown that residual soils consist of different zones of weathering with differing morphological, physical, and geotechnical characteristics; which also vary for different locations due to the heterogeneous nature and highly variable degree of weathering (Adekoya 1987; Rahardjo et al. 2004). Indraratna and Nutalaya (1991) and Adeymi and Wahab (2008) highlighted variable nature of lateritic soils and emphasized the need of detailed investigations for reliable estimation of engineering behaviour of such soils. The proposed storage site for this study is located under a hill which has a maximum elevation of 80 m above mean sea level. The investigated area consists of hills gently sloping down into two valleys, where elevation goes down from 80 m above mean sea level to about 25 m on other side. The major rock type found in the area is granitic-gneiss with cap of laterite of variable thickness. Plan and cross section of the portal location along the starting chainage of 0–0 m is shown in Fig. 1. Slope height is observed to vary from a minimum 10 m on left side to up to 30 m maximum on the right hand side of access tunnel location as shown in Fig. 2. The figure also shows tentative positions of proposed access (AT) and water curtain tunnels (WCT) along with completely weathered rock (CW) and laterite soil formations.

A two stage investigation at portal location was conducted to minimize the uncertainties of the geological model. The first phase of the investigation was part of the overall investigations carried out for the storage site and were mainly confined to investigations using core recovery and destructive hole drilling at the portal location to ascertain soil, weathered rock and bedrock cover thicknesses along with their interfaces. The second phase of the investigation was specific to slope area and determined shear strength properties of the laterites and weathered rock. This kind of two step compulsory site investigation program adopted during portal design, helped in identifying uncertainties related to characteristics of laterite as well as geomechanical characteristics of the weathered rock and bedrock beneath laterite.

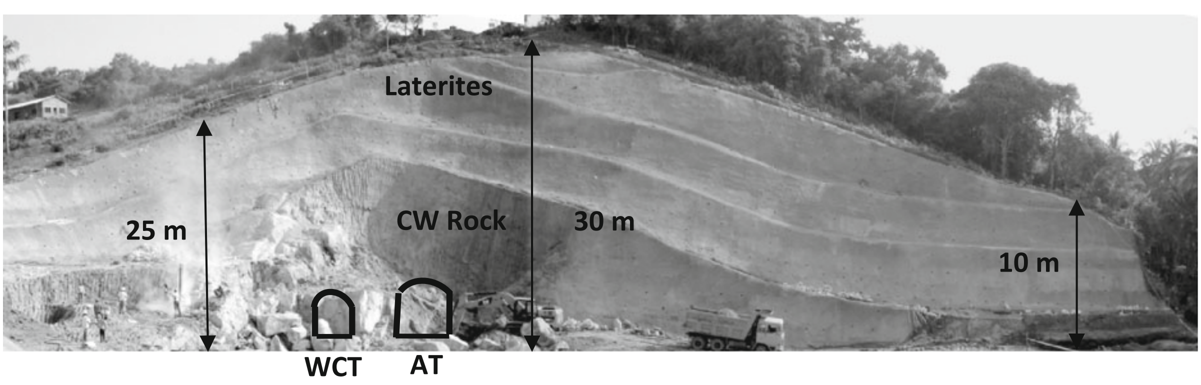

Fig. 2 Pictorial view of the portal area during construction. *AT* access tunnel, *WCT,* water curtain tunnel, *CW* completely weathered

As part of the first phase of investigation, two cored boreholes and eight destructive boreholes were drilled for the portal region. One cored borehole was located at the water curtain tunnel portal location, and another cored hole was located at the main access tunnel portal. Destructive holes for each portal were located at 25 m interval from each cored bore hole along the access tunnel direction. Destructive holes were drilled up to 2 m inside the fresh rock and geological descriptions were recorded by logging the extruded samples. The coreholes examination revealed that bedrock is mainly composed of granite gneiss which is covered with top soil that consists of anthropogenic soil cover, laterite, residual soil and weathered rock with thickness varying from 5 to 20 m. The average RQD value for the bedrock was found to vary from 71 to 87%. Standard penetration test were also carried out in soil part of the cored holes to evaluate their penetration resistance. Standard penetration resistance of upper soil layer (laterite) was found to be of N value of $\geq$34; corresponding very stiff to very hard categories while for weathered rock test could not be completed since number of blows was recorded to be greater than 50 for 10 cm penetration. However boundary between rock and soil interface (1–2 m section) was inferred to be consisting of weaker zones of silty clayey material having highly permeable layers. The laboratory test carried out on soil samples taken from core boxes indicated specific gravity in the range of 2.13–2.83, water contents 3.4–41.0%, liquid limit varying from 13 to 55% and plastic limit of average 33%. The result of direct shear tests carried out on disturbed lateritic soil and weathered rock sample indicated cohesion in the range of 10–20 kPa and 25–30 kPa respectively and internal friction angle varying from 30° to 36° and 35° respectively. This kind of variation in shear strength properties for laterites were earlier also reported by Tuncer and Lohnes (1977). Physical rock properties were also evaluated from core samples, indicated density in the range of 25.4–30.2 kN/m^3, specific gravity of 2.64–3.14 and porosity of 0.71–8%. Uniaxial compressive strength of the rock samples was found to vary between 40 and 150 MPa and elasticity modulus between 4 and 50 GPa.

Second phase of the investigation started at the same time as the preliminary construction activities at site. As part of the planning, clearing of bushes and vegetation along with open blasting at the portal area was carried out. Excavation of loose soil and blasting of loose rock was specifically carried out along the slopes, in order to examine the ground morphology more closely. It was observed during visual inspection along the slopes that laterites are hard and porous at the surface, which is followed by soft silty clay layer, weathered rock in the form of residual soil (completely weathered) and hard granite gneisses. However the thickness of hard lateritic soil and medium hard laterite (soft silty clay layer) was found to vary considerably all along the portal region from some 2–8 m. It was also noticed that thickness of medium hard laterite and completely weathered rock is comparatively higher (10–15 m) along the longest section of slope (H = 30 m as shown in Fig. 2) at the portal location. Therefore in order to provide reliable shear strength parameters for the hard and medium hard laterite and completely weathered rock, systematic investigations were carried out which was mainly aimed at testing undisturbed samples of soil and weathered rock from actual slopes location as per "IS 2132 1972: Thin Walled Tube Sampling". Three samples from each of the top hard laterite, medium hard laterite and completely weathered rock were collected from the portal slope by pushing open drive samplers of 38 mm diameter and 15 cm length into the exposed cut. The samples collected were packed and covered with polythene bags to avoid drying. Samples were extruded from thin walled tube sampler for unconfined compression tests and triaxial testing. Final shear strength parameters worked out on the basis of site investigation results are summarized in Table 1. It was observed from the results that in the upper part of hard laterites, cohesion was higher as compared to medium hard laterite, while friction angles values for weathered rock were comparatively lower, particularly in completely weathered rock region resembling more to combination of coarse grained soil and loose rock particles. The weathered rock avlaibale at some part of portal slope section was further distinguished as completely weathered (CW) rock resembling to a residual soil, and a highly weathered (HW) part consisting of dyke zone, which is moderately weathered consisting of highly jointed rock fragments. The moderately weathered rock in this slope section was very thin and the majority of weathered rock was observed to be completely weathered. Thus results of the second phase of investigation helped to define the presence of weak

Table 1 Shear strength parameters of soil and weathered rock

Site investigation	Cohesion (kPa)				Angle of internal friction (°)			
	Lateritic soil		Weathered rock		Lateritic soil		Weathered rock	
	Hard	Medium hard	CW	HW	Hard	Medium hard	CW	HW
Values used in analysis/design	68	42	40	20	30	30	31	40

CW completely weathered, *HW* highly weathered

soil layers (soft silty clay) and completely weathered rock layers which were modeled as separate layers in finite element analysis for determining an optimum slope design.

3 Analysis and Design of Portal Slope

In general, slope stability assessment investigates sensitivity of input parameters (strength, slope geometry and water etc.) based on field and laboratory observations. In the present study tunnel entrance area is located along the slope surface, where the depth of top lateritic soil is shallow and surrounding rock quality is poor. Tunnel portal is located in steep cut and has shallow cover where rocks are mostly completely weathered and highly jointed. In addition, it is likely that portal stability will be further affected by the surface drainage water and working loads further reducing the stability during construction. There are also chances that collapse or subsidence of earth surface due to lack of soil bearing capacity can occur since ground loosening may start due to blasting and excavation of portal area. Under such conditions, portal area needs to be selected and designed in consideration of actual ground conditions, climate, surrounding environment, constructability and maintenance convenience. In order to design an optimal and stable portal structure, different models of slope embankments were analyzed using varying combinations of thicknesses of laterites and weathered rock along different sections of the slopes. Slope stability analysis was mainly carried out for the slope at entry of access and water curtain tunnels along with sections representing right and left hand side of tunnels. This study was carried out using SLIDE software (RocScience 2008a), an interactive limit equilibrium slope stability program that uses the method of slices to calculate a factor of safety. In the present study Bishop simplified method with a circular slip surface was used for slope stability analysis along with Mohr–Coulomb failure parameters. Bishop simplified method is easy to use and gives relatively accurate results and agrees favorably (within about 5%) with the factor of safety calculated using finite element procedures (Albataineh 2006). SLIDE analysis in general involves a critical surface search, in order to attempt to find a slip surface with an overall minimum factor of safety to analyze the stability of soil slope.

Three different types of environmental conditions (dry, rainy and earthquake) were considered for evaluating stability of portal slopes under all possible combinations of soil and weathered rock formations. The present modeling exercise is intended to create an understanding of the effect of shear strength parameters on slope geometry, while assessing stability of slopes in worst condition of rainy season, considering water table along the surface of slope. Various base models selected for this study consider variable thicknesses of laterites and weathered rock layer are shown in Figs. 3 and 4. The models were created from the results obtained from final investigation results along with sections referring to centre of water curtain and access tunnels and sections representing maximum slope height along with different soil, weathered rock combinations. In these models thicknesses of hard laterite and medium was varied from 2 to 10 m, weathered rock 1–6 m and bedrock from 10 to 15 m.

Slope design factor of safety values provided by different organizations like the Korean Expressway Corporation (2002), US Corps of Engineers (2003) criterion and National Coal Board, UK (1970) were reviewed along with studies carried out locally on lateritic soils. Considering the international practice followed and information provided by Sreekanatiah (1993) on local behaviour of laterites, a factor of safety of greater than 1.5, 1.3 and 1.1 were adopted for dry season, rainy season and seismic loading respectively. Cut slope stability analyses were carried out by selecting areas with highest height or poor geological

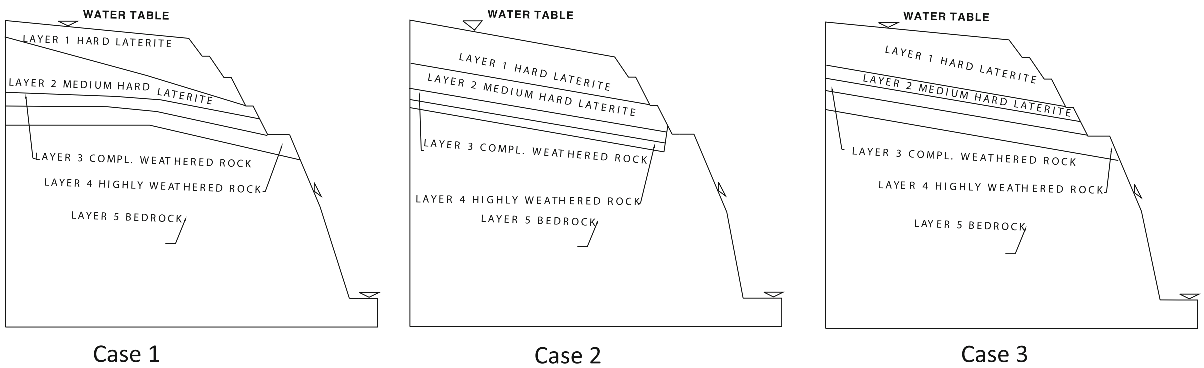

Fig. 3 Different models of portal slope adopted for numerical study: Part I

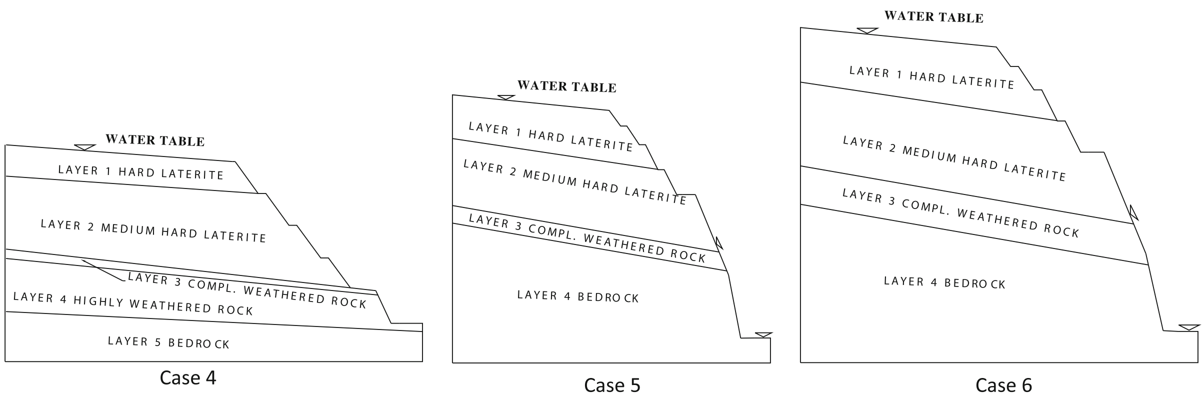

Fig. 4 Different models of portal slope adopted for numerical study: Part II

conditions as a main cross-section. As per the adopted design criterion in this study, a soil slope of 1:1, weathered rock slope of 1:0.5 and bedrock slope of 1:0.2 was selected in the present modeling for numerical evaluation of slope stability. It was further planned from a stability point of view to install 1 m wide berm every 5 m interval in the soil region and 3 m wide berm at the boundary between rock and soil or below the interface. However if required during construction, berm width can be adjusted as necessary to obtain workspace for equipment entry and underground tunnel requirements.

The results of the analysis in terms of factor of safety for different slope combinations are shown in Table 2. It is observed from Table 2, that slope stability is ensured in terms of factor of safety for all conditions except for saturated conditions along the larger sections, i.e. on right hand side of water curtain tunnel and access tunnel. As observed from Figs. 3 and 4, these cases represent locations where height of the slope is maximum along with comparatively thick zones of completely weathered (CW) rock and medium hard laterite consisting of silty clay. It was also observed during the analysis that the critical failure surface cuts across the laterite surface to finally intersect completely weathered rock layer as shown in Fig. 5a for Case 6.

Based on the results of numerical studies, it was inferred that slope stability under saturated conditions cannot be guaranteed for the available ground conditions and further strengthening in terms specific reinforcements needs to be determined. Since the right hand side of access tunnel (Case 6) was observed to be most endangered section due to its height and thick zone of completely weathered rock, it was planned to carry out reinforcement trials on this side of slope section. The reinforcement in the form of passive rock anchors fully grouted along its full length providing 100% bond length and extending beyond the slip circle surface was adopted as the primary reinforcement for increasing the slope stability. Passive rock anchors, 25 mm diameter untensioned bars

Table 2 Different cases analyzed for numerical study

Case no.	Location	Dry (FS ≥ 1.5)	Saturated (FS ≥ 1.2)	Seismic (FS ≥ 1.1)	Remarks
1	Rear part of AT at starting point	3.26	2.41	2.89	Stable
2	Rear part of WCT at starting point	2.84	1.89	2.47	Stable
3	Left side slope of tunnel at starting point	4.10	2.38	3.15	Stable
4	Right side slope of WCT at starting point	1.72	0.91	1.54	Unstable in saturated conditions
5	Right side slope of AT, 10 m outside	1.89	1.09	1.67	Unstable in saturated conditions
6	Right side slope of AT at starting point	1.76	0.99	1.57	Unstable in saturated conditions

AT access tunnel, *WCT* water curtain tunnel

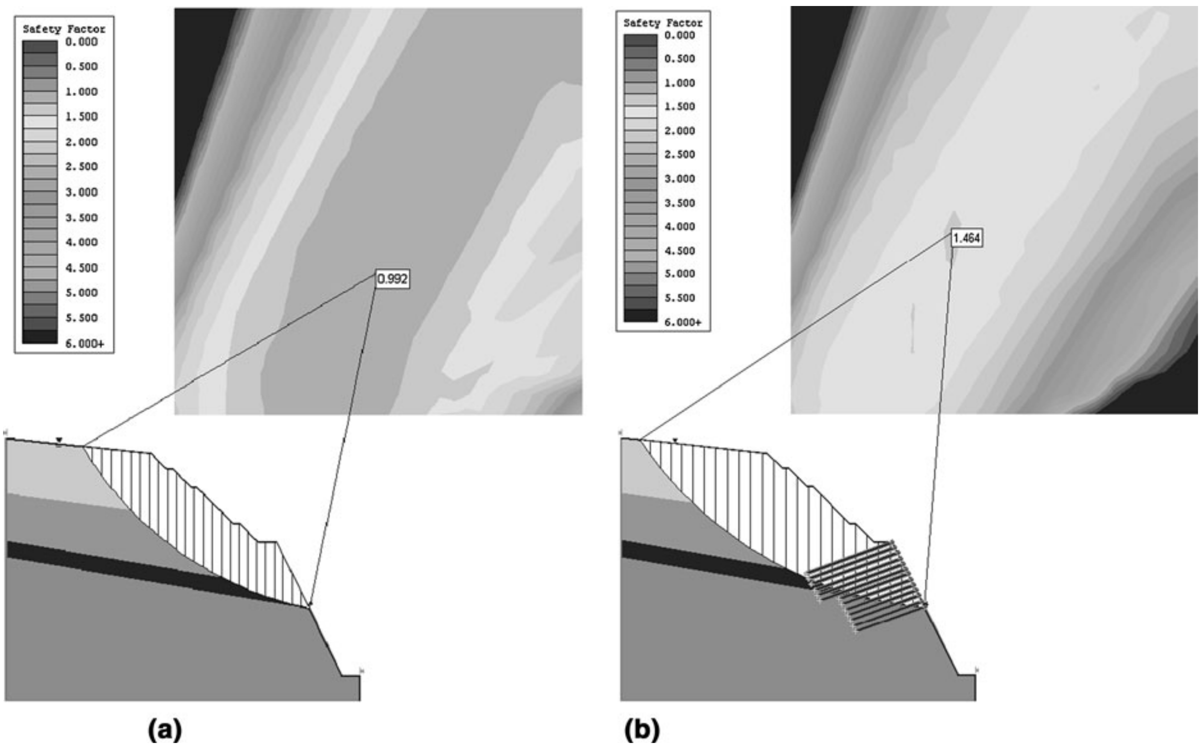

Fig. 5 Critical slip surface obtained in limit equilibrium analysis **a** with and **b** without reinforcement for Case 6

were planned to be installed at an inclination for a length crossing slip surface zone and covering whole of the completely weathered rock region. Rock anchors which only develop resisting force after some movement within the slope has taken place are considered as passive support elements in numerical models and they normally do not incorporate initial loading or tensioning. In a rock anchor support, the orientation of the applied force is always parallel to the orientation of the rock surface; thereby rock anchors were placed at an inclination of approximately perpendicular to slope surface. In order to decide on the optimum pattern of rock anchors position along slopes, three different arrangements of anchor reinforcement were reviewed, initially for Case 6, as it represents the most critical section. These combinations were analyzed only for saturated conditions. In the first pattern, passive rock anchors were installed along the slope right from the top of the hard lateritic soil to the bottom highly weathered rock for a length extending beyond the critical slip surface. Rock anchors were placed nearly perpendicular to rock surface in lower portion and vertically down in the above lateritic soil layers. In the second case, rock anchors were placed nearly perpendicular to rock surface only below 3 m berm (Fig. 5) at the interface between the medium hard laterite and completely weathered rock, while in the third case inclined anchors were placed in top and bottom layers only. It was inferred from numerical analysis results that while stability is ensured in last two cases under saturated conditions, the most favorable conditions in terms of construction difficulty were envisaged in installing rock anchors only in the lower 3 m berm part considering large area and height of the slope. In order to further verify this pattern (Fig. 5b); numerical studies were also carried out to study effectiveness of this reinforcement pattern for Cases 4 and 5 slope sections that were also critical under saturated conditions (Table 2). The results of the numerical analysis obtained after reinforcement for Cases 4, 5 and 6 are shown in Table 3. The slip surface developed before and after reinforcement for case 6 are also shown in Fig. 5. As seen from Table 3, factor of safety using this pattern (Pattern 2) increase factor of safety considerably ensuring stability all along the slope in saturated conditions.

4 Tunnel–Slope Interaction Study

In order to ascertain effect of tunnel excavation on stability of slopes, a tunnel–slope interaction study was carried out using the numerical modeling finite element code Phase 2 (RocScience 2008b). The portal in this study as explained earlier consisted of a water curtain tunnel, 6.5 × 6.5 m size and main access tunnel of 12 × 8 m size, both having D shaped cross-section. The portal of the water curtain and access tunnels is located in poor bedrock conditions consisting of highly jointed rock and low overburden. The numerical model conceptualized for carrying out this study is shown in Fig. 6 which is selected to review the overall stability of slopes and tunnels. For numerical model, a plane strain model is adopted with water curtain and access tunnel placed 10 m apart representing beginning section at 0–0 chainage. The overburden above the tunnel varies from 15 m for the water curtain tunnel to about 20 m for the access tunnel. The material geometry of the model was divided into three layers; soil, weathered rock and bedrock. Vertical boundaries of the model were restrained in x-direction and the bottom of the model was restrained in both x- and y-directions. In order to avoid any influence of boundary condition on stress–strain distribution of tunnels, model dimensions were extended to a minimum distance of four times diameter of equivalent tunnels. Three nodded triangular elements were used to model soil elements along with graded type mesh with higher mesh density considered around tunnels which is critical to stress concentration during excavation, effecting overall stability of structure. The input parameters used for deformability characterization of soil, weathered rock and bedrock are shown in Table 4, while Mohr–Coulomb failure criterion was adopted for shear strength determination of soil and rock materials. During construction of the access and water curtain tunnels in the portal region was supported by steel ribs in addition to rock bolts of 3.5 m length and fibre reinforced shotcrete of 200 mm thickness. However in numerical modelling only effects of the rock bolting and shotcrete is considered to represent the interim stability conditions. Complete simulation of tunnels

Table 3 Different cases analyzed for slope reinforcement study

Case no.	Location	Saturated (FS $\geq$ 1.2)		Remarks
		Without reinforcement	With reinforcement	
4	Right side slope of AT at starting point	0.99	1.30	Stable after reinforcement
5	Right side slope of WCT at starting point	0.91	1.31	Stable after reinforcement
6	Right side slope of AT, 10 m outside	1.09	1.23	Stable after reinforcement

AT access tunnel, *WCT* water curtain tunnel

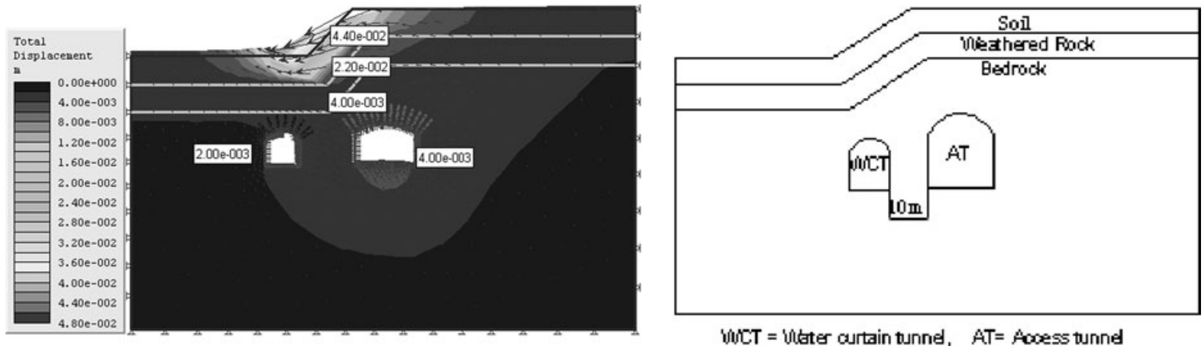

Fig. 6 Model geometry along with total displacement contours after analysis (contours reported in metres)

Table 4 Input parameters for tunnel–slope interaction study

Material	Unit weight (kN/m^3)	Young's modulus (MPa)	Poisson's ratio	Internal friction angle ϕ (°)	Cohesion c (MPa)
Soil	20.0	30–60	0.35	30	0.02
Weathered rock	22.0	1,500–4,000	0.30	31	0.03
Bedrock	27.5	5,000–10,000	0.23	36	2.50

Table 5 Displacement values for slope and tunnels

Case no.	Displacement in soil layer (slope part) (mm)		Vertical crown displacement (mm)		Max horizontal displacement right side of access tunnel (mm)
	Vertical	Horizontal	WCT	AT	
Case 1	7	24	0.45	0.90	1.7
Case 2	15	40	0.8	1.7	4.0

AT Access tunnel, *WCT* Water curtain tunnel

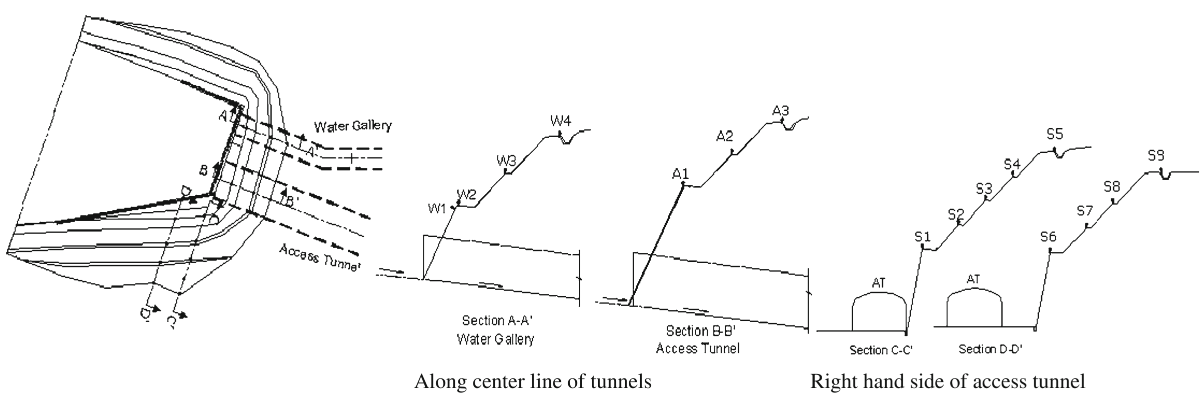

Fig. 7 Optical survey target installation on slopes in portal area

excavation along with support installation were carried out in eight excavation stages, starting with application of field stresses followed by excavation and support application. The water curtain tunnel excavation was carried out first as planned in actual construction schedule, because of size consideration and poor rock condition followed by initial support application in form of shotcrete and rock bolting in the next stage. Load splitting criterion was used in numerical simulation to account for partial load application, between different stages of modeling, rather than applying entire load in the first stage of excavation. Load splitting method is a concept and means of simulating 3D effect of an advancing tunnel face, using a 2D model (Panet and Guenot 1982). During tunnel slope interaction study, the elasticity modulus of the soil, weathered rock and bedrock were also varied as shown in Table 4 to take into account variable character of soil and rock behaviour at the portal region. Displacements values obtained for slope region, at crown of access (AT) and water curtain tunnel (WCT) from tunnel–slope interaction study are summarized in Table 5. Case 1 in Table 5 refers to upper bound value of modulus of soil, weathered rock and bedrock selected for finite element analysis while Case 2 represents lower bound values. Based on the finite element studies it was observed that maximum horizontal displacement of 4 mm was observed along the right side of access tunnel for lower-bound values of modulus of elasticity (Fig. 6). The maximum vertical displacement of top soil layer in slope region was found to be 40 mm in case 2 where lower-bound values of soil modulus (E = 30 MPa) were considered. As noted from Table 5, displacement around the tunnels are observed to be stable while comparatively larger displacements are obtained in soil slope region against lower bound soil modulus values. Since in this interaction study, no slope reinforcement was considered and only tunnel support in terms of rock bolt and shotcrete was considered, stability of slopes and tunnels is expected to be better and further improved by use of steel ribs in tunnel portal and rock anchors for slope reinforcement as discussed before in slope stability study. During construction, slopes were also further reinforced with additional layer of wiremesh and shotcrete all along the surface to ensure stability.

5 Geotechnical Instrumentation and Monitoring

Geotechnical monitoring has now become an important part of construction mechanism all over the world and especially in underground projects where proper monitoring can results in preventing large failures and damages. It helps us in understanding the performance of circumferential ground along with the effect of tunnel supports ensuring stability of tunnels, as a structure is subjected to varying loads. After completion of main construction part of portal slope and before the excavation of underlying water curtain and access tunnel, it was planned to continuously monitor slope movements for any adverse indications. Convergence monitoring was conducted using total station and optical targets all along the slope in order to understand their impact in relation to blasting of underlying access and water curtain tunnels. The distribution of optical targets proposed for slope area above water curtain and access tunnel is shown in Fig. 7. Optical targets were installed all along the slopes referring to centre line of access and water curtain tunnel and on right hand side of the access tunnel (along highest slope) and water curtain tunnel.

Monitoring of displacement was carried out on daily basis initially and further intervals were adjusted in reference to displacement pattern or distance of the working face from the main portal location. Monitoring of targets was carried out from day one of installation of measuring instruments to the convergence confirmation of displacement. It was further decided that even when convergence of displacement was confirmed, the convergence recording of continuing displacement would be continued for at least next 30 days to eliminate any chances of error. The optical monitoring data in form of resultant displacements of a point were plotted against the actual date of recording for overall assessment of slope movements as shown in Fig. 8a, b. The figure shows plotted distribution of convergence measurements for “centre of access tunnel” and on “right hand side of access tunnel” for main monsoon period (June to September) at site which was critical to stability of slopes. Timing of convergence readings coincided with start of excavation of portal of access and water curtain galleries. Some higher values of displacement were recorded during this period (Fig. 8a) which was mainly induced as part of blasting occurred in tunnels just beneath the slope location. Any displacement in slope movement was substantiated with similar pattern of measurements recorded inside tunnels which showed overall accuracy of measurements. Similar trend of observation were also observed for slope just above the centre of water curtain tunnel in starting chainages where poor rock conditions were encountered. Monitoring of tunnels and slopes were continuously examined during

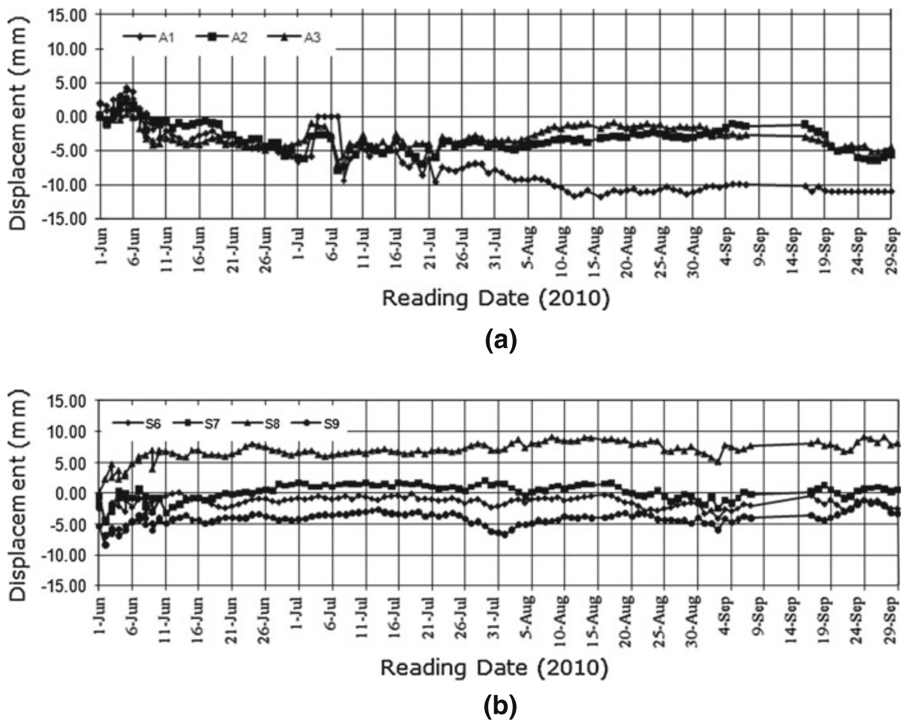

Fig. 8 **a** Convergence monitoring above the centre of access tunnel. **b** Convergence monitoring on the right hand side of the access tunnel

construction period and it was inferred from data observations of one monsoon period that overall displacements at the slope were found to be within the acceptable limits and do not poses any stability problems.

6 Construction Requirements for Ensuring Slope Stability

Portal slope construction needs to be executed safely following design guidelines so that any form of collapse resulting due to lack of bearing power and sliding is completely prevented. During construction of slopes, in order to ensure proper design confirmation, some basic guidelines need to be ensured. Since the site is located in an area having a heavy monsoon period; receiving average rainfall of above 4,000 mm, proper ditches at the ridge of a mountain needs to be installed for drainage so that the rainwater around the portal shall not flow into portal region during rainy seasons. In order to drain out percolating water from lateritic soil and weathered rock during heavy monsoon period, installation of 30 cm long perforated pipe (diameter 50–100 mm) at the interval of 2.0–3.0 m at the slope of 5°–10° was carried out so as to provide shallow drainage (horizontal drainage materials) channels along the slope to avoid the development of excessive pore pressure due to critical nature of weathered rock to saturated conditions. Further long drainage holes of about 20 m long were planned to drain out infiltrated water from underneath the lateritic soil layers. In the long run, this will reduce the development of high pore pressure and increase the stability of the slopes as a whole.

7 Results and Discussion

The significance of investigations on laterite soils with regard to slope stability of a portal is an important problem in tropical geological environments. A thorough investigation planning as illustrated during this study is undeniably important to prevent reported cases of failure in lateritic soils. Shear strength determination using undisturbed samples was found to be consistent for laterite due to variable nature of such soils reported. It was observed during stability analysis that presence of cohesion in laterite significantly affects the overall stability of slope. Similar observations were also made by Gogo-Abite (2005) who measured the stability of slopes in terms of reliability indices. It was also noted during stability analysis that saturated condition as compared to dry condition significantly effects stability of slopes in terms of factor of safety, which ranges 30–40%, while depending on other field conditions as height of slope and good bedrock availability. Similar impact of saturation were also reported by Tohari and Sarah (2006), Shou (2000) and Gogo-Abite (2005) who indicated that continuous rain water seeping significantly reduce the factor of safety of laterite soils. Gogo-Abite (2005) has observed that the shear strength of laterite can also decrease by as much as 40% at a degree of saturation of above 95% with all other parameters kept constant. The decrease in factor of safety due to saturation was mainly attributed to the loss of fines and the iron oxide, which acts as binding agent in laterite soils. Saturation condition finally leads to a less stable embankment with changes to the skeletal microstructure owing to presence of porous regions with more segregated particles.

8 Conclusions

In this paper, a case study of stability of slopes existing in varying combination of laterite soil and weathered rock formations were described. The results of the study support the following conclusions:

1. Variable character of laterite soil needs to be properly investigated along with use of testing techniques like undisturbed sampling that provides better estimation of shear strength parameters.
2. Different environmental conditions along with varied representative slope sections should be analyzed for stability determination while selecting factor of safety considering both global and local conditions.
3. Saturated conditions shows significant impact on stability of laterite soil that needs to be properly analyzed with provision of optimum reinforcement.
4. Slope stability of the structure should be verified both in individuality and in coordination with underlying tunnel construction for studying any alarming interaction effects.
5. Geotechnical instrumentation and continuous monitoring are the most important aspect of evaluating a successful design approach and should be implemented for preventing catastrophic failures.
6. Finally construction strategy should be planned favoring site conditions resulting in strong design practices to achieve a stable and safe configuration of structure.

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