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WEDGE FAILURE ANALYSIS AND REMEDIAL MEASURES IN LARGE UNLINED ROCK CAVERN: A CASE STUDY

ANALIZA PĘKNIĘCIA KLINA W DUŻEJ, NIEOBUDOWANEJ KOMORZE SKALNEJ ORAZ PODJĘTE KROKI NAPRAWCZE: STUDIUM PRZYPADKU

This article presents a case study of a large wedge failure. It took place during excavation of the last bench of storage cavern with an approximate dimension of 80 m long having a depth of 8 m. The adopted intervention followed a structured approach, which included immediate rock support, geotechnical and geological investigations in the failure zone and design modifications. Back analyses of the failure zone were also carried out to assess design parameters with observed geological conditions. Re assessment in the failure zone was carried out using modified design parameters, which included shorter benches, rock support installation schemes such as longer rock bolts, reinforced ribs of shotcrete and reduced construction advances. Geotechnical monitoring in and around failure zone were carried out for recording any alarming movements in the rock mass. Initially, geotechnical monitoring was carried out in the recently excavated zone of the cavern on a daily basis. Based on continuous monitoring data for at least one week, the frequency of subsequent monitoring can be decided. In most cases the deformation of rock mass was considerably less than the alarming values which were calculated based on detailed design for different rock classes. The paper discusses the failure, investigation, cause, assessment and remedial measures to complete the construction of cavern.

Keywords: Wedge Failure, Unlined Rock Cavern, Design and Analysis, Geological Investigations

W artykule przedstawiono studium przypadku pęknięcia dużego klina w trakcie prac wydobywczych prowadzonych w obrębie dolnej ławy pokładu w komorze magazynowej, o przybliżonych wymiarach 80 m długości i 8 m głębokości. Przyjęty model postępowania oparty był o podejście strukturalne, zakładające natychmiastowe zastosowanie podpory, badania geologiczne i geotechniczne w strefie pęknięcia oraz projekt modyfikacji. Przeprowadzono analizę wsteczną przebiegu powstania strefy pękania aby określić parametry obliczeniowe uwzględniające istniejące warunki geologiczne. Ponownej oceny strefy pękania dokonano w oparciu o nowe parametry i z uwzględnieniem projektowanych modyfikacji: skrócenie ławy, instalacja podpór, dłuższe śruby kotwowe, wzmocnienie żeber zaprawą, ograniczenie tempa postępu konstrukcji. Monitorowanie geologiczne w samej strefie spękań i w jej otoczeniu prowadzono w celu

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rejestrowania wszelkich niepokojących ruchów górotworu. W początkowym etapie, monitoring geotechniczny w strefie urabianej prowadzono w trybie codziennym. W oparciu o dane z ciągłego monitoringu zebrane w przeciągu co najmniej jednego tygodnia ustalono następnie tryb monitoringu w dalszych okresach. W większości przypadków odkształcenia górotworu okazywały się znacznie mniejsze od wartości krytycznych obliczonych na podstawie szczegółowych analiz poszczególnych rodzajów skał. W artykule omówiono zagadnienia związane z samym pęknięciem, badaniem jego przyczyn i określeniem niezbędnych działań naprawczych w celu ukończenia konstrukcji komory.

Słowa kluczowe: pęknięcia klina, nieobudowana komora skalna, obliczenia i analizy projektowe, badania geologiczne

1. Introduction

Wedge failure in large undergroun excavation can result in ground instability even in hard rock condition (Potvin & Hadjigeorgiou, 2008). Such failures are always associated with increased cost and significant delay. Few studies are documented (Struthers et al., 2000; Beck & Sandy, 2003, Potvin & Slade, 2007; Sandy et al., 2010 etc.) regarding the failure of underground openings in hard rock condition. However, most of these case studies are linked with mining activity where the size of excavation and type of rock support is completely different when compared with large scaled cavern excavation. These differences mainly due to increased span of excavation and higher life expectancy associate with more geological surprises. To handle such large underground excavation requires robust planning including different stages of geotechnical investigations, adoptive design approach, necessary contract provision to deal with associated geological risk and most importantly sharing of past documented experiences. The case studies related to challenges faced in similar underground excavation is scanty (Read, 1994; 2004; Andersson, 2005b) due to limited number of projects and also most of these projects were built for different strategic purpose which restrict them from publication.

The present paper discusses a case study involving a large wedge failure during the final stage of construction of large unlined crude oil cavern excavation due to adverse geological conditions. The problem was analysed using a structured approach which included immediate rock support installations, stage wise geological and geotechnical investigations in the failure zone, followed by design modifications such as change in dimension of the cavern, modified rock support installation and reduced size of staged excavations.

The rock cavern taken for the present case study is being used as Strategic Crude oil Storage project in east coast of India. The project consist of caverns of U-shaped in plan, D shaped in cross section and are kept parallel to each other. Orientation (trend) of the underground cavern is kept at N70°E. The underground facilities as shown in Fig. 1, essentially consist of access tunnel, main cavern (30 m height, 20 m width and about 840 m length) and water curtain tunnels running parallel to and 20 m above each U shaped cavern with a series of water curtain boreholes drilled perpendicular to it. Excavation of cavern was carried out in different stages (Fig. 1) to reduce the unsupported span thus to increased the stability of excavation (Mandal et al., 2013).

Water curtain tunnel, which is excavated before the cavern excavation, also acts as a pilot tunnel exposing the actual geology to be encountered during cavern excavation. The project is located within the geological setting of Eastern Ghat mobile belt, which is characterized by khondalites as the predominant litho unit. The topography of the area exhibits an undulatory hilly terrain flanked by two valleys one to the north and oriented east west and one to the east oriented north south. The ground elevation varies from +10 m to +125 m. Khondalite in the area is covered

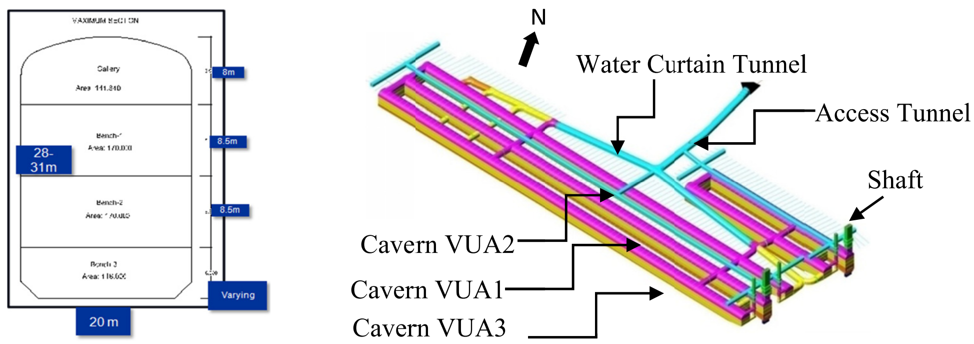


Fig. 1. Schematic layout of the underground storage facility

with soil and laterite as capping (thickness varying from 1 m to 6 m). Weathered Khondalite layer persists up to a depth of approximately 30 m below ground level.

2. Site investigation and geoetchnical parameters

Construction of underground caverns requires detailed set of investigation to develop geological and geotechnical model for all the parts of underground storage system. Two phases of site investigation campaigns were performed to get better understanding, firstly during the detailed feasibility stage and secondly during performance of the basic engineering design (BED) stage. As part of the site investigation campaign various investigations were carried out viz. geological mapping, core drilling, laboratory testing of core samples, seismic refraction survey and in situ stress measurement uses hydro-fracturing method at cavern level. Hydro-geological containment being the guiding principle of storage, required water pressure tests, groundwater level monitoring and groundwater analysis were also carried out. Results from both the site investigations were compiled and studied for characterization of the proposed underground rock cavern storage site.

During the course of investigations, two major shear zones which running through the project area were identified from first stage of investigation and later on re confirm from next stage investigation. One of them running along the valley trending E-W, the other one along N-S aligned to the valley. The cavern layout was chosen in such a way to avoid interference with these shear zones. The poles and rosette plotted from the geological data in site investigation showed strong bipolar distribution, with a total of three joint sets having a dip direction/dip of J1:N165/65, J2:N70/65 and J3:N10/20 respectively.

Geotechnical parameters of rock mass, in-situ stresses and joint orientation etc were derived from the results of geotechnical investigation, surface mapping, laboratory and fields tests performed during site investigation stage. The Geological Strength Index, GSI was also measured from field observations of blockiness and discontinuity surface conditions. Unconfined compressive strength values (UCS) and tensile strength were obtained from laboratory tests. Majority of the excavation work is expected to be made in a rock mass with an average Q value (Barton, 1974) of 10 corresponding to "fair/good" rock. Based on the input data from site investigations and subsequent analysis, intact rock parameter design values adopted at the basic engineering stage are presented in Table 1.

TABLE 1

Geotechnical parameter adopted at Basic Design Stage

	Bulk density [t/m^3]	UCS, dry [MPa]	UCS, wet [MPa]	σ_t [MPa]	Young's modulus [GPa]	Poisson's ratio
Intact Rock	2.82 ± 0.11	78.4 ± 24.9	63.5 ± 21.7	9.0 ± 2.9	70	0.18

In-situ stresses measured in the rock using hydro-fracturing 9.2 MPa major horizontal stresses oriented at N20°E and 3.7 MPa as minor horizontal stress oriented perpendicular to major horizontal stress. The proposed cavern is excavated at an angle 50° with respect to the maximum horizontal stress. The Hoek-Brown (1980) parameters used in the analysis are given in Table 2.

TABLE 2

Measure geotechnical parameter used for numerical analysis

	Barton's Q-value	Rock Parameters					Hoek Brown Criterion			Mohr Columb Parameters	
Rock Type		σ_{ci}	GSI	m_i	E_I	E_{RM}	m_b	S	A	c	Φ
Fair Rock	4 < Q <= 10	70	60	28	25000	7838	4.695	0.0059	0.5028	—	—
Poor Rock	1 < Q <= 4	50	50	28	15000	2572	3.004	0.0016	0.5057	—	—
Very Poor Rock	Q <=1	30	40	28	8000	1420	1.922	4.56 e-04	0.5114	—	—
Sheared Rock	Q ≪ 1	—	—	—	—	710	—	—	—	0.1	25

3. Design appraoch for caverns

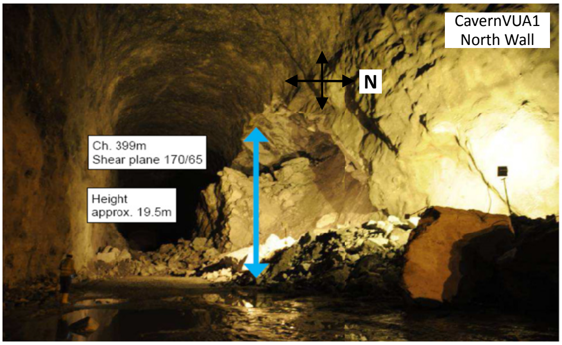
initial design of cavern consists of essentially two parts, the first part included assessment of rock support requirements based on empirical approach like Q system (Barton, 1974)which is derived based on classification of rock mass according to different geological parameters as outlined in the Q classification system. In continuity to first part, second stage of design involves verification of adopted approach using numerical analysis. This is carried out first by evaluating the extent of possible wedges formed using statistical joint sets and analyze stability of such wedges in the jointed rock mass conditions. The other aspect includes continuum two dimensional and three dimensional analyses using finite element programs to analyze stress-deformation patterns of caverns due to distressing of the rock mass resulting from the excavation sequences. Finally based on the results of numerical analysis and Q system, typical recommended rock support in terms of 25 mm diameter rock bolt of length 4-6 m with a spacing varying from 1.2 to 2 m and steel fibre reinforced Steel fibre reinforced shotcrete (SFRS) of thickness varying from 50 mm to 150 mm were adopted.

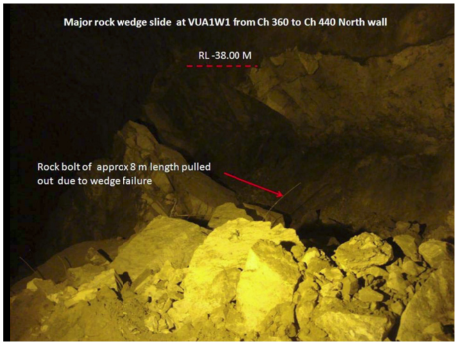
Geotechnical monitoring of all cavern units were performed using optical targets and three point borehole extensometers. Monitoring stations within the caverns were typically arranged at every 25 m interval with five optical targets being placed within the top heading and two targets on either side in each bench of excavation. In addition, some extensometers at geologically critical areas were also installed in the drilled boreholes downwards from the water curtain tunnel located above the cavern. Regular geotechnical monitoring was carried out and recorded during cavern excavation. Further during the course of construction as part of an active design approach, initially developed geological and geotechnical model are continuously updated with excavation progress especially during excavation of water curtain and during excavation of the top heading of the cavern.

4. Wedge failure: causes and interpretation

during the construction of one of the cavern (VUA1), a large wedge failure as shown in Figure 2 (a,b) took place. It occurred between the chainage of 360-420 on the northern side wall of the central cavern VUA1 during excavation of the last bench (bench 3) with an approximate dimension of 80 m long with a height of 20 m and extended upto 7-8 m inside the cavern wall. Scaling on the north wall at the said location was completed and mucking operation was on when failure occurred. The rock wedge slide started rom the bottom of the top heading (Reduced Level-38.0 m) and continued till the bottom of the third bench (Reduced Level-58.0 m). All the installed steel rock bolts were completely yielded, however the interface of grout was intact as shown in Fig. 2(a,b).

During site inspection of the wedge failure, it appeared that one of the shear failure planes (Fig. 3) found during the course of investigations, intersected north wall of cavern VUA1, causing





(a) (b)

Fig. 2. (a) Wedge failure in VUA1 cavern as seen from the east looking west (b) Yielded rock bolt

Fig. 3. Shear zone exposed after failure

significant impact on the wall stability resulting in wedge failure. This shear plane had although crossed the cavern earlier at different chainage, on both north and south wall sides, did not cause any damage. It further continued sub parallel to north wall for a longer distance before dipping towards cavern north wall. The dip was observed to be about 75 degrees and the shear appeared to run about 20 m beyond the wall at heading level which moved closer and hit closely bench 3 of the north wall. It was observed that during excavation of the 3rd bench, the toe of the wedge on which the shear plane was resting, got removed and thus got destabilized leading to a major wedge failure.

The rock mass was found to be damp to dry at the time of failure with no apparent water inflows. Immediately after the wedge failure, additional investigations were carried out in the northern wall to study the actual geological conditions within the wall area on both eastern as well as western sides. Series of core holes were drilled in the pillar area to assess the geology and necessary rock support.

5. Remedial measures and design changes

5.1. Initial Remedial Measures

Soon after reviewing the geological data derived from the excavation mapping sheets corresponding to wedge failure zone, some initial remedial measures such as scaling, core drilling, installation of additional rock bolts, shotcreting, etc were performed. Water pressure in water curtain tunnel bore holes were reduced.

Eight numbers of core drilling from Cavern VUA2 south wall through the pillar to Cavern VUA1 north wall was done to establish reason that could have triggered wedge failure and to ascertain other comparable features that could be encountered or not in the remaining excavation portion of VUA1 bench 3 region. Since after the wedge failure, there was also concern about the stability of roof, additional rock support in form of anchor rock bolts from the water curtain tunnel as shown in Figure 4 (B_1, B_2, B_3, B_4) into the cavern crown was installed to provide additional lateral confinement stress to the rock arch above the cavern crown. This was also intended to provide adequate safety for the man and machine that would be deployed for undertaking final

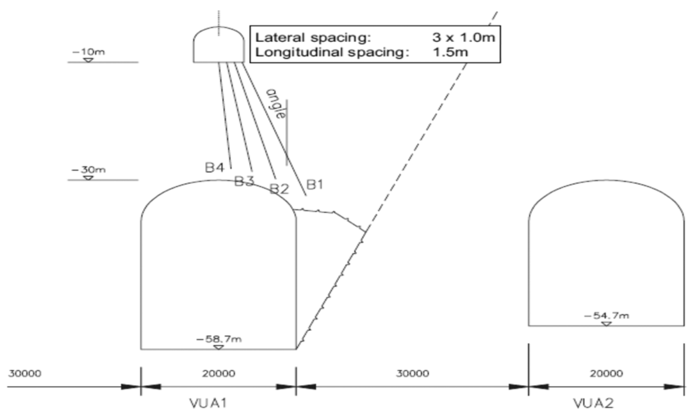


Fig. 4. Rock support from water curtain tunnel located over the affected zone

remedial rock support inside the cavern. Access ramp in VUA1 failure zone was built up to reach to the top of the failure wedge for installation of permanent rock support. Some additional optical targets were also installed in this zone.

5.2. Stability Check and Rock Support Assessment

Based on the additional geological / geotechnical investigations performed after the failure, detailed rock support assessment was made to check the global pillar stability between the two parallel caverns along with stability of VUA1 cavern north wall at different locations. In this paper, three such locations; Chainage 360 (representing section just before failure zone), Chainage 400 (section in the failure zone) and Chainage 560 (section after the failure zone) were re-analysed for stability and additional support requirements considering position of shear seam and other poor rock conditions encountered during coring carried out as part of additional post failure investigation.

In order to derive geotechnical parameters relevant from the wedge failure zone, back analysis was carried out as per the actual geometry. A limit equilibrium analysis was performed considering the friction and cohesion along the failure plane as retaining forces and the dead weight of the rock material within the wedge as driving force. The ration between driving and retaining forces can be interpreted as factor of safety against sliding along the shear plane. Based on the above analysis, an ultimate friction of 25^0 and cohesion of 100 KPa was considered for the sheared rock material. Rock mass properties for all other classes were taken as same as calculated during the detail design stage. However revised models were prepared for the above sections taking into account revised geology encountered at different chainages.

Table 2 shows the mechanical characteristics of different rock types adopted from rock mass classifications. These parameters were used for analysis and design purpose using Phase 2.0 software (RocScience Inc.).

5.3. Assessment of Pre and Post Failure Conditions

Detailed analysis of the required additional rock support in the sidewall of the affected cavern Bench 2 and the redesign of the rock support in the Bench 3 was carried out through Finite element analysis considering the updated geotechnical information. Based on the mapped geological features, present analysis was undertaken for three selected segments namely between Chainage (Ch) 360, between Chainage Ch400 and Chainage Ch 560.

5.3.1. Case-I, Chainage 360

This section represents section just before the failure zone and thus stability of this zone was re-assessed following revised rock mass assessments arising due to sheared zone. Model for Chainage 360, representing revised rock mass conditions is shown in Figure 5. Model shows three cavern units VUA1 in the centre and VUA2 & VUA3 on either side.

Initial rock support for benches was applied in terms of grouted rock bolts and Steel fibre reinforced shotcrete (SFRS) on cavern wall and heading. Fully grouted rock bolts (applied normal to the excavated boundary face) of 25 mm diameter and minimum yield strength of 500 MPa along with SFRS of minimum compressive strength of 40 MPa and shear strength of 1 MPa were used in model. Rock-bolt lengths measuring 5-6 m long in the crown and walls at an average spacing of 2.0/1.5/1 m in good/fair/poor rocks were typically provided based on rock-support chart recommendations using the Q-system of Barton (2002). Steel SFRS was applied in thicknesses of 50 mm as per the encountered rock classes.

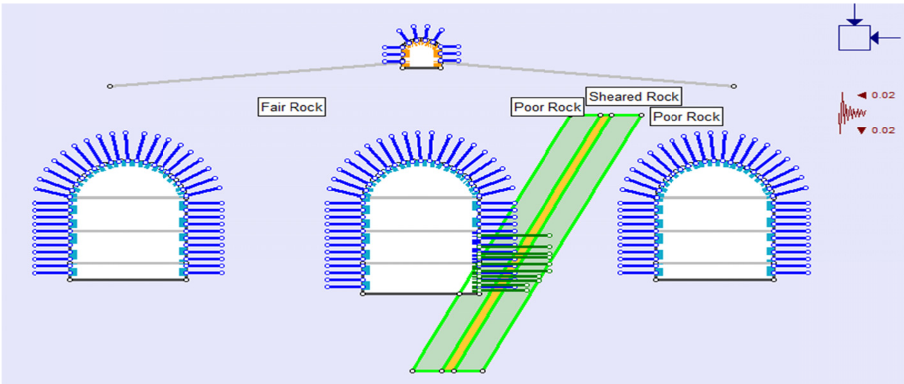


Fig. 5. Model representing Chainage 360, post failure condition

During analysis, it was observed that rock bolts installed earlier (25 mm diameter and 6 m length) in Bench 2 and 3 were found to be fully stressed to their ultimate capacities and yielding was observed to be extending beyond rock bolt length (5/6 m) as shown in Figure 6. Based on these findings and owing to encountered poor/sheared rock conditions, strengthening of rock mass around Bench 2 & 3 was carried out using 32 mm diameter rock bolts with lengths varying between 8-12 metres, as shown in Figure 6.

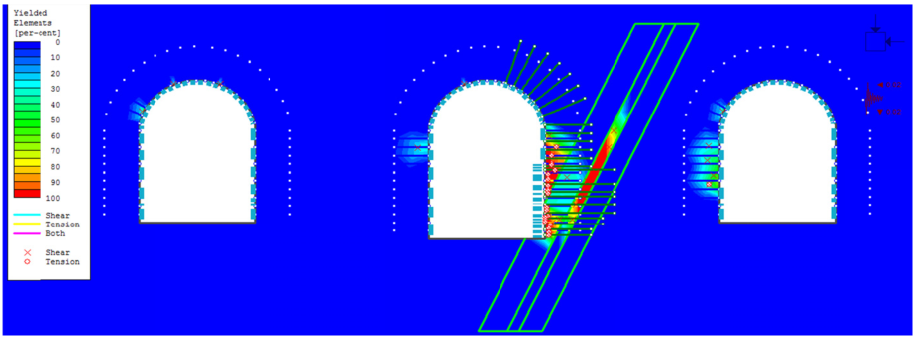


Fig. 6.Yielded zones around Bench 2 and 3, post failure condition

These additional long bolts served dual purpose of stabilizing the yielded zone as well as stitching the poor rock mass around the cavern north walls. Maximum displacement around the walls was estimated to be around 15 mm, well within the permissible limit of allowable displacement of 0.1 % of cavern height (Zhao & Cai, 2010). As part of the geotechnical monitoring carried out in this area using optical targets, no alarming movements were recorded around cavern sidewalls during any stage of excavation.

5.3.2. Case II, Chainage 400

This section as shown in Figure 7 represents actual failure zone and thus revised model was developed as per the actual failure zone geometry and encountered poor rock conditions around walls for VUA1 cavern.

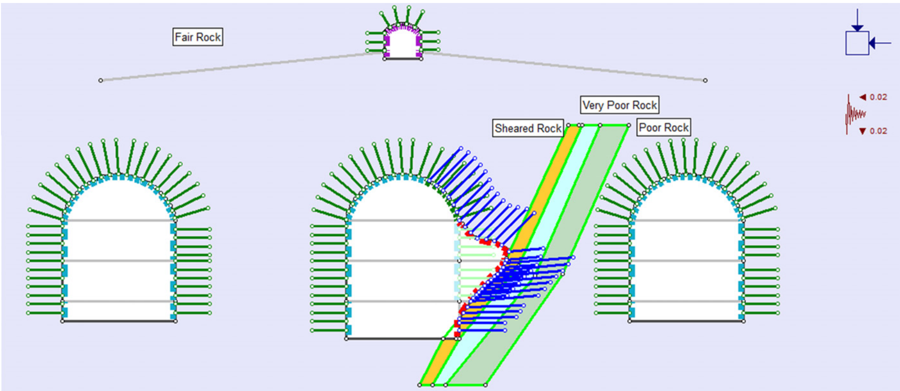


Fig. 7. Model representing Chainage 400, post failure condition

Since at the time of failure, bench 3 was incomplete and big wedge on north wall collapsed while taking a blast in Bench 3, these two sections (Bench 3 and additional wedge area) were excavated together in this model to study post failure stability conditions. As part of this revised model, initial rock supports in the failure zone were completely removed owing to part of the failed rock mass and revised rock supports in reference to poor and sheared rock conditions were applied. Based on the revised analysis, long grouted rock bolts of diameter 32 mm and lengths varying from 10 to 12 m along with 250 mm SFRS and additionally a vertical SFRS columns (width × depth = 650 × 400 mm) were sprayed additionally over the initial thickness of SFRS. This SFRS column was modelled as rock support liner with higher modulus values compared to previous SFRS layer. Zone of shear, poor rock and very poor rock were modelled around the caverns as per the results of cored investigation holes as shown in Figure 7.

It was observed from the analysis that revised support system installed in terms of long rock bolts and Steel fibre reinforced shotcrete (SFRS)were able to take up additional stresses build up on account of sheared rock mass conditions around the failure zone. Most of the yielding observed (Fig. 8) was confined to zone of around 4-5 m from excavated cavern boundaries. Stress levels around the cavern, and in particular above the failure zone, were moderate and in acceptable limits. The overall total displacements are relatively small and in the range of 15-20 mm which was well within acceptable limit of 0.1% of cavern dimension.

Additional optical targets were also installed in this zone based on the interpretation of monitoring plots of optical targets; however there was no indication of any significant rock mass movements in the vicinity of the wedge failure zone afterwards.

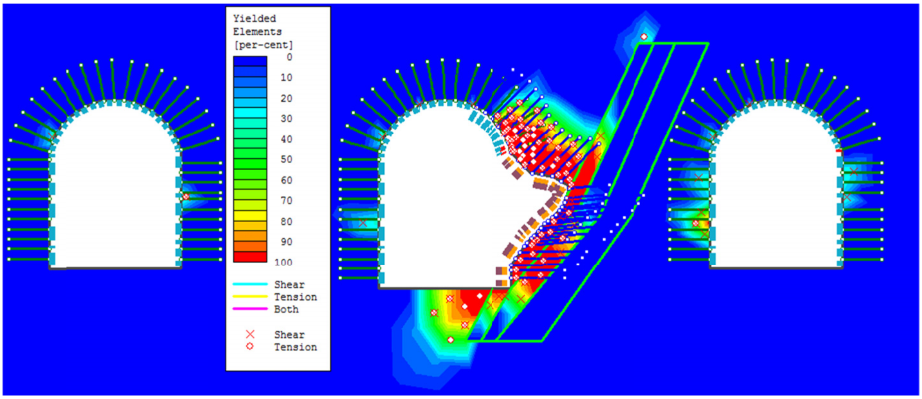


Fig. 8. Yielded zones around north sidewall, post failure condition

5.3.3. Case III Chainage 560

This section was selected as the zone between chainage Ch. 450 m to Ch. 615 m where excavation of third bench was yet to be performed. Based on the results of core drillings carried out within the pillar between caverns VUA2 and VUA1, possibility of similar sized wedges as that involved in wedge failure (Ch. 360 m to Ch. 450 m) were found to exist in cavern VUA1 northern sidewall between chainage Ch. 450 m to Ch. 615 m.

As no wedge failure has occurred in the cavern excavation of VUA1 from Ch. 450 m to Ch. 666 m, it was reasonable to conclude that the existing support measures were adequate to hold any possible wedges in the sidewall between Benches 1 and 2, in place. However there was a possibility that upon the excavation of the third bench a wedge may slide out. To guard against this possible sliding, side slashing of 2.5 m was left in place on both northern and southern sidewalls. By leaving the side slashing in place, it was found that it may not be possible for a wedge to slide out if the dip of the joint is greater than the slope surface angle of the side slashing, as the joints will be unable to daylight out of the excavation surface.

Thus referring to the stereo plot of the zone, slope of the third bench was restricted to 45 degrees. Model of this section as shown in Figure 9 indicates third benches with side slashing (top width of 2.5 m) intact for presence of shear zone on either side.

Rock support installed in the revised model included 32 mm diameter rock bolt of length varying from 10 to 12 m with a spacing of 1.5 m c/c along with steel fibre reinforced shotcrete (SFRS) of thickness 250 mm with one layer of wire mesh, as well as additional column of SFRS (650 mm width, 400 mm depth, spacing 2-2.4 m c/c fully extended over bench 2 side wall height. Further additionally, 8 m long spiling rock bolts of 25 mm diameter were also provided for protection of left-in place bench 3 side slashing.

The results indicated that along the side walls, yielding were mostly concentrated in the intact bench part with some fractions also observed near Bench 2 (Fig. 10). Yielding coupled with some stressing of the earlier rock bolts led to installation of additional rock bolts of 32 mm diameter and 10-12 m lengths in this zone for both northern as well as the southern side wall.

Post failure eight number of three point rod extensometers, recording measurements at lengths of 6, 12 and 20 m were installed at different locations along the north wall and four along the

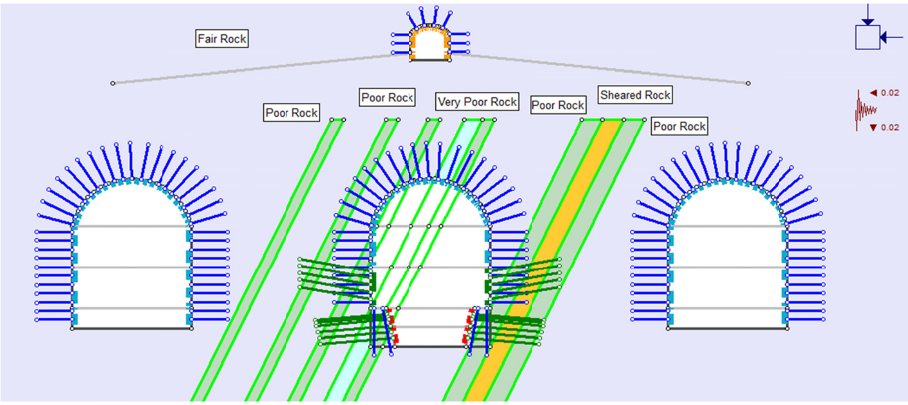


Fig. 9. Model representing Chainage 560, post failure condition

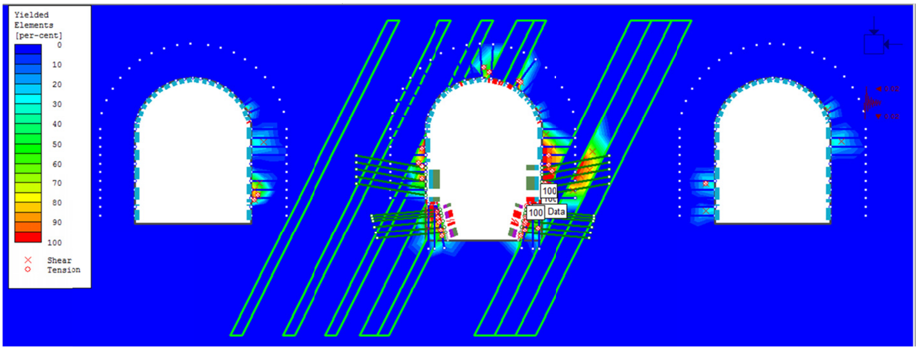


Fig. 10. Yielded zones around north& south side wall, post failure condition

south wall from about 2 m above the Bench 2 floor level, inclined 5-10 degrees down. Extensometer data obtained from this location pointed to 2 mm maximum horizontal displacement in post failure conditions at a distance of around 6 m inside rock mass after bench 3 excavation. Results obtained from numerical model for Chainage 560 indicated a net increase of 5-6 mm of horizontal displacement after Bench 3 excavation (keeping the side slashing intact) in post failure conditions. Thus displacement values in both the cases were found to be in acceptable limits. Further during excavation of the complete last bench (Bench 3) within the wedge failure zone, all geotechnical monitoring results (optical targets, rock extensometers, etc.), indicated stable rock mass conditions at all times. No monitoring results indicated any ongoing movements or any unpredicted excessive movements during the excavation of Bench 3 within the area of the wedge collapse.

6. Conclusions

This paper presents a case study involving back analysis and design modification carried out post large wedge failure during excavation of one of the benches (bench 3) in a crude oil storage cavern. Discussion of various investigations carried out post failure followed by remedial measures, analysis, design and geotechnical monitoring. The study showed that investigation carried after failure provided useful information for re-modelling various sections of the cavern wall for support enhancements with minimum time and cost implications on the project. The balanced excavation works were completed successfully providing stability all along the cavern length. This project was commissioned in the year 2015. Methodical approach as described here establishes the reliable way to deal with enormous geological uncertainties involved in large underground excavation. Lesson learned from this case study will help to develop confident among professionals in large underground excavation.

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