Slope Stability Measures Adopted In Kimi Powerhouse - Case Study of Kameng Hydroelectric Project

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Abstract

Kameng Hydro Electric Project (600 MW) is under construction in Arunachal Pradesh, India. The proposed semi underground powerhouse is being constructed at Kimi. The power house area of Kameng HE Project is located on Gondwana Supergroup overlain by thick overburden of about 25 m comprising river terrace deposit followed by colluvium deposits. The Gondwana Supergroup comprises alternate sequence of gray Sandstone, Carbonaceous Shale and Carbonaceous Siltstone with layers of coal partings. The rock mass is highly folded, jointed and crushed thus making the slopes unstable if not protected in a systematic way. The surface excavation of powerhouse is in progress. The slopes were protected by shotcreting with wire mesh and anchoring. Even after protecting the slopes, it failed by toppling and subsidence. The previous slopes had to be regraded after slope failure at some stretches. The new slopes have been treated as per the insitu rock mass characteristics and are stable since then. For excavating further down the slope to the foundation level, some more protection measures have been proposed by the consultants and executed partly. This paper describes the protection measures applied before and after the slope failure along with a critical analysis of the geotechnical aspects.

Introduction

North Eastern Electric Power Corporation Limited is executing the Kameng Hydro Electric Project in West Kameng district of Arunachal Pradesh, India. Kameng Hydro Electric Project (4 X 150 MW) is a run-of-the- river scheme to harness the hydroelectric potential of Kameng River utilizing a gross head of 536.00 m. The project envisages construction of a 75 m high concrete gravity dam across the river Bichom 3.65 Km downstream of the confluence of Bichom and Digien rivers and a 26 m high concrete gravity dam across the river Tenga 1.05 Km upstream of confluence of Bichom and Tenga rivers. The other components of the project include a 14,452 metre long, 6.7 m dia. Modified horse shoe shaped HRT, 70 m high and 25 m dia surge shaft of restricted orifice type, a semi underground power house (109m X 20 m) on the right bank of Kameng river and 3690 m long steel lined partly underground and partly surface penstock (5.30 m dia. – 490 m, 3.75 m dia. – 2 X 1470 m, and 2.65m dia – 4 X 260 m). The geological and geotechnical investiga-tions for the project were carried out by Geological Survey of India and later on by NEEPCO during project report preparation and execution. CMPDI, Ranchi was involved in the geo-structural mapping of the project area based on remote sensing data.

Regional Geology

The Kameng Hydro Electric Project area is geologically complex and tectonically highly disturbed, forming part of the Lesser Himalayan range. The area is dissected by a number of major thrusts and faults striking E-W to SW-NE associated with subduction of the Indian plate beneath the Asian plate (CMPDI, 2003).

The rock types occurring in the area belong to the older meta-sedimentaries of Pre-Cambrian age in the northern part, sedimentary rocks of the Gondwana Supergroup in the central and southern part. The Tertiary sedimentary rock of Siwalik Formation occurs in the southeast of the

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project area in the vicinity of Kimi power house (GSI, 1982).

Topography is steep and actively dissected by the main river systems being the Bichom/ Digien, Tenga and Kameng rivers.

The major rock units range in age from Pre-Cambrian to the Tertiary. Quaternary sediments are all over the project area. Table-1 presents the tentative geological sequence of the area.

Geology of the power House area

The powerhouse area is located on Gondwana Supergroup overlain by 25 m thick overburden comprising of colluvium deposit followed by river terrace deposit. The colluviums material is composed of assorted angular rock fragments ranging from 1 cm to 30 cm in one dimension which are graded in nature with gravely soil matrix with thickness of approx. 5 – 10 m. The colluviums' materials are composed of fine grained to coarse grained Sandstone, Carbonaceous Shale and Carbonaceous Siltstone. The river terrace deposit comprises horizontally bedded gravel and cobbles up to 200 mm size with a silt matrix and in some locations there are beds of fine sand and silt up to 2m in thickness. The alluvial deposit also comprises boulders of up to 200 cm in size. The pebbles, cobbles and boulders are of Sandstone, Quartzite, Gneisses, Schist and Phyilites. The contact surface of the overburden and the rock mass is much undulated due to presence of streams down the hill. Heavy seepage is observed at the contact plane of overburden and rock. The underlying rock of Gondwana Supergroup comprises Sandstone, Carbonaceous Shale, and Carbonaceous Siltstone with coal partings. Sandstone is light to medium gray, fine to coarse grained and medium strong rock. Carbonaceous Shale is weak and friable in nature. Carbonaceous Siltstone is medium strong. Coaly partings are totally crushed and graphitic in nature. The rock mass is fresh to stained, moderately to highly jointed, folded and sheared at places. The geological plan of power house area is shown in **Fig.2**.

Rock mass is dissected with 3 – 4 joint sets and appear undergone polyphase of deformation in different directions. Due to intense polyphase deformation, rock mass is highly jointed, folded and crushed. The engineering properties of rocks encountered in the power house area are widely different that the behavior is a totally varied to the stress applied at the time of polyphase deformations. Sandstone is strong and therefore behaved in a brittle manner to the stress, resulting into moderate to highly jointed rock mass. At some locations, Sandstone bedding is even detached from one another due to shearing. Whereas Carbonaceous Shale and coaly partings are so weak that they behaved in ductile manner. Due to folding and shearing, Carbonaceous Shale and coal partings adjusted itself in the space left between the competent rocks.

Table 1

AGE	ROCK UNIT	LITHOLOGIES
Quaternary		Alluvial terraces, Outwash fans, colluviums deposit
Tertiary	Siwalik Fm.	Fine-grained greasy Sandstone, grey to chocolate Shale with lenses of lignite and variegated/mottled Clay and Mudstone
		-(**Main Boundary Thrust**)————————
————————	————**THRUST**	Quartzitic Sandstone, Carbonaceous Shale/Siltstone with
Permian	Gondwana Group	coal bands (?=Damudas)
		Pebble Slate, Phyllite, Conglomerate, Quartzites etc. (?=Talcher)
		————————————————
————————	————**THRUST**	Chlorite-Sericite-Phyllite, Quartz-Muscovite-Sericite Schist,
Pre-Cambrian	Bomdila Group	Quartzite, streaky Gneiss and porphyroblastic Granite Gneiss

(Source : GSI report, 2001)

The joint characteristics have been tabulated in **Table 2**.

Sequence of excavation works

The Power House excavation work started on 03.03.2005 as per approved construction drawings. The NSL of the powerhouse area is at about EL 300 m, sloping steeply southwards to Kameng River at a angle of about 35 to 50 degrees, becoming steeper at the lower levels. After removal of the overburden materials, blasting of underlying rock was started in a systematic way to get the proposed slopes and berms. The first berm on rock was at EL 263.00 m. Kameng River is on the south side of the power house and therefore the slopes on the north, east and west sides were being made initially. Slopes on the overburden were kept at 2H:1V and therefore the slopes from EL 288.00 m to EL275.50 m and from EL 275.50 m to EL263.00 m were 2H:1V. The contact of overburden and rock was observed to be in between EL 275.50m and EL 263.00 m. Below EL 263.00 m rock was observed to be in all the slopes and therefore it was kept at 1H: 4V. Although the powerhouse excavation reached a bottom level of 227.50 m in the pit area, slopes were cut only upto EL 253.00 m.

Problems encountered

In April'2006, open excavation work was being done below EL 263.00 m berm. When the northern slope(1H:4V) below EL 263.00 m was being cut to 4 m depth, a crack was observed at about 4 m from the edge of the berm in between RD (-)55 m to (-) 33 m and Ch. 49.00 m to 38.50 m on 16th April, 2006. The width of the crack was observed to be approx. 30 cm at the top and filled with seepage water. The rock type encountered at this location was Carbonaceous Shale with the strike parallel to the berm edge and 80° - 90° dip. The strike of the bedding took a turn towards north at the extremes of the crack. The probable reasons for the crack formation could be attributed to the vertical dip and strike parallel to the berm edge of the poor rock mass combined with seepage water pressure. At EL 263.00 m berm parallel to C-line, the rock mass is moderately jointed with clay filling of 2 mm in the bedding joint.

As observed in August'2006, joints started to open up day by day and as a result the berm width (~10m) reduced to 5 – 7 m at some locations. Cracks of width up to 30 cm were observed to be forming in thickly bedded sandstone at EL 263.00 m berm for a stretch between RD (-)32 and (-)50m. This was localized toppling of the rock mass due to unfavorable attitude of the smooth bedding plane and heavy seepage from the upslope. The southern side of the berm at the place of slope failure comprised Carbonaceous Shale with coal partings. The berm width reduced due to collapse of the weak zone formed by Carbonaceous Shale. At EL 257.00 m cracks were formed between Carbonaceous Shale and Sandstone transition and filled with water (**Fig.3**). Immediate protection of the berm was felt necessary to stop further deterioration of the rock mass. As a temporary protection measure, the cracks were filled with sand/

Table 2

Joint Set	Attitude	Spacing	Persistence	Characteristics	Aperture/Filling
BJ1	51º-84º Due 323º - 348º	<5 – 20 cm	2 – 5 m	Smooth, Curvy planar	0 - 3 mm/ clay
BJ2	76º -87º Due 142º – 155º	<5 – 50 cm	> 5 m	Smooth, Curvy planar	0 - 3 mm/ clay
J3	40º – 79º Due 40º – 90º	30 – 200 cm	1 – 2 m	Rough, Planar	0-2 mm/Quartz filling
J4	41º – 73º Due 105º - 152º	20 – 60 cm	> 2 m	Rough, Planar	0-2 mm/ Clay
J5	45º -66º Due 250º -280º	10 – 100 cm	1 – 2 m	Rough, Planar	0-2 mm/Quartz filling

cement grout and then with PCC in proper grading as per the dimension of the cracks. This was followed by placing grouted rock bolts of 7m length, Ø25 mm, @ 2.0 m-2.5 m c/c in staggered pattern. Rock bolting was done from the slope face between EL 263.00 m and EL 257.00 m and from the berm at EL 263.00 m in 20 º downward angles towards switchyard. After adopting the above protection measures, slopes were found to be stable even in the monsoon season.

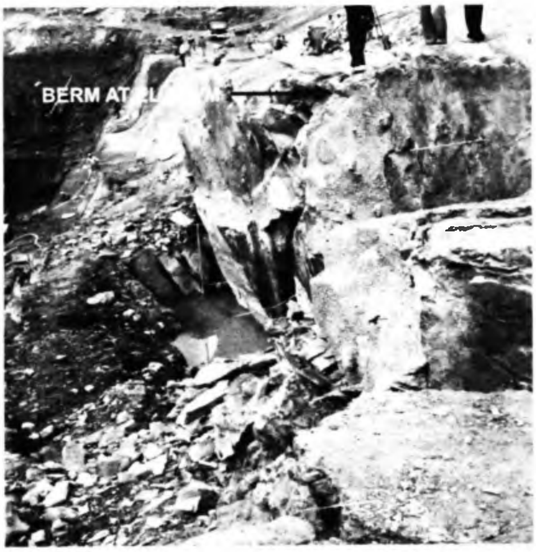

Fig. 3: Effect of toppling in Sandstone at EL 263 m (view to west)

Retrogression of cracks at different levels & observations made

The slopes seemed to be stable after adopting temporary protection measures, but, in the month of October '06 multiple vertical cracks were observed at the berm EL 263.00 m and 258.00 m. As excavation in the power house was in progress and reached the level of about 235 m (between A & D line), some subsidence cracks were observed at EL 253.00 m and EL 275.5 m and in the slope below EL 275.5 m (beneath the shotcrete face). Multiple cracks were also observed almost along the entire berm at EL 263.00 m. A crack measuring about 77 m long, 1 mm to 10 mm wide parallel to the berm at EL 275.50 m on the northern side was also observed. By the month of November'2006, cracks were found to be continuously widening up and propagating to the turfed slopes too. It seemed as if the whole mass was settling down differentially. The rock mass was either toppling or subsiding at different stretches. New monitoring pegs were placed on the cut slope below EL 243.00 m between D-line and E-line. The cracks in Sandstone beds were opening up along joint planes and the Carbonaceous Shale/ Siltstone beds were settling differentially between RD (-)21.50 m to (+) 19.00 m (**Fig.4**). Due to fast deterioration of the slopes, further excavation of power house was kept on hold from 25/11/06. To know the exact nature of failure, four nos. of drill holes (EBH 1, EBH 2, EBH 3 and EBH 4) were drilled on the berms at EL 275.50 m and 263.00 m. After drilling of the holes to a depth of 40 m, 25 - 30 mm PVC pipe were inserted in the hole to the base. These pipes were slotted in the lower 3 m to allow water flow and not the sand that was backfilled in the hole between pipe and the hole. A steel rod of one metre length connected to a string was lowered in all the holes. The rods were lowered to observe:-

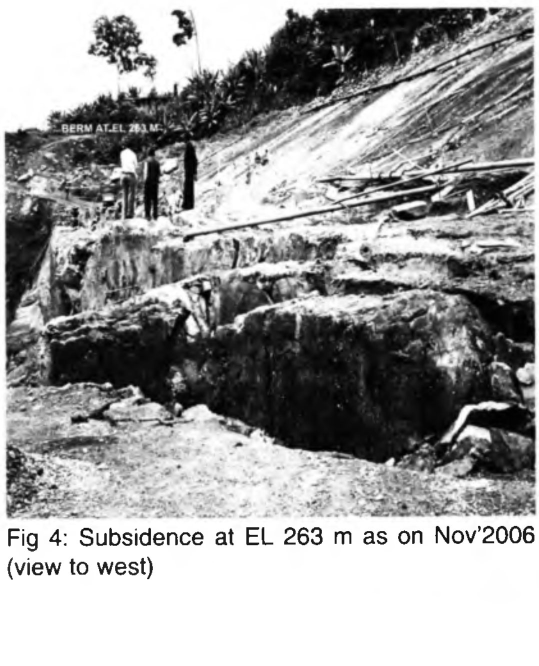

Fig 4: Subsidence at EL 263 m as on Nov'2006 (view to west)

a. water level

b. to locate the possible plane of failure.

If there was any movement in the rock mass, the rod would not pass through the hole and thus the depth to that plane could be calculated. The observations made in the hole, cracks and the pegs are as follows:-

a. No movement was observed from the drill holes. Thus, it was inferred that the failures were localized.

b. Cracks were observed to be widening up

c. Pegs were observed to be moving relatively to southward direction

Remedial Measures

Based on the observations of boreholes, crack and the geological formation, northern slopes were redesigned. The proposed remedial measures (**Fig. 5**) are as follows:-

i) Re-gradation of the entire northern side slope right from the level EL 288.00 m down to bottom most level EL 214.80 m along with all slope stability measures like turfing, shotcreting with wire mesh, rock anchoring, drainage etc.

ii) Re-gradation of the western side slope from EL 268.00 m down to EL 243.00 m including lowering/lining of natural nallah and construction of masonry wall at the bottom of the slope.

iii) Providing 30 cm diameter cast in-situ RCC bore piles in two benches namely at EL 248.00 m and EL 237.00 m in three rows in both the benches. The bore piles are having maximum length of 25.0 m and will be provided with all instrumentations required for monitoring.

Present status of re-gradation work

The re-gradation work on the northern slope of Power House was taken up from. 25th January'2007 as per the revised drawing from the topmost level EL 288.00 m. Excavation is immediately followed by installation of support system viz. shotcreting, rock anchoring and drainage holes which is designed on the basis of the rock strata encountered. Masonry walls have been erected at different levels to act as additional toe support wherever slopes were vulnerable. These walls have been placed at the stretches of weak and friable Carbonaceous Shale. Rock slopes have been restricted to 5 m height up to EL 248.00 m and slopes have been cut and supported taking 2.5 m height only at a time. The turfed slopes (upto EL 268.00 m) were provided in gradient 2H: 1V while the slopes with shotcrete & wire mesh were 1H: 2V and 1H: 1V. PCC was done on the top of all the berms while concrete catch drains were constructed on the berm at EL 268.00 m, EL 263.00 m and EL 248.00 m. The berm width is normally kept at 2-3 m, but higher berm width ranging from 5m to 7 m have been provided at EL 268.00 m, EL 248.00 m and EL 237.00 m for accommodating the proposed bore piles and from slope stability point of view. The slopes have been supported by spraying M-25 grade shotcrete of 100 mm minimum thickness provided in two layers separated by welded wire mesh/chain link fabric which is fastened to the face by the heads of dowel bars of 25 mm dia and 5.0 m length inserted into 100 mm dia holes filled with cement grout provided @ 2.0 m c/c spacing. Immediately after supporting, drainage holes of 6.0 m length were provided with 50 mm dia. Slotted PVC pipes @ 4.0 m c/c in staggered fashion. The work of re-gradation of the northern slopes has been done in the above fashion and the same was completed upto EL 248.00 m (i.e. the top piling berm).

The western side slopes also failed by bulging and sliding due to presence of large volume of overburden. The problem was complicated by the sand bedded natural nallah flowing along the western edge of the power house down to Kameng River from where heavy seepage was taking place towards the service bay. The re-gradation on the western side was taken up from EL 268.00 m where berms were kept at 5 m vertical drop excavating only 2.5 m at a time. Slopes were kept at 2H: 1V and 1H: 1V while the natural nallah was lowered and lined with RRM providing a suitable gradient. The re-gradation has been completed down to the service bay at EL 243.00 m where a masonry retaining wall of height 6.0 m is presently under construction.

Conclusions

In the Himalayan rocks, it is really very difficult to carry out underground or surface excavation. It has been well established that underground excavation is very tricky especially near the thrusted, faulted or sheared areas. The powerhouse of Kameng Hydro Electric Project area lies near the main boundary fault. Therefore, the powerhouse area is highly folded and sheared.

The stability of the powerhouse slopes was affected due to following reasons:-

a. Presence of Carbonaceous Shale/ Siltstone with coaly partings,

b. Unfavorable orientation of the bedding with respect to slope

c. High seepage

d. Removal of the toe support by excavating the pit area

The powerhouse slopes have been stabilized by re-grading the slopes and excavating in a systematic manner. The lessons learnt from this slope failure have been summarized below.

a. Slopes should be designed as per the actual rock mass behaviour at the site.

b. Excavation of the slopes should be restricted to a minimum depth in case of weak rocks.

c. Stabilisation activities like shotcreting, rock anchoring and drainage must be carried out concurrently and systematically with excavation.

d. Extra support such as wall or buttress should be provided at very unfavourable stretches.

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