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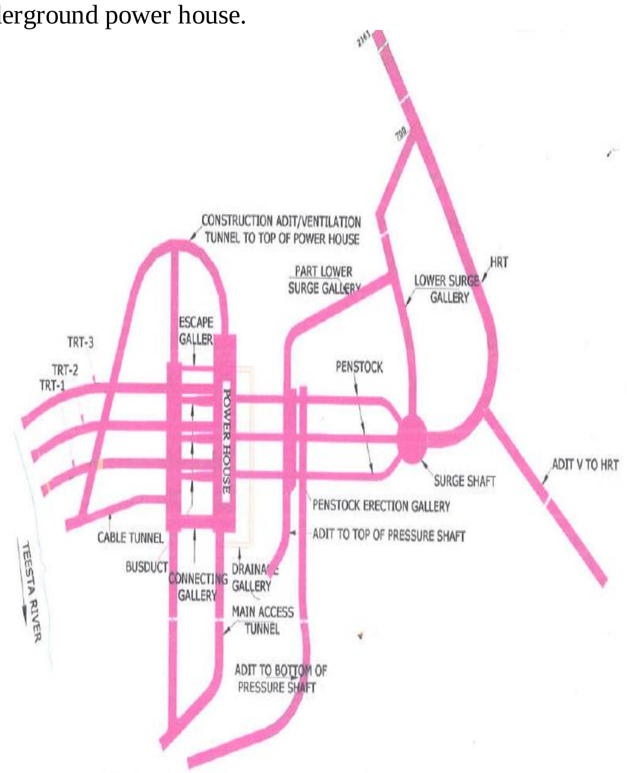
CONSTRUCTIONAL ASPECTS OF PRESSURE SHAFTS AT TEESTA-V PROJECT

B.K. Ojha
Ex-Chief Engineer (NHPC)

Abstract: This paper deals with constructional aspects and the problems encountered during excavation of the three pressure shafts of Teesta-V project. The pilot shaft excavation earlier planned with Raise boring machine could not be done and was alternatively executed by a combination of conventional Sinking method and Alimak Raise climber because of poor geological condition of the rock medium. The paper also summarizes the fabrication and erection aspects of steel liner.

I. INTRODUCTION

The Teesta Stage-V Hydroelectric Project is one of the six stages planned in the cascade development of Teesta river basin in Sikkim State. It is run of the river scheme with limited storage capacity utilizing a gross head of 216.73 M to generate 510MW (3X170MW) of power. The Project envisages the construction of three No. circular 4.7 M diameter steel lined vertical pressure shafts off-taking from base of Surge Shaft in a horizontal length of 115 M before dropping through a 90ºbend for a depth of 175 M and the running horizontally for a length of 55 M and feeding three number of generating Units each of 170 MW in an underground power house.



PLAN OF POWER HOUSE COMPLEX

II. GEOLOGY

The Project is located in the meta-sedimentary rocks of lesser Himalayas. The pressure shafts were excavated in foliated to jointed Quartziticpyyllite, phyllite quartzite inter bedded with bands of phyllite and quartzite. Pockets of sheared and poor rock mass zones were encountered especially in the penstock erection gallery and upper 60-70 M length of the three

vertical pressure shafts. The interfolial shears of thickness varying from 50-300 mm consisted of thin clay parting and crushed material. However bands of fair rock was observed from EL 460 to EL 350 with occasional to frequent poor rock and interfolial clay shear bands along with seepage of moderate to drips of ground water flow. The rock mass in the vicinity of the structure was subjected to extensive weathering. These conditions collectively reduce the strength characteristics of the rock mass. The highly weathered to occasionally decomposed condition in this low cover areas posed problems during excavation. The major part of Adit to PEG and PEG itself was excavated through a dominantly weathered Quartziticphyllite bearing rock mass by way of excavator only and not by blasting. The 115 M top horizontal pressure shafts were also excavated in poor tunneling media which required steel rib supports throughout the length.

III. EXCAVATION OF SHAFTS

The erection of steel liner was planned through a 62 M (L) 12 M (W) X 10 M (H) penstock erection gallery (PEG) having invert at EL 542.0 M. Three vertical shafts were excavated to join the top horizontal portion of the penstocks at its bend location at EL 525 M. The three Shafts with excavation diameter of 6.3 M was planned for excavation with Raise Boring Machine. This involved boring a pilot hole of 280 mm dia and then reaming it to 1800 mm dia. Thereafter the excavation was to be done by conventional drilling and blasting from top to bottom and mucking to be carried out from bottom by dropping/falling the excavated material through Pilot shaft. The details of Raise boring machine used for the work are:-

Name of Machine Robbins Raise Drill (Overhauled and reconditioned) Model No. 61R

Dimensions

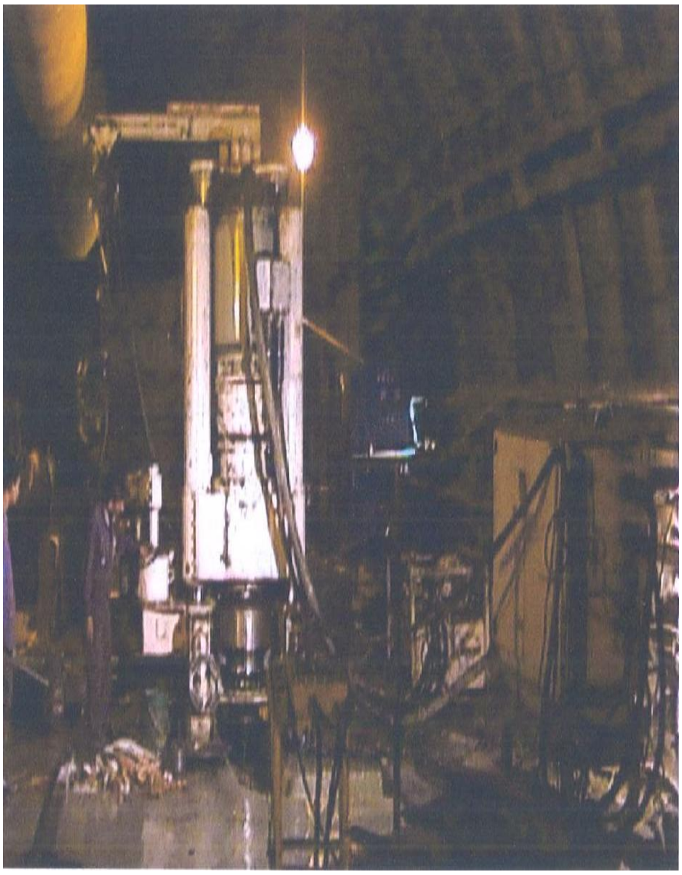
- Total Height (Derrick Extended) 5100m
- Total Height (Derrick retracted3710mm length

While on crawler 1275mm wide

Total weight i/c. weight of crawler 12.555mm T

Dia of Pilot Bit 280mm

Dia of reamer Bit 1800mm



A. Problems during Raise Boring:-
Raise borer was installed at PS-2 location on concrete platform in Erection Gallery and drilling started on 28th October 2002 to make a 280mm diameter hole. While drilling below the concrete foundation and upto 4.5M depth it was observed that the bed underneath consists of weak phyllite with thin clay seems. After drilling upto 14.89 M about 50% water loss was observed and drill material was not bailing out from the hole even after applying water pressure as high as 18 bar/ cubic meter. Grouting was again resorted to in the entire depth below bottom of foundation concrete and boring of hole was again started on 8th November with increased pressure at 350 psi. After boring a further 6 M, drilling was stopped as substantial water seepage was noticed at PS-3 top horizontal crown which resulted in formation of large cavity. It is to add that there was no trace of any seepage in PS-3 tunnel before this occurrence. Thereafter 108 bags of cement grouting was done. As the water loss was observed at different stretches and seepage from PS-1 and PS-3 top horizontal also increased it was decided to use high pressure air with reduced quantum of water for bailing out the drilled material. Re-drilling was again started on 18th November and drilling upto 40 M depth was achieved after injecting high pressure air (5 to 8 Kg/Cubic M) together with water. At around 48.5M – 49.5M depth no bailing out of material was seen and drill bit got jammed at 51.5 M depth due to collapse of drill hole in the upper zone. Though efforts were made to takeout the bit and drill strings but with no result. Water was injected at high pressure to open the holes of the holes of the bit, but no return of water was observed. After 4 days of this exercise the machine was shifted to new location at about 300mm away from the earlier one. While drilling at a new location too it was felt that there is a cavity formation below bottom of the foundation concrete padding and therefore after taking out the pipes about 10 cum of concrete (10mm down) was injected. Apart from this 100mm dia hole at 15 degree was also drilled and 300 bags of cement was grouted but it did not help in further drilling. The entire operation was done under the guidance of experts from Canada. Also a team of the representatives of M/s. SoletancheBachy of France who were working in Dam Site for Jet grouting works visited the site but no workable solution could be worked out and ultimately it was decided to stop further drilling with Raise Borer.

B. Excavation with Raise Chamber:-
It was decided to install Raise Climber from Power House end i.e. bottom horizontal of pressure shaft and to do excavation through conventional sinking method in upper zone of the shaft. Accordingly the Pilot Shaft excavation from Power House end was started with Alimak Raise Climmber in Pressure Shaft-3 bottom horizontal at EL 550M. During the excavation of this 2800mm dia Pilot shaft, work had to be stopped on many occasions due to frequent loose fall, which also resulted in re-fixing of rails of raise climber. The controlled blasting was resorted to and a powder factor of 2.5 Kg/cum was achieved during the excavation of Pilot shafts on an average. The Pilot Shaft in PS-3 was excavated for a height of 140 M in 140 days. Subsequently PS-2 and PS-1 Pilot shafts were also taken up in January 2003 with Raise Climber and completed in 140 days and 120days respectively. The following progress was achieved in Pilot shaft excavation.

DESCRIPTION	TOTAL	PROGRESS ACHIEVED IN A MONTH	
	In M	Average in (M)	Maximum (M)
PS-1	157	39	52
PS-2	140	35	49
PS-3	142	25	48

As the rock strata was poor, Pilot shaft excavation could not be advanced beyond the mentioned heights and it was decided to do balance excavation from Penstock erection gallery by normal sinking method using conventional drilling and blasting. Originally it was envisaged to do Pilot Shaft and Subsequent widening one by one. But as the excavation work got delayed, simultaneous working in all the three shafts was taken up. The controlled blasting was used and a powder factor of 1.15 Kg/cum was achieved in widening of the shaft. The full face excavation of shaft was done in a stage of 1 M each duly supported with shotcreting wire mesh, 3 M long anchors and installing steel rib support in the upper zone. During this excavation, problems of collapse of side wall, seepage of water etc. was also encountered which slowed down the pace of work. After the full face excavation met the earlier excavated Pilot shaft, the mucking was carried out through Audit to Pressure Shaft bottom. The excavation of PS-3 was more problematic owing to poor rock mass condition and excessive seepage. The progress achieved in the widening of 2800 mm Pilot Shaft to 6300 mm full dia is as Under:-

Description	Progress Achieved in month	
	Average	Maximum
PS-1	26 M	40 M
PS-2	28 M	33 M
PS-3	20 M	30 M

The widening of PS-1 was completed by February 2004, PS-3 by May 2004 and PS-2 by August 2004 from the Power house end. Since the excavation of shaft was considerably delay it was decided to take up steel liner erection before completion of shaft excavation in totality. The steel liner erection was started in PS-1 in March2004 and in PS-3 in June 2004 from the Power House end. Though Shaft widening was still continuing in PS-2, the steel liner erection was started in this shaft from bottom end in March 2004 by suitably co-coordinating with the civil contractor.

IV. STEEL LINER FOR PRESSURE SHAFT

The work included procurement of special quality steel plates designated as ASTM-A537 Grade-I steel and 100% ultrasonically pre-tested at manufacture workshop. The same steel was also used for backing strips for field circular joints. The thickness of steel plates used for liner varied from 16mm near Surge Shaft to 50mm in the end joining the main inlet valve (MIV) in the Power House Carven. However thrust collar provided 15 M upstream of MIV was fabricated with 63mm thick plate. Compressible bond breaking material was painted over the shell on the downstream of the thrust ring to allow face movement during surge condition steel conforming to above specification was imported from Romania.

A. Design criteria and Material Specification: -

There are two possible types of structural failure mechanisms to be considered when designing an underground embedded steel liner. The first is a tension fracture of the lining due to internal water pressure and the second is buckling of the lining in the case when the penstock is empty but subject to external water pressure. Depending on the topography, geological set up, rock cover and design principles, one of the two possible types of failure mechanism will determine the dimension and selection of material quality of the steel lining. Without exception, it is the internal pressure which is the guiding factor for the lowest part of the lining adjacent to the power.

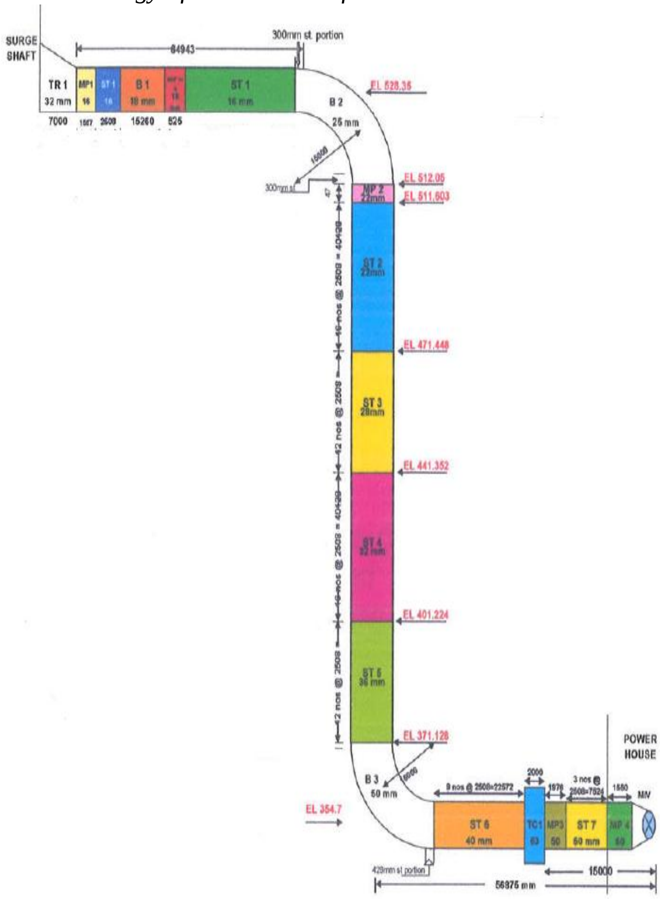
In view of poor rock condition and hence considering no rock participation this steel lining of pressure shaft was designed with the following objective :-

- The lining shall resist full internal and external water pressure and hence was designed considering following loading condition.
- Normal operating condition
  - Internal water pressure corresponding to maximum water level in the reservoir/maximum surge in the upstream surge shaft.
- Maintenance condition
  - Pressure Shaft empty i.e. no internal pressure.
  - External water pressure due to normal reservoir/natural water table.
- Transient condition
  - External water pressure due to normal reservoir level.
  - Internal water pressure due to surge and water hammer.

The erection temperature of penstock liner was taken as 25 degree. The temperature of penstock liner would later vary from 8 degree to 35 degree owing to temperature of water.

The water hammer forces due to valve closure shall be suitably transferred without unduly stressing the power house carven/structure or the surrounding rock mass. As the surrounding rock mass is weak and internal pressure is decisive it was decided to use high strength steel confirming to ASTM-A537 Grade-I steel.

B. Methodology of Fabrication of Steel Liner:-



The fabrication of steel liner was done in conformity with design specifications and corresponding approved fabrication drawing. Due to large size of ferrule dia, they were fabricated using two plates each to make 2500mm long shells. These ferrules were transported to erection site i.e. in the penstock erection gallery and were further lowered in the vertical shaft. While aligning 2500mm long shells, care was taken to ensure that longitudinal joints are not placed in one line but are staggered with each other by at least 5 times the thickness of the thickness of the thicker shell. The longitudinal welds were so located that they do not intersect the nozzles, manholes or other outlets. Various activities involved in fabrication of steel liner are detailed.

Marking, Cutting and Edge Preparation:-
The required size of steel plate were segregated and visually checked for any surface defects. Then the same were marked and cut to required size. Plate cutting was done by way of plasma arc oxy-Acetylene Gas/Fusion Cutting Process. Any material damaged in the process of cutting plates to size or forming welding grooves was removed by machining ring or grinding so as to remove all burnt material, slag and scale etc. The edges were prepared as per drawing e.g. double V groove for shop joint and single V for field joint milling on the milling machine or by flame cutting and grinding the same. However all shop joints for plate thickness upto 22mm and above and for the thrusts collar, double V groove welding was done. The angle for double V was 30 degree. These angles were checked with templates. All the bending of plates was done at contractor's workshop at Kolkata and were transported to worksite on trailers.

Stiffener Rings:-
Stiffeners were shop manufactured and welded to pipe after completion of welding of pipe segments prior to taking them to erection site. Stiffener rings were fabricated in not more than eight circumferential sections butt welded together and properly fitted and welded to the ferrules, so that the plan of rings were normal to the axis of the ferrule. The rings were welded at a distance of 1250mm center. The stiffeners were fixed in such a way that no joint of the segment came in alignment with the joint of the other stiffener rings. At least one such joint was tested ultrasonically.

Welding Process: -
Weld test were done to select the type of welding electrode and welding procedure for defined base material steel plate and to make sure that the final characteristics of the weldfront conforms to standard specification. Most of the joint welding of ferrules both for longitudinal joint as well as circular joint was carried out by automatic welding machine (SAW process inside the workshop). At first the procedure qualification tests were conducted through qualified welder to firm up the required welding process. Members being welded were brought in sufficient intimate contact at the time of welding so that ferrules will not be forced more closely together with cooling of the weld and thus setting up additional strains and distortions in the weld and parent metal. Proper quality control measures were adopted to minimize the defect in welding and educating the welders to correctly follow the mandatory provisions and approved procedures. Before commencing welding, the joint was heated to 100 degree and the inner-pass temperature was restricted to 200 degree. The joint was post-heated to 250 degree upto 2 hours after completion of welding. All shop welds, both longitudinal and circular were 100% radio-graphed. However, field joint welding at site were tested ultrasonically. All NDT was conducted as per IS: 2825, all these efforts helped in minimizing the weld defect to about 1% to 5%. All plates being welded above the thickness of 36mm and above were stress relieved. The stress relieving process took about 8-10 hours each.

Radiographic Examination:-
Radiographic examination was carried out to indicate the flaws in the welding such as pores, slag inclusion, lack of fusion and crack etc. All longitudinal joints were subjected to 100% radiographic examination. Other circumferential joints were spot radio=graphed to 10%. Also all the T-Junction between longitudinal and circumferential joint was also radio graphed upto 100mm either side longitudinal joint.
The following defects were not acceptable whenever observed in the film:

- Any kinds of cracks in zone of incomplete fusion or penetration
- Any elongated slag inclusion, which had length greater than 1/3 t where t' is the thickness of the weld.

Any defect that were noticed to be beyond the prescribed limit were arcgouged, removed and the portion was grinded to sound metal, re-welded and then radio-graphed. This process was repeated till sound weld was obtained.

C. Methodology adopted for the erection of Steel Liners: -
A steel column and beam combination were provided in the erection gallery and two Nos. of EOT Crane 15 T capacities were installed. The EOT Crane moved on a rail laid along the length of the PEG to facilitate ferrule erection and concreting work concurrently in all the three shafts. The fabricated steel liners were transported to erection site on a specially designed low bed trailor. The ferrules were loaded on the trailor keeping its axis perpendicular to the direction of movement. The ferule was unloaded by this Crane installed in the PEG and was either lowered in the shaft or was kept in the PEG as per site condition. The steel pipe was positioned with the help of jacks and turnsbuckles in the respective location and aligned accurately to the line and grade. It was checked for the flow direction, center line level, top and bottom, staggering of the longitudinal joints, etc, An Air gap of 8 mm was kept between the two ferrules while aligning the shells. Butt straps on the outside shell were used for the steel lining to facilitate making the circumferential joints in situ. The complete butt strap was arranged in two halves, the bottom half being welded in the shop to the erection section to be erected first and the other half of the butt strap being also welded in the shop to the following erection section. When adjoining sections are in their final positions the butt straps should fit closely to the adjoining shell thereby enabling a satisfactory welded joint. All in – situ welding of the shaft lining was carried out from the

inside. Before starting welding it was ascertained that the chamfered edges are in alignment and that the defects in alignment at the surface of the plates are not more than 10 % of the maximum nominal plate thickness plus 1 mm with a maximum of 4 mm for circumferential joints.

- Circumferential field joints on downstream of thrust ring were carried out without the provision of backing strip to allow the free movement of pipe.
- The joint was pre-heated by way of flexible heating coil. During welding proper inter-pass temperature was maintained and post-heating also done. Low hydrogen electrodes were used for welding.
- The roundness of the ferrule was checked regularly. The difference between the maximum and minimum diameter at any cross-section of the shell welded longitudinally was not to exceed 1 % of the nominal internal diameter subject to a maximum of D + 1250 200

Where D is internal diameter in mm

D. Ultrasonic Examination: -

All fields' welds were fully ultrasonically tested by ultrasonic flaw detector after 24 hours of welding. Cold cracking in the welds usually occurs within 24 hours of welding and therefore testing was done after this time had elapsed. Defects beyond permissible limits were rectified by gouging and re-welding and checked again before handing over for concreting.

TYPICAL CYCLE TIME FOR ERECTION OF STEEL LINERS (FOR A TYPICAL 2-5M LONG 28MM THICK AND 4.7 M DIA FERRULE)

- Lowering to position, matching and alignment 08 Hours
- Survey and support fixing 04 Hours
- Welding including pre-heating and post-heating 24 Hours
- Ultrasonic testing 24 Hours
- Concreting 24 Hours.

V. PROGRESS RATES ACHIVED

The erection work was started with bottom horizontal (55) length followed by 90degree bend. A total of 4000 MT steel was used in all the three pressure shafts. The erection of steel and thrust collar and the bottom bend took three month on an average in each pressure shaft. The erection in the vertical shaft accelerated thereafter and anaverage of 24M per month was achieved with a maximum progress of 30M (about 100 MT) in a month in one shaft. However in the top horizontal portion a maximum progress of 53.50 M (110MT) was achieved in PS-3 in June 2005 i.e. in the last month of erection work.

VI. CONCLUSIONS

The use of Raise Boring Machine should be restricted to good hard rock as high pressure water jet used for bailing out drilled material may damage the poor surrounding mass leading to collapse of hole and cavity formation beneath. The deployment of two Alimak raise climber after failure of a Raise boring machine and use of conventional sinking method for excavation of shaft from the penstock erection gallery in this poor mass zone proved efficacious.The steel liner erection work could be expedited if proper check on weld defect is exercised. Better control of welding parameters like auto temperature controls with auto-recorders for monitoring the pre and post-heat temperature and continuous training of engineers and welders is essential to reduce the weld defects and thus save on time and quality.

VII. ACKNOWLEGEMNT

The Author is thankful to NHPC and host of officers who worked in the project during that time and helped in compiling the reports.

REFERENCE

[1] ASTM-designation/Standard A537/A537M-95
[2] IS 2825-code for unfired pressure vessel
[3] ASME- Boilers and pressure vessel code sect II, part C specification for welding electrodes.
[4] Various unpublished technical reports of Teesta-V project

Biographical details of the Author:-

Brajesh Kumar Ojha "FIE", Project Director (Energy Infratech Pvt. Limited)
"Presently working in Teesta III HE Project (1200 MW) in Sikkim".

Graduated in Civil Engineering from Bhagalpur University Bihar in 1984 and joined National Hydro Electric Power Corporation (NHPC) in Feb 1985. The author has worked in NHPC, the largest Hydropower Utility in India for more than 23 years. During this period he was involved in Design and Construction of various Hydropower Projects in India. He worked as Chief Engineer in charge of power house complex Teesta Stage V (510 MW) HE Project. Healso worked as project director/executive director in Lanco Infratech Ltd for 4 years in Teesta VI HE project. Member of Tunneling Association of India, Indian National Hydropower Association and Fellow of Institution of Engineers.

