GEOTECHNICAL CHALLENGES IN CONSTRUCTING THE KALI GANDAKI "A" HYDROELECTRIC POWER PROJECT (HEPP), NEPAL

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1.0 GENERAL

Nepal's Kali Gandaki "A" HEPP is a 144 megawatt (MW), run-of-river hydroelectric plant with a daily pondage capacity of 3.1 million m^3. The project is located on the Kali Gandaki River approximately 180.0 km west of Kathmandu near the town of Tansen (see attached maps). The main features of the project are a concrete gravity diversion dam, desander, power tunnel, and powerhouse. The Government of Nepal plans to have the project on line by the third quarter of the year 2000. The basis of the project is to short-circuit a loop of approximately 45.0 km in the Kali Gandaki River by the power or headrace tunnel, about 6.0 km in length, thus developing a gross head of about 124.0 m.

The main objective of the project is to meet the demand for electric power in Nepal at least cost in an environmentally sustainable and socially acceptable manner. With its installed capacity of 144 MW, the project will generate 842 gigawatt-hours (GWh) of renewable energy annually using the flow of the Kali Gandaki River.

The project is owned by the Nepal Electricity Authority (NEA) and financed by the Asian Development Bank and Overseas Economic Corporation Fund of Japan. The consultant services for construction design and project implementation have been awarded to Morrison Knudsen International, Inc., USA. The construction contract (construction of civil works for the diversion dam, desanding facilities, headrace tunnel and power plant facilities) has been awarded to Impregilo S.p.A, Italy. Noell Stahl und Maschinenbau GmbH, Germany will furnish and install the hydraulic steelworks and Mitsui/Toshiba of Japan will furnish and install the electrical and mechanical works.

Kali Gandaki "A" is currently the largest hydroelectric power project under construction in Nepal and is estimated to cost approximately 470 million U.S. dollars.

The writer served the Kali Gandaki "A" HEPP as Chief Geologist/Geotechnical and Principal Design Engineer under a contract with Morrison Knudsen International, Inc. (MKI) for a period of seven months in 1998. During that period all major surface excavations for the main civil facilities (desander, powerhouse, and surge tank) needed to be completed. The writer was responsible for conducting the detailed geologic and geotechnical investigations for the construction design and project implementation phase of Kali Gandaki "A". This included the stability evaluation, design, instrumentation, and construction supervision of all major excavations. He directed and supervised the work of a team of native professional geologists, civil and structural engineers, surveying and drafting personnel, and a private consultant.

2.0 FACILITIES

The main civil facilities of the project are comprised of a 44.0 m-high concrete gravity dam with an adjacent spillway, comprising three 15.0 m-wide bays, to be closed by 19.0 m-high radial gates (the power intake on the left bank leads to twin desanding basins that have a combined width of 70.0 m, from which the flow will be conveyed to the headrace tunnel by a box culvert conduit); a 6.0 km-long headrace tunnel, which will be 7.4 m in diameter inside the concrete lining; and a surge shaft (28.0 m in diameter and 56.0 m deep); a steel-lined high pressure shaft and pressure tunnel; and the surface powerhouse itself.

3.00 GEOTECHNICAL CHALLENGES

Throughout the construction period the consultant and contractor have been faced with a number of challenges that continue to require the consideration of technical, financial, environmental, and social factors. A particular challenge are the variable geologic and geotechnical conditions throughout the project area, the subtropical climate, and the monsoon rains which can last several months. These required substantial redesign of cut slopes, rock support, and drainage measures to assure the stability and safety of all major excavations. Of particular concern was slope stability in both overburden soils and bedrock.

3.10 Desander Backslope

Excavation of the desander backslope (see attached photograph and drawings) was critical in order to meet the construction schedule for river diversion in November of 1998 and begin foundation excavation for the 44.0 m-high concrete gravity diversion dam.

At the location of the diversion dam and desander facility bedrock consists mainly of dolomite and graphitic phyllite. The phyllite comprises the top part of the slope and overlies the dolomite. Beneath the dolomite is a thick undulating layer of black slate which is also found in the foundation of the diversion dam. The bedrock is extensively fractured, faulted, and locally deeply weathered and therefore of low strength. Construction of the 70.0 m-wide desander basin required extensive excavation into the left bank of the river near the left abutment of the diversion dam. The upper 50.0 to 75.0 m of the 120.0 m-high cut slope are located in weathered phyllite. The phyllite was cut too steep and became unstable. After re-evaluating the stability it was decided to flatten this part of the slope from about 45 degrees to 32 degrees and install drainage galleries. The additional time required to flatten and protect the slope with fibercrete was approximately four months. In order to make diversion as scheduled in November of 1998, it was decided to widen the desander basin to only 50.0 m and leave part of the lower cut slope in place.The stability of the desander backslope is monitored with inclinometers and optical survey points that are installed on the benches. Stability analyses were carried out on both pre-and-post diversion cutslope configurations by implementing a number of computer codes such as EXAMINE2D, PHASES, SARMA, PCSTABL5, and the finite element code FLAC. The stability analyses were carried out using the most recent information available on geology and rock mass properties. Based on these observations and analyses it was decided to support the desander backslope between Elev. 575+00 and Elev. 543+00 with 24.0 m-long rock dowels installed on 2.0 meter centers. The support system is identical to that installed in the surge tank backslope which is described in detail in the ensuing section. The decision on how to complete the remaining excavation of the desander backslope, including any amount of additional rock support needed, could be deferred until after diversion.

3.20 Powerhouse and Surge Tank Excavations

The powerhouse and surge tank excavations are located approximately 45.0 km downstream from the diversion dam and desander facility. The supported cut slopes at these locations are steep and over 20.0 m high. The bedrock consists of graphitic phyllite which is fractured, folded, and locally deeply weathered. A number of steeply dipping prominent joint sets (one set parallel to the foliation) are present in the rock mass. These contribute to the formation of large wedges controlling the stability of rock excavations. At both locations, the fresh phyllite is overlain by weathered phyllite and colluvial overburden soils or landslide debris. These soils vary in thickness and composition and become very unstable when saturated with water. Maximum rainfall at the project site is in the order of 2,000 mm/year. Most of the rainfall occurs during the monsoon season which usually lasts four months starting in June and ending in September.

3.21 Remedial Measures at the Powerhouse Excavation

Slope failures in both the overburden soils and bedrock in the area of the powerhouse excavation required substantial regrading of the backslope and increased rock support. The nearly vertical cuts in the phyllite are over 23.0 m high and required additional support. Rock support was increased from 4.0 m-long, 25 mm-diameter dowels spaced at 1.5 m to 12.0 m-long, 32 mm-diameter dowels spaced at 2.0 m. All dowels function as fully grouted, untensioned rock bolts. They were installed over most of the approximately 2,800 m^2 slope face exposed by the excavation. After installation of the dowels, the face was also covered with wiremesh and 10 to 15 cm of shotcrete. To prevent water pressure buildup behind the shotcrete facing, weepholes were punched through the shotcrete at regular intervals. The same type of rock support and drainage measures were also installed in the foundation of the 22.5 m-high gabion retaining wall immediately above the powerhouse excavation. The gabion wall rests on phyllite and conglomerate and was designed to permanently support the overburden soils of the powerhouse backslope. The powerhouse excavation also includes two drainage galleries which run along the entire approximately 120.0 m-long backslope. The stability of the powerhouse excavation, gabion retaining wall, and backslope was analyzed using the above referenced software. In addition, the computer codes DIPS, GAWAK and SWEDGE were implemented to evaluate fracture data, the stability of the gabion wall itself, and the stability of rock wedges. These wedges were present in the weak graphitic phyllite rock of the powerhouse cut slopes. The excavations are monitored with inclinometers and optical survey devices. The open cut of the powerhouse is comprised of approximately 72,000 m^3 of common excavation and 57,000 m^3 of rock excavation.

All rock support installed in the powerhouse excavation is being treated as temporary support. After completion of the powerhouse structure the space between the supported excavation and building will be backfilled, and tendons will be installed to function as permanent support. The tendons will be designed to take up additional loads including backfill loads, water pressure, and earthquake loads.

3.22 Permanent RCC Cofferdam

The powerhouse structure is over 35.0 m high, about 21.0 m wide, and about 90.0 m long. It is positioned in such a way that its foundation elevation lies 25.0 m below the flood level of the Kali Gandaki River. A permanent cofferdam was therefore required to complete the powerhouse excavation and protect the facility from flooding. The construction schedule required that the permanent cofferdam be completed in three months time to divert the Kali Gandaki River and protect the powerhouse excavation from the 1998 monsoon floods. In the meantime, the excavation would proceed under protection of a temporary cofferdam, an embankment dam, that was completed in the fall of 1997.

The many advantages of roller compacted concrete, especially speed of construction, led to the selection of an RCC dam. The RCC structure has a typical gravity dam section that is 18.0 m high and 15.5 m wide at the base. However, test borings completed in early March of 1998 revealed that the rock beneath the foundation of the proposed RCC dam was about 10.0 m below that shown on the contract documents. Thus, there was only limited space to construct a cofferdam downstream of the tailrace outlet channel. The main concern in hesitating to proceed with the construction of the RCC alternative was differential settlement. Therefore, a more slender structure consisting of formed concrete anchored to the foundation was also considered. The thin anchored formed concrete alternative would be founded on rock (the number and capacity of anchors had not yet been defined) and the RCC dam would be founded partially on rock and partially on alluvium.

Following several meetings, the RCC alternative was selected. To prepare the foundation, the upper two meters of alluvium would be removed and filled with tremie concrete. The RCC structure would be built on this concrete and the adjacent rock. Provisions would be included for a longitudinal joint to accommodate some differential settlement and, if the settlement was more than desirable, provisions would be included to construct a sheet pile wall on the river side and to grout the alluvium. The RCC cofferdam with the planned foundation treatment, if needed, was the preferred method.

The permanent RCC cofferdam (the first RCC dam built in Nepal) is 18.0 m high and about 120.0 m long. The dam required placement of 26,500 m^3 of roller-compacted concrete. Placement of RCC commenced April 8, 1998 and was completed July 7, 1998 just in time to protect the powerhouse excavation from a two-year flood. During July the river discharge at the powerhouse remained at 1,200 to 2,500 m^3/second before experiencing the, so far, highest flood of 3,979 m^3/second on the 7th. This was a 2-year flood for the powerhouse site. This flood eroded the temporary cofferdam. Foundation settlements of the RCC cofferdam during the three-months construction period reached a maximum of 90 mm. No cracks were visible in the RCC at that time and seepage under and through the dam was minimal (< 10 liters/second).

Additional information on construction of the powerhouse excavation and RCC cofferdam can be found in the attached photograph and drawings.

3.23 Surge Tank Backslope and Shaft

Final design of the surge tank backslope includes a 25.0 m-high steep cut in overburden soils and weathered phyllite that is supported with soil and rock dowels up to 18.0 m in length. This *"permanent ground-nailed structure"* (face area approximately 2,000 m^2) is currently one of the largest in the world. The method was selected to prevent cutting into the toe of an old landslide which would have removed much of the natural support of the slope. In applying this method, the 28.0 m-wide surge shaft was raised 13.0 m, thereby eliminating the need for extensive excavation and treatment of the surge tank backslope, and reducing the risk of a potential large slope failure.

In the original design, the surge tank backslope was comprised of a 64.0 m-high, nearly I.0H:1.0V cut, that was to be excavated in colluvium and weathered phyllite. Prior to excavation, the overburden soils were sampled at several locations along the natural slope. The test borings penetrated the overburden soils and bedrock and clearly indicated the presence of an old landslide. The slip surface of the slide and water table followed a zone of weathered phyllite which was found at a depth between 5.0 m and 15.0 m below the surface. The slide came to rest on a bench forming a passive wedge. Preliminary stability calculations confirmed that this wedge supported the natural slope in which the surge tank backslope was to be excavated. Before making a decision in steepening the cut slope to 1.0H:3.5V max. and thereby reducing its height to 25.0 m, which required raising the surge shaft 13.0 m, adding 1,700 m^3 of concrete, additional more rigorous slope stability analyses were carried out to determine the benefits of changing the design. These analyses showed that the 25.0 m-high cut, although much steeper, had no effect on the global stability of the natural slope as most of the landslide material contained in the passive wedge would remain in place. The new design would have less quantity to be excavated and less area to be supported. About 30,000 m^3 in common excavation would be saved, thereby reducing environmental impacts.

Excavation and support of the 25.0 m high cut slope commenced mid-June, 1998 and was completed by the end of August, 1998. Common excavation was 22,500 m^3. Rock excavation amounted to 200 m^3. Soil and rock dowels consisted of 32 mm-diameter-Dywidag double corrosion protected bars (Grade 60 steel with ultimate strength of 72 ksi), 12.0 m and 18.0 m long, installed on 2.0 m-centers. Depending on the availability of the 32 mm-diameter bars, 25 mm-diameter bars were substituted and the spacing reduced to 1.75 m. Borehole diameter was 150 mm. The dowels were fully grouted and left untensioned. Three pullout tests were performed on dowels installed in overburden. These dowels took a load of about 15 tons without failure. The facing consisted of 15 cm of shotcrete reinforced with wiremesh. In the horizontal direction of the cut slope, dowel heads were supported by four sets of continuous waler bars, 16 mm in diameter. The same number and size of bars were used to support the dowel heads in the vertical direction. The vertical reinforcing bars were cut in 0.75 m sections. In addition, the dowel heads were supported by 20 mm-thick steel plates (300 mm x 300 mm in size) that were encased in shotcrete. Drainage of the slope was provided by 15.0 m-long perforated PVC-pipes (100 mm in diameter) installed on 4.5 m centers at two specified elevations and weepholes punched through the shotcrete at regular intervals.

Design of the permanent ground-nailed structure of the surge tank backslope was performed with the aid of the computer software SNAIL. The computer codes PHASES and EXAMINE2D were used to determine the stresses that were transfered from the supported cut slope to the rock mass beneath the 5.0 m-wide and 5.0 m-deep cone-shaped, reinforced concrete shaft collar. In these analyses the height of the supported cut slope was increased to 30.0 m maximum. Stability evaluation of rock wedges in the fresh phyllite below the shaft collar and design of rock support for the 28.0 m-wide surge shaft were performed with the aid of the computer codes DIPS, SWEDGE and UNWEDGE. Three inclinometers were installed to monitor the stability of the surge tank backslope and one inclinometer to monitor the stability of the shaft itself.

Additional information on design and construction of the surge tank backslope and shaft can be found in the attached photograph and drawings.

4.00 NEPAL

Nepal (population of about 18.5 million) is a land-locked country between India and China, with an area of 147,181 km^2. The main mountain ranges are the Himalayas in the north, the Mahabharat (middle mountains), and the Siwaliks in the south. Eight of the total of fourteen mountains of the world which are over 8,000 meters (26,000 feet) high are in the Nepal Himalayas.

4.10 Hydropower Development

The two basic conditions for hydroelectric power, namely, an abundance of water and great differences of altitude, are present in Nepal to an extraordinarily favorable extent. About twelve big rivers cross the country from the north to the south. These rivers all originate in the snow-covered Himalayas, and thus have abundant water reserves in the form of snow and ice. As a consequence, the water level in the rivers does not fall below a relatively high point, even during the dry season and in the winter. The principal rivers in Nepal are the Karnali, the Kali Gandaki, the Marsyangdi, the Buri Gandaki and Trisuli, the Arun, and Kosi. These rivers have a very high potential to generate hydropower, giving a total potential for the country of about 83,000 MW.

Only about 0.5 percent of the country's technically feasible hydropower potential has so far been developed. The technically feasible potential is estimated to be 45,600 MW. The country has 292 MW of installed capacity at power plants of all types, of which 250.5 MW is hydrocapacity. The three largest hydro plants in operation are: the Kulekhani I and II storage plant (92 MW, 99.67 GWh), Marsyangdi (60 MW, 101 GWh, run-of-river plant) and Trisuli/Devighat (35.1 MW, 191.8 GWh, run-of-river plant).

Of the 817 MW of hydro capacity originally planned for new development, which included Arun 3 (201 MW, 68 m gross head, 3 units) and West Seti (360 MW, 187 m cross head, 4 units), only 341.4 MW are currently committed. The committed hydropower plants include Middle Marsyangdi (61.2 MW), Upper Bhode Koshi (36 MW), Kali Gandaki (144 MW), Khimti (60 MW), Modikola (14 MW), Chilime (20 MW), and Puva Khola (6.2 MW). Of the committed plants Kali Gandaki "A", Khimti, and Upper Bhode Koshi are presently under construction.

4.20 Future Outlook

The government has formulated a policy to encourage private sector participation in the development of hydropower and delicensed hydropower plants of less than 1 MW capacity. Private sector projects presently under construction include the 36 MW Upper Bhode Koshi HEPP located approximately 100 km east of Kathmandu in the Himalayas near the Chinese border. This project will eventually be extended to include the 115 MW Lower Bhode Koshi HEPP.

Much potential has been identified for dams exceeding 200 m height, with hydro plant capacities of between 200 and 10,000 MW. Medium-sized hydropower development such as Kali Gandaki "A" is a priority to meet domestic demand, whereas larger hydropower projects such as West Seti and other projects planned on the Karnali are a priority for power export to India. Small, Mini, and Micro hydropower development (defined as plants with capacity less than 5 MW, up to 1 MW, and up to 100 kW respectively) is being encouraged to supply electricity to remote locations through an isolated system basis. There are 347 small, mini or micro plants in operation according to these definitions, with a total capacity of 15.5 MW. Funding the high costs of medium-sized and large projects, and the market, are the main constraints for future hydro development in Nepal.

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Headwaters of Kali Gandaki River
Project
XIZANG
TIBET
(BOD YUL)
HIMALAYA
Annapurna
Annapurna Sanctuary
Dhaulagiri
Manaslu
Pokhara
Bindya Vasini Temple
KATHMANDU
BHAKTAPUR
LALITPUR (PATAN)
Tansen
Narayan Temple
Butwal
Bhairahawa
Birthplace of Buddha
Nautanwa
Royal Chitwan National Park
Parsa Wildlife Reserve
Hetauda
Langtang National Park
Shivapuri Wildlife Watershed Reserve
Vantage Point
Fort, Royal Palace
Narkatiaganj
Bagaha
Bettiah
Raxaul Bazar
Birganj
Xixabangma Feng
8012
8167
8091
7937
8163
0 10 20 40 60 80 km
0 10 20 40 60 miles
1 : 1500 000
1 meter = 3,28 feet

KALI GANDAKI "A" HYDRO ELECTRIC POWER PROJECT, NEPAL

<u>Head Waters of Kali Gandaki River</u>

The Kali Gandaki River originates in the Mustang Himal near theTibetan border at an elevation of about 5,000 meters (15,000 feet) north of the Himalayan Range, the Annapurnas (see attached maps). The drainage area of that region is roughly 1,000 miles^2. However, the climate here is very dry as the region lies in the rain shadow of Himalayas. As shown in the photograph below, the river is little more than a trickle during most of the year and is easily forded on horseback or even on foot. In contrast, the project is located about 100 miles air distance to the south in the hot and humid subtropics, affected by the monsoon rains, at an elevation of about 400 meters (1,200 feet) near the town of Tansen.

Photo by H. Ueblacker, May 27, 1998

View to the south from Kagbeni near Jomsom with Kali Gandaki River and Nilgiri North (7,061 meters, 23,160 feet) of the Annapurna Range in the background.

KALI GANDAKI "A" HYDROELECTRIC POWER PROJECT, NEPAL

<u>Surge Tank Excavation</u>

Illustrated below is the permanent soil-nail structure of the surge tank backslope. The cut (1.0H:3.5V max.) in colluvial soils (landslide debris) and weathered phyllite is 25.0 m high. The support system consists of double corrosion protected soil and rock dowels 25 mm and 32 mm in diameter (Grade 60 Steel, 72 ksi UTS), 12.0 m and 18.0 m long, installed on 1.75 m and 2.0 m centers. The face of the cut slope is covered with 15 cm of shotcrete (fibercrete) reinforced with wiremesh and waler bars. The area of the slope face covered with shotcrete is approx. 2,000 m^2. Construction time was 2.5 months. Please refer to the attached drawings
(Status of Surge Shaft and Penstock - March and August, 1998; Surge Tank Excavation Plan; Surge Tank Excavation Sections; and Surge Tank Geological Cross-Section A-A) for additional details on geology, design , and construction.

Photo by H. Ueblacker, September 9, 1998


Kali Gandaki 'A' HEPP, Powerhouse Excavation and Permanent RCC Coffer Dam
H. Ueblacker, July 1998