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LAYOUT OF UNDERGROUND POWER STATIONS WITH REFERENCE TO IDIKKI UNDERGROUND STATION

THIS DISSERTATION IS SUBMITTED IN FULFILMENT OF THE REQUIREMENTS FOR THE AWARD OF THE DEGREE OF MASTER OF ENGINEERING (WATER RESOURCES DEVELOPMENT) OF THE

UNIVERSITY OF ROORKEE

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ABSTRACT

Since the last World War, more than 300 underground hydro-electric stations have been constructed by different countries in the World. An equal number is under advanced stage of planning. A review of the literature indicate that most of these stations are located underground on economic reasons added with many other inherent advantages. Underground construction results in as much as 30 to 35% savings in cost than an overground construction all other conditions remaining more or less comparable. Neither the head nor the flow limit the choice of underground location of the power station.

Economy in underground construction can be effected only if the rock strata in which the station to be located is suitable for large stable openings. If the rock mass in which a station is located is so poor that it must be considered a load rather than a building material no economy is possible by selecting an underground type. The layout should be approached with the full assurance that rock is available as a safe structural material. Geologic investigations and rock mechanic investigations are thus important in the layout of underground stations. Based on the surface geological data, only tentative layouts can be made and such layouts before finalisation have to be confirmed by exploration tunnels or adits. Considerable savings can be effected if rock mechanic investigations are correctly interpreted. With proper interpretation, a major portion of the water pressure in a steel lined pressure shaft can be transmitted to the rock while

steel lining need only act as a water sealing membrane. This will considerably reduce the project costs.

Selection of generating machinery and auxiliary equipment deserve serious consideration in an underground layout. In a layout, the largest cavern is for housing the generating machinery. The span of the power house cavity should be as small as possible to reduce the construction hazards. The basic parameter in the machinery is the diameter of runner and for this to be a minimum the highest possible speed for the runner is to be adopted. Generators are of different type. Use of the umbrella type generator reduces the size of the underground cavity. Whether it is preferable to locate the transformers underground or overground is a controversial problem. If underground, whether the transformers are to be in the main power house cavern or in a separate cavern is decided based on economic studies.

An underground layout can be finalised only after studying the various alternatives. Whether a scheme is to be designed as a 'head' development or 'tail' development or 'intermediate' development is a matter to be decided by economic studies. Merits and demerits of each alternative have to be independently analysed. In the alignment of pressure conduits, rock cover should be minimum to reduce the construction costs. Large spans for underground cavities should as far as possible be avoided. Underground power stations require additional facilities like access, ventilation, cable ducts etc. These are provided by means of tunnels and shafts.

Multiple use of these shafts and tunnels would reduce the construction costs.

The Idikki Power station as originally planned consisted of three dams and an overground power station. Further investigations and studies indicated that underground location of the station is cheaper. Consideration that led to the choice of underground location has been mainly for reasons of economy, the main factor being a substantial reduction to the quantity of steel for the penstocks as a result of the participation of the rock to withstand the hydrostatic pressure. Another major consideration was that underground location will preclude damage from landslides.

The rock conditions at the power house site and appurtenances are generally found satisfactory and based on these data tentative layout was prepared. Initially independent caverns are suggested for turbine valves, power house machinery and transformers. Before finalising this layout the main access tunnel to the power station was driven and based on the actual data at site it was considered better to accommodate all the facilities in one single cavern. Based on detailed geology at site the power station was shifted by 10°. Rock in the pressure shaft area was excellent as revealéd by driving an intermediate adit. To reduce the length of the pressure shaft, the two shafts will cross before they reach the power station. Cable shaft has been used for ventilation also. Six machines each of 130 MW (375 RPM) are proposed to be installed in the power station. The above proposals are that of the Kerala state Electricity Board.

The author has suggested some alterations in the layout. Machines having a speed of 428 RPM have been suggested to be used. This will reduce the size of the cavity. The cable shaft is suggested to be enlarged and used for taking cables, providing ventilation, and access. The cost of the underground layout can be reduced by about 20% if the above alterations are incorporated.

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CERTIFICATE

Certified that the dissertation entitled "Layout of underground power stations with reference to Idikki Underground station" which is being submitted by Shri.M. Gopalamunni in fulfilment of the requirements for the award of the degree of Master of Engineering (Water Resources development) of the University of Roorkee, is a record of the students' own work carried out by him under our supervision and guidance. The matter embodied in this dissertation has not been submitted for the award of any other degree or diploma.

This is to further certify that he has worked for a minimum period of Nine months from October 1966 for preparing this dissertation.

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CHAPTER-I

UNDERGROUND STATIONS---REVIEW OF DEVELOPMENT

1.1 HISTORICAL DEVELOPMENT:

A large number of underground hydro-electric plants had been successfully completed prior to World War II, but most of these were located in Europe. Since the War, development of Underground Stations has been noticeable in Europe, U.K., Australia, Philipines, Canada, Japan, U.S.A., U.S.S.R., and recently in India. In all, there are about 300 Underground hydro-electric stations in service or under construction in various countries, with a total installed capacity of 31 Million kw. An equal number of stations with an installed capacity of approximately 30 Million kw is under planning stage.

Increased attention towards the construction of underground power stations all over the World is indicated by the fact that only 40 plants existed before 1946. The basic reason for adopting underground plants has been economics. Many underground plants have cost less than the alternative surface layout; or with credit for the additional effective head, they produce Power at the lowest cost. Others are considered economic when due credit is given to scenic benefits, freedom from slides, avalanches or freezing or other semi-tangible benefits. Although major underground plants were built as late as the years immediately preceding World War II, similar projects were planned considerably earlier. This indicates that the idea of locating power stations underground was not suggested by the intention of protecting them against air raids, but by technical and economical considerations.

The following tabulation gives ani approximate distribution of the Underground plants in various countries:-

Algeria	-	2	Italy	-	72
Australia	-	9	Japan	-	5
Austria	-	2	Mexico	-	4
Brazil	-	4	Norway	-	53
Canada	-	3	Peru	-	1
China (Formosa)	-	3	Phillipines	-	1
Cuba	-	1	Portugal	-	5
Egypt	-	1	Rhodesia	-	2
El Salvador	-	1	Scottland	-	5
Finland	-	2	Spain	2	2
France	-	23	Sweden	-	44
Germany	-	3	Switzerland	-	28
Guinea	-	2	United States		4
Iceland	-	1	U.S.S.R.	-	4
India	-	5	Yugoslavia	-	12

Italy leads the World in number of Underground plants despite poor geological formations of sedimentary and metamorphic rocks with heavy seepage problems. By contrast with Italy, geological formations are generally

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good in Norway and Sweden. The total underground station capacity in Italy would be 5 Million kw (30% of the total), about 4 Million kw in Sweden (50% of the total) and 3 Million kw in Norway and 2.5 Million kw in Canada and nearly a Million kw in Switzerland.

Underground layout of hydrostations is more popular in European countries perhaps due to the advancement made in tunnel work and economical excavation processes. Underground layouts require more labour and less material. Labour is relatively more important cost item in the United States. However advances in underground excavation techniques, equipment and knowledge are tending to increase the economic use of underground excavation in that country too.

1.2 ELEMENTS OF UNDERGROUND STATIONS:

The principal electrical machinery such as turbines and generators, transformers and switching equipment and ancillary equipment; station service and operating machinery are all the same whether the proposed station is underground or on the surface. Underground plants comprise all the elements found in the ordinary types of developments with power houses on the surface. These elements are:

1) Storage and diversion facility: This is obtained by the construction of dams or weirs or barrages.

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ii) Intake Works: In low to medium head developments, mostly the intake is combined with the main dam constituting the diversion and storage works and ordinarily the concentration of head in the reach of river developed. In high head schemes, the intake works are normally separate from the main dam. Intake works ordinarily consist of trash racks and control gates together with operating facilities.

iii) Conductor system: In low head plants, the pressure conduit will be relatively short, sloping or vertical and these may be lined or unlined as dictated by rock conditions. In high head developments, the pressure conduit may be a combination of a pressure tunnel and a sloping or vertical penstock embedded in rock with concrete. In some cases, surge tanks would be necessary at the commencement of the pressure shaft or tailrace tunnel or at both the locations.

iv) Power House Cavity: The cavity accommodating the generating units is excavated in the rock. Sometimes provision is made for housing the step-up transformers adjacent to the generators in part of the same chamber or in a separate chamber; otherwise the transformers are placed on the surface. Control valves in the branches of the penstock may either be located within the power house cavity or in a separate chamber adjacent to the power house cavity with its own drainage connection to the tailrace tunnel.

v) Tailrace Tunnel: The discharge from the water wheels is handled through a pressure or free flow tunnel. For stable hydraulic operation, a surge chamber adjacent to the power house would be required if the tailrace is designed as a pressure tunnel.

1.3 FACTORS FAVOURING UNDERGROUND ARRANGEMENT:

Considerations for underground design can be summarised under four categories:

i) Economical aspects;

ii) Operation aspects;

iii) Nature conservation and land protection; and

iv) Defence requirements.

These are discussed in detail below:

1.3.1 Economic considerations: An analysis of the costs of underground stations shows that costs are 30 to 40% less than for conventional overground stations. Of course, this may not be true in all cases. However, in nearly every case, the main reason for the decision to construct an underground station has been that it was more an economic proposition than one aboveground. The economics can be divided up as follows:-

i) If the steep canyon profile at the site of a high dam is deep and narrow with steeply rising sides, there may not be enough space for locating the station overground at the foot of the dam. (Fig.66) Deep cutting for locating the station overground may affect the stability of the slope. In such cases underground location may prove to be economical since there is more freedom in positioning the station. In low to moderate head plants, Underground arrangement may permit a more effective concentration of head and thus the utilisation of larger generating units with lower specific costs.

ii) In many cases, the cost of structures may be almost equal to the surface installation. But the real savings occur in the reduction in frictional losses, due to shorter water conductor system (Example: Idikki)

iii) With suitable topographical and geological conditions the conduit for water conveyance between the headworks and the underground station will be much shorter than the headrace tunnel and exposed penstock in an overground development. This will reflect in the cost of underground layout. Costly anchors can be omitted in underground penstock shafts. Capacities of pipe lines can be increased which will be well neigh impossible, because of unfavourable slopes and bad ground in the case of exposed penstocks.

iv) Tunnels in rock may be lined or unlined depending upon the quality of rock. Rock of good quality can be relied upon to carry full or part of the internal pressure. The construction of an underground conduit instead of an exposed penstock may therefore result in savings in steel lining. In good quality rock unlined conduits have been constructed even for a hydrostatic pressure as high as 300m. In some cases, the steel lining is omitted entirely

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and a thin concrete layer is provided to prevent seepage losses and reduce friction.

v) When the conditions are such as rockfill dam is the most feasible and economical type, the best overall solution is frequently that in which rock removed from the other elements of the power wole development such as the power house chamber, is used for the construction of the dam. This will entail economy in overall construction cost of the Underground development if the power house is close to the dam.

vi) If the dam structure across the water course is an earth dam or earth and rockfill structure, diversion of river water during construction of dam will be through tunnels constructed in the abutting hills. These tunnels may perhaps be utilised as tailrace of an underground power station with advantage of incidental economy. The available head may considerably be increased for the development, may be with a slight increase in the length of the tailrace tunnel.

vii) With reduced length of pressure conduit, smaller surge tank dimensions are found sufficient in Underground developments.

viii) The ratio of pressure conduit to the tailrace tunnel is of great significance. Overall costs of the conduit system are the lower; the greater the relative length of the tailrace tunnel since the latter can be left unlined or provided with a plain concrete lining to reduce friction loss.

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In the case of Underground layouts the power station can be located close to the intake and having long tailwater tunnel. Such arrangement may prove to be economical than the surface alternative in overall costs.

ix) Compared to the penstock of the equivalent surface arrangement, the length of the pressure shaft of Underground station would be very short and sometimes the synchronous relief valves can be omitted without affecting the governing conditions.

x) Foundation difficulties can be overcome, for example many Swiss valleys and Himalayan valleys have steep sides and are filled to a considerable depth with glacial drift. A station in the open valley therefore requires deep and costly foundations.

xi) The underground excavation works can be carried out round the year without being hampered by seasonal variations as the work is free from the effect of wind, snow and rain. This helps in great deal in achieving the object earlier, than it would have been, had it been a surface station. Thus, not only is the overall period of construction reduced, but the project begins to yield positive returns earlier. Further, the construction of the dam and the power station works do not overlap as they constitute two absolutely independent units.

xii) An underground power station can be most economically constructed when the site provides sound rock.

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Modern tunnelling technique may make such a solution quite as economic as a surface type power station, depending on site topographical and geological features.

xiii) Property to be acquired for underground plants is far less.

xiv) Underground arrangement possesses greater freedom than a conventional one from damage in an area subject to frequent and heavy earthquake shocks since intensity of shakin below ground is less severe than on the surface.

Underground arrangement is not without negative traits as far as costs are concerned. These considerations are given as under:

i) For housing the generating machinery and other equipments, cavities are to be constructed underground which are expensive. However, in many cases, only a part of this cost can be accounted as increment over the outlay necessary for equivalent surface arrangement since the foundation costs of overground stations located in poor quality rock become excessive. Costs of the interior structure of the machine hal are materially lower than those of the superstructure of a surface plant of similar dimensions.

ii) For access to the Underground stations, tunnels or shafts are necessary. These may increase construction costs of underground layouts.

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iii) Cost of separate cavity excavated for housing inlet valves and transformers may have to be regarded as excess cost.

iv) Extra expenditure will have to be incurred for ventilation and air conditioning and cable galleries.

v) In some underground layouts a surge tank would be required downstream of the station which would also increase the cost of the underground layout.

vi) It is necessary to protect the generators against the damage by water discharged from ruptured penstock located underground. Arrangements for pumping the flood waters or construction of a separate tunnel to lead the waters to the tailrace tunnel would become necessary. This increases the cost of Underground layouts.

1.3.2 OPERATIONAL ASPECTS:

Operational assets of the Underground layouts could be summarised below:-

i) Shorter conduit length results in smaller head losses. The amount of energy thus regained annually can be considered an asset over the comparative surface arrangement

ii) Maintenance and repair costs are lesser for Underground stations. The annual cost is smaller in Underground arrangement. Hardly any painting or other maintenance is necessary for the pressure shaft and the life of the station is almost infinite. Therefore a much lower depreciation (0.5% in Sweden) can be taken. If the capital cost of the two types of stations are the same, the reduced annual costs make the underground station a more economical proposition.

Operational liabilities in a station located below ground are (i) incremental cost of lighting (ii) Cost of ventilation and air conditioning and (iii) disposal of seepage water.

1.3.3 NATURE CONSERVATION & LAND PROTECTION:

In areas where there is wish to retain the natural unspoilt beauty of the country-side, an underground station has the advantage of being entirely out of sight. Aesthetical, recreational and tourist interest alike require the conservation of the original landscape features and the protection of these against spoiling by exposed penstocks, power houses etc. In West European countries, preservation of scenery is a major consideration and technical reports on water power developments include a detailed account on the measures envisaged for landscape protection besides technical considerations and economical estimates in support of the project.

1.3.4 DEFENCE REQUIREMENTS:

The most obvious advantage of an underground station is its immunity from air attack. Though greater security is achieved against air attack, the real reason for locating the station underground has been economics. Virtually all of the approximately 31 Million kw installed capacity in the various underground hydro-electric projects of the World have been developed purely on economic and technical grounds. In sweden, which is militarily exposed, undergroun stations are built on economic grounds with added advantage of wartime safety.

As in any project, a very careful evaluation of the various alternative solutions is necessary if the full economics inherent in an underground design are to be realised.

1.4 SCOPE OF UNDERGROUND STATIONS:

Underground power plants may be storage or run-of-the-river type depending upon the topography. Some plants are designed to operate as base load stations while some are peak load stations. A selected few are pumped storage schemes. These have been constructed in all types of geological formation comprising igneous, metamorphic and sedimentary rocks. Most of the stations in Sweden are built in hard rocks such as granite requiring practically no support at all. A few have been constructed in badly fissured and disintegrated rock strata (in Brazil, Peru and Australia) for which continuous roof support was required.

Where the underground Plant necessitates geological and economic requirements, neither topography nor climate affect their use. Plants have been constructed in Arctic region, examples being Porjus and Harspranget developments in Sweden and the Niva development in Russia. Plants have been constructed in Algeria, Brazil, Peru, El Salvador, which are hot countries.

Economic consideration of Underground plants vary depending on the hydraulic head, capacity of the plant, load factor and availability of water. Hydraulic head in Underground plants vary from 36 ft. (Norrfors in sweden) to 4822' (Forges in France). Design flow ranges from as low as 15 cusecs at Force Ponale in Italy to a maximum of 14200 cusecs at Harspranget in sweden. The number of generating units runs from two in several plants to an ultimate 16 in Kemano. Unit capacity varies from 350kw to 185MW (Mont cenis plant, France). A low load factor of even 20% has been adopted in certain stations. From the above it may be seen that neither head nor flow nor size of generating unit limit development of water power resources by means of underground plants.

The construction of underground power stations is governed all over the World primarily by consideration of economics. First costs of the development may in many cases be reduced materially by the underground location of the power house. These considerations are justified in recent times by the rapid improvement in tunnelling techniques and methods and one can conclude that Underground development will prove to be more and more economical in the years to come.

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CHAPTER-II

CLASSIFICATION OF UNDERGROUND STATIONS.

2.1 TOPOGRAPHICAL CLASSIFICATION:

Among the various layouts adopted for hydro-electric projects with an underground powerhouse, three are the typical ones to which others can be referred. (49, 83). The location of the power station may be virtually at any point on the power water way (tunnels, penstocks, tailrace) and this marks the type of development with underground powerhouse. With the site of headworks established, the underground power station can be sited:

a) near the intake and such a development is popularly known as "head development";

b) near the return point and such a type is known as "tail development"; and

c) midway between intake and return point and this type known as "intermediate" development.

2.1.1 HEAD DEVELOPMENT: (Fig.2a)

In a "head" development, the power station is located close to the headworks and water is directly fed from the head pond into the power station and discharged through a long tail-water tunnel. Being close to the head pond, upstream surge tank in the power water way may be eliminated. However a collection chamber or surge chamber might be required just downstream of the powerstation to accommodate the discharge at full load. In "head" developments, though the length of the pressure conduits is minimised, access to the station and

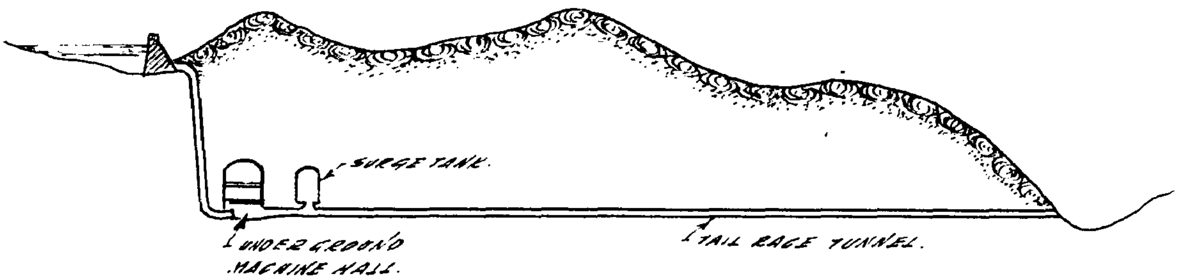


FIG: 2(a) HEADRACE ARRANGEMENT.

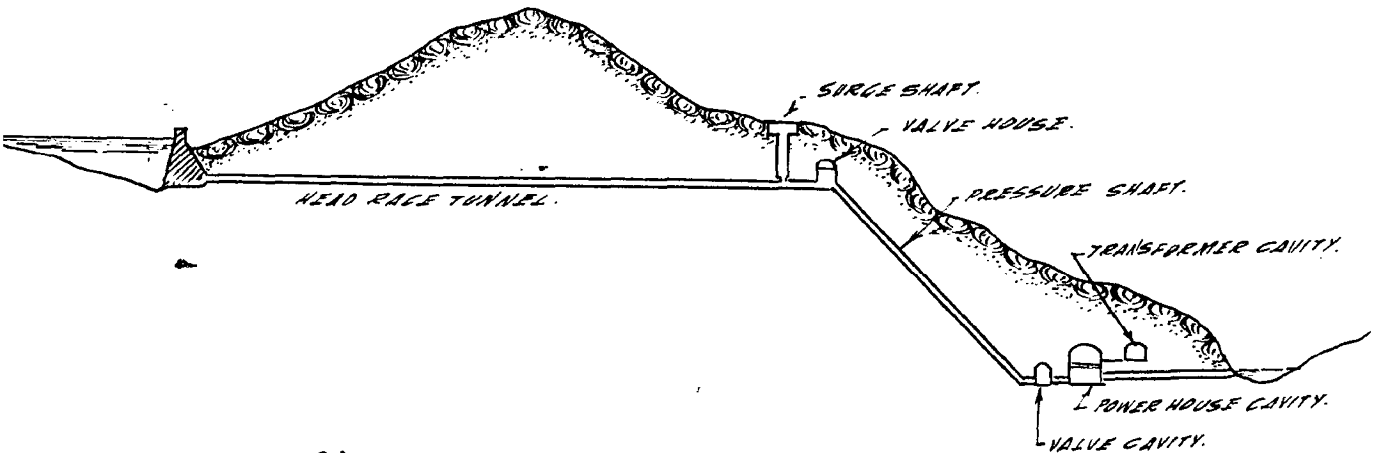


FIG: 2(b) TAIL RACE ARRANGEMENT.

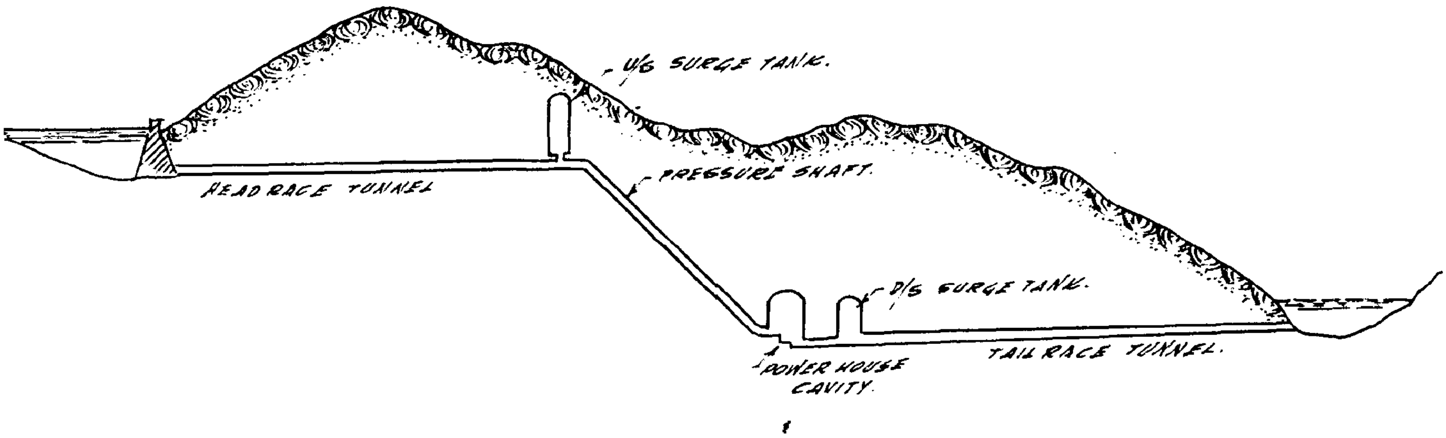


FIG: 2(c) INTERMEDIATE ARRANGEMENT.

long electrical connections present problems for heads in excess of three or four hundred feet. If the underground station is located adjacent to the reservoir as is the case with 'head' development, seepage can be expected during construction and maintenance, if the rock strata is not impervious.

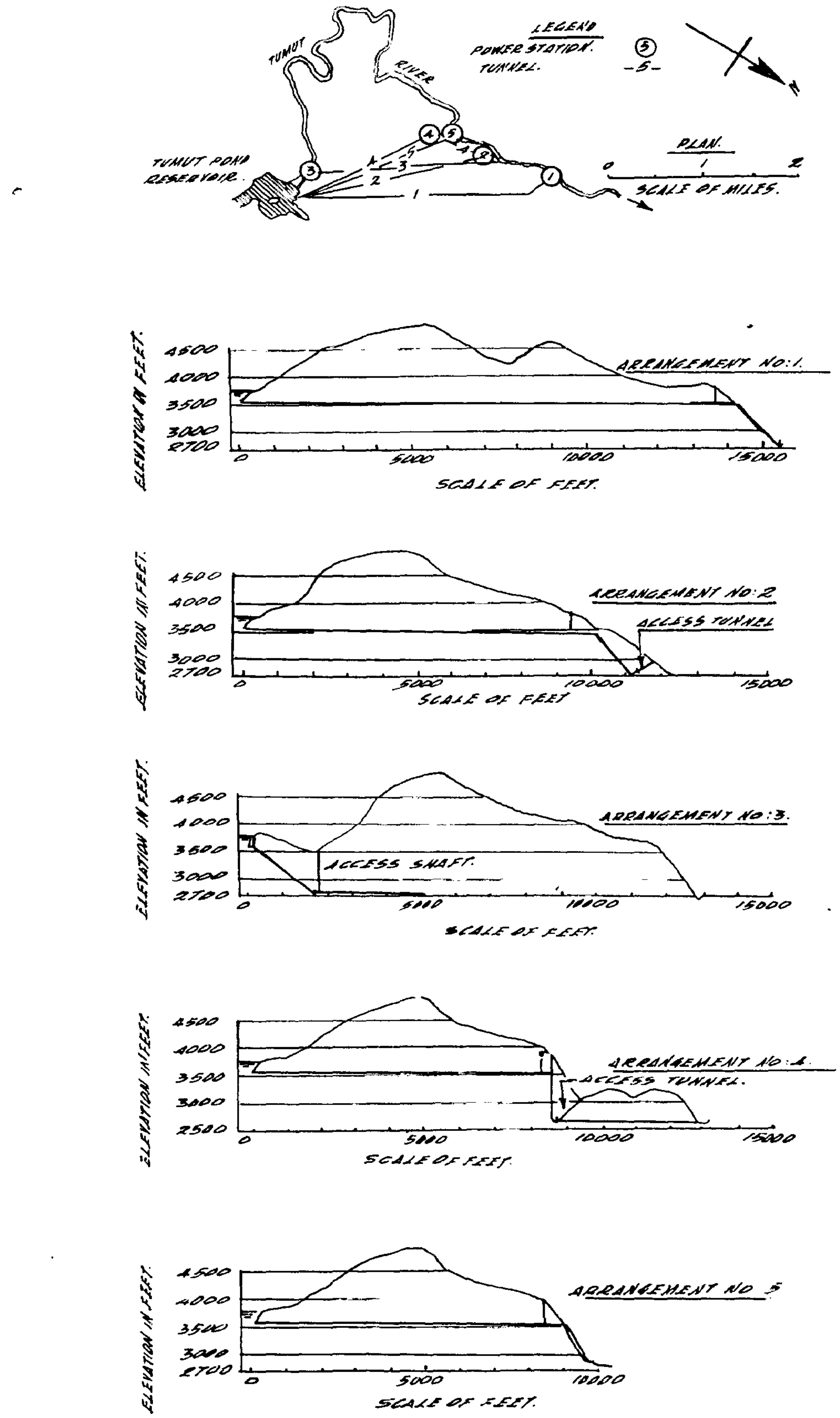
'Head' development is often referred to as the "Swedish type" as this arrangement is frequently adopted for the typical low head - high flow projects of that country.

2.1.2 <u>TAIL DEVELOPMENT: (Fig.2b)</u>

In a 'tail' development, the power station is located near the return point. The length of the tailrace tunnel will be much shorter than the headrace tunnel. High head plants are usually 'tail' developments. This type is almost similar to the conventional hydro-power development with overground station. When comparing the two possible alternatives with underground or surface power house, in favour of the former, there will be chiefly the savings obtainable in penstock while in favour of the latter there is the unquestionably lower cost of the machine hall and access. The choice of the alternative is determined only on economic considerations.

2.1.3 <u>INTER MEDIATE DEVELOPMENT:</u> (Fig.2c)

The Power station is located halfway between the intake and return point along the power water way. Surge tanks at upstream and downstream of the power station might be required in the power water way.



SECTIONS.

FIG: 2(d) TUMUT 1 PROJECT - ALTERNATIVE PROPOSALS.

In a scheme layout the choice between a 'head' development or 'tail' development or 'intermediates' development is dependent on topography, geology and economics. In Tumut[1] (Australia) Project, (Fig.2d) both 'head' development (Alternative-3) and tail development (Alternatives 2 & 4) were examined in detail (43a). 'Head' development involved 1000 ft. deep access shafts which would have incurred high costs and a long construction period and investigations then concentrated on the adopted layout (Alternative-4) with vertical pressure shafts, the station 1000 ft. underground and headwater and tailwater tunnels of 8200 ft. and 4500 ft. length respectively. Detailed investigations confirmed the suitability of the layout as per alternative-4 and the same was adopted.

2.2 HYDRODYNAMICAL CLASSIFICATION: (Fig.2e)

Underground power stations can be classified based on the hydrodynamic characteristics of different developments, especially with regard to governing conditions. There are five categories (31) in this classification:

i) The tailrace tunnel is designed as a free flow tunnel without a surge tank downstream (HUINCO STATION-Fig.6m)

ii) The tailrace is designed as a pressure tunnel with a conventional surge tank located just downstream of the power station and there will be no surge tank on the upstream of the station (GLENMORISTON STATION - Fig.6c)

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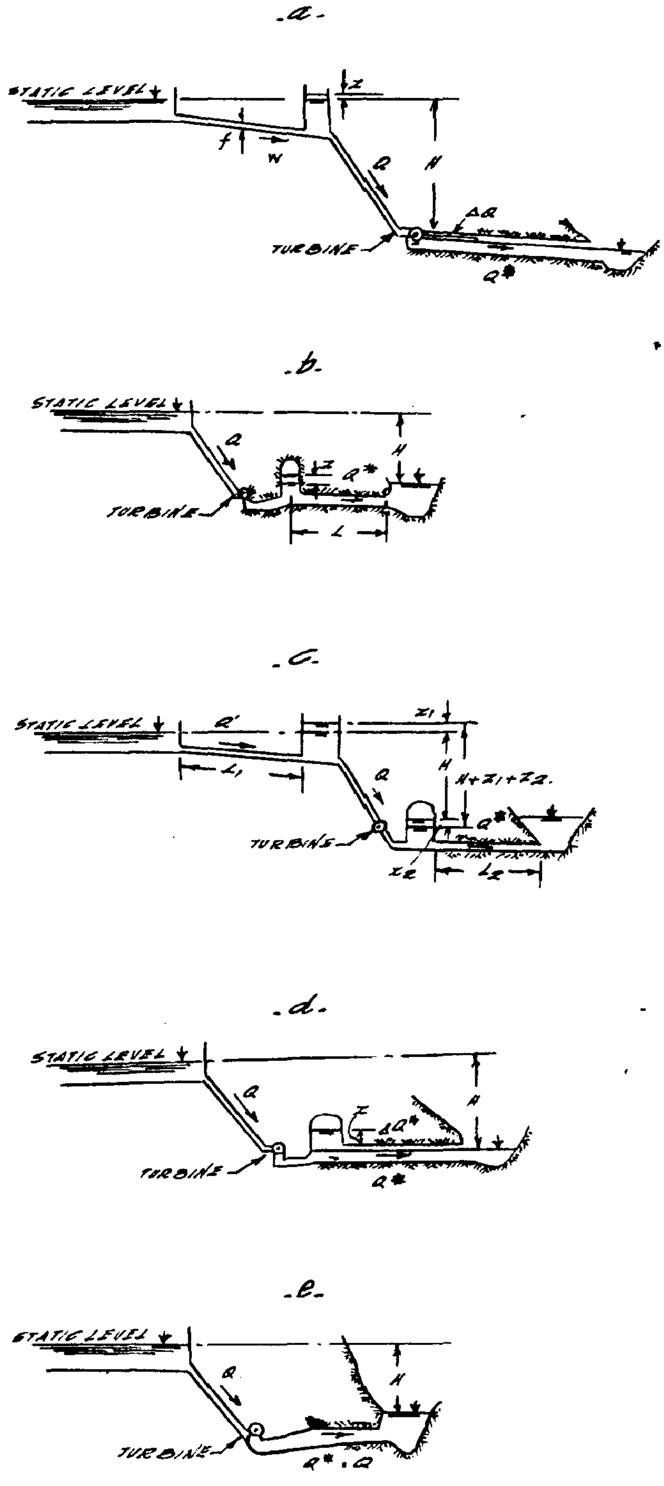


)FIVE PRINCIPAL HYDRO DYNAMIC TYPES OF UNDERGROUND HYDRO ELECTRIC POWER STATION.

iii) The whole waterway is under pressure with surge tanks at upstream and downstream of the power station. (Ex.CHUTEDEs - PASSEs STATION - Fig.6f).

iv) The tailrace tunnel acts as a free flow tunnel during normal steady conditions and pressure flow exists during major disturbance in the flow conditions.

v) The whole power waterway is very short without any surge tanks.

The above cases are dealt in detail below:

CATEGORY-1: Free level tailrace tunnel without downstream surge tank:

This type of arrangement is adopted usually for high head plants. However, many examples with reaction turbines can also be mentioned. In this type of layout, an upstream surge tank is usually provided in the power waterway which however does not interfere with the flow in the tailrace tunnel The tailrace tunnel will be large and steep enough to conduct any discharge from the power station without putting the tunnel under pressure, both for steady and unsteady flow conditions.

An example of this type with Pelton turbines is the Innertkirchen power station. The gross head is 672.3m. length of pressure tunnel being 10,000m and tailrace tunnel 1400m with 0.25% slope. When Pelton turbines are used, the calculation of the water level just downstream of the turbines

contd...

is difficult. A theory analogus to the theory of lateral spillway is developed for this purpose and the same is used for Innertkirchen. Several Schemes have been developed on the same lines as Innertkirchen.

Krangede Power station with 8 Francis turbines (60 m head) is of the same type. Downstream of the power station there is an air shaft, but this air shaft does not work as a surge shaft, the flow in the tunnel remaining free flow.

The dimensions and levels of the power station and tailwater tunnel under category 1, are to be suitably fixed so as to avoid any possible flooding of the generator floor by a penstock burst or a turbine valve rupture. The flood water due to a penstock or valve rupture is calculated in two ways (i) by assuming correct closing of the penstock valve located at the bottom of the surge tank in the power water way and water draining into the station from the penstock below the penstock valve; (ii) by assuming that the penstock valve and turbine valve in the power waterway do not function in an emergency case, causing the entire flow from the intake draining into the power station.

It is considered quite necessary to make extensive studies and calculations on all normal and abnormal operating conditions and possible failures to avoid any accident similar to the flooding of the Lac Noir Power station in the Vosges (France) where several engineers were drowned in the power station after a penstock rupture and failure of the emergency valves.

In the case of high head stations with free flow tailrace tunnel, the design and construction of upstream surge tank in the power waterway may not present problems. However, in the case of small head stations the design of a proper surge tank for stability conditions is difficult or perhaps construction too expensive due to topographical or geological conditions and in such cases an alternative is to design the station with a free flow tailrace tunnel but without a surge tank.

<u>CATEGORY-II.</u> Tailrace Pressure Tunnel with Normal Working Downstream Surge Tank:

In this case there is no surge tank upstream of the pressure shaft; but the flow in the tailrace tunnel will be under pressure and there will be a surge tank downstream of the power station. With a reservoir having wide fluctuations in level, the solution with upstream pressure tunnel and surge tank is expensive because of the high pressure in the tunnel and height of the surge shaft. In some cases the topography may not permit a headrace surge tank. In such cases the solution with downstream surge tank is often adopted. The theory of surges in downstream surge tank is similar to the classical surge tank theory, some algebric signs being reversed. The limiting conditions of Thoma for stable governing also applies to this type of surge tank as for an ordinary surge tank. The turbines relating to such development will be of the reaction type. The entrance of the pressure tunnel has to be fixed in such a way that no air enters the tunnel when the level in the surge tank is oscillating.

Power stations viz. Harspranget and Hjalta in Sweden and Glen Moriston Station in Scotland are of the type described above.

<u>CATEGORY-III</u> - Systems with Headrace & Tailrace Surge Tanks:

Under the above type, the turbines will be of reaction type. If both the headrace and tailrace tunnels are long, surge tanks are generally required both at upstream and downstream of the power station, when the tailrace tunnel is designed as a pressure tunnel. An alternative is to design the tailrace tunnel as a free flow tunnel without a downstream surge tank. This alternative would prove to be economical in the case of developments with short tailrace tunnels.

With headrace and tailrace tunnels under pressure, the oscillations in both upstream and downstream surge tanks would interfere, so that the problem of dampening the oscillation becomes important. To avoid instability of oscillation in such a system, the minimum areas for both the headrace and the tailrace surge tanks are multiplied by a factor which in some cases is more than 2 or 3.

Examples of the type of development are Yamuna Stage II scheme with Chibbro Power Station, India; Santa Giustina Station in Italy.

<u>CATEGORY-IV</u> - Tailrace Tunnel with partial working Downstream surge tank:

A "Partial working" surge tank with a tailrace tunnel would prove to be cheaper when there is wide fluctuations in the tailrace channel. Alternative to this would be to construct either (i) a surge tank large enough to meet the requirements of the Thoma formula or (ii) a free level tunnel able to absorb any translatory wave. These arrangements may not be cheaper than the alternative with "Partial working" surge tank. In this alternative, the tunnel is constructed large enough to allow steady flow of the maximum turbine discharge during high water conditions at exit end of tailrace, but unsteady flow conditions produced by the opening of the turbines would put the tunnel under pressure by the formation of positive waves. To meet these unsteady flow conditions, a surge tank is introduced in which the water could rise and afterwards fall. By the fall of the water level in the surge tank, back to steady level, the tunnel becomes a free level tunnel again and the surge tank is put out of operation, the water no longer rising. Under such conditions it is obvious that the condition of Thoma does not apply to a "Partial working" surge tank. To avoid any damage to the generators, the water in the surge tank must never rise enough to reach the generator floor, so giving a clear limiting condition for the problem.

In some cases the tunnels have to operate momentarily fully under pressure, if flood conditions are ruling at downstream end. In these cases the area of the surge tank has to be somewhat larger than limiting value given by

Thoma.

CATEGORY-V - systems fully under pressure with no surge tanks:

This alternative is considered for designs with very short pressure pipes and galleries, the example being a underground power station by passing a dam. This arrangement having no surge tanks, there are no surges and no surge instability. Unsteady flow conditions will cause pressure waves of the elastic type or water hammer to be produced in the pipes. The problem will be to estimate the pressure rise and the pressure drop caused by water hammer, and to check the conditions for turbine design and turbine regulation.

The advantages or disadvantages of any solution have to be discussed in relation to the scheme under consideration. The designer has to choose between, reaction or impulse wheel, vertical or horizontal shaft and economic studies have to be conducted before deciding on the type of layout.

CHAPTER-III

GEOLOGICAL FACTORS IN CHOOSING UNDERGROUND SITES FOR POWER STATIONS AND TUNNELS

3.1 GENERAL:

The previous section has dealt with the general arrangement of underground developments as affected by topographic and hydraulic considerations. In underground hydroelectric projects shafts and tunnels which form access and waterways and large cavities in rock for housing generating machinery and other equipments are prominent features. For the location of the station and appurtenances underground, a suitable geological conformation is an obvious necessity. Actual location of the conduits and the dimensions of the machine hall are greatly influenced by the soil characteristics, rock types, their condition and behaviour as well as ground water conditions of the site. Therefore it is of paramount importance to carry out all exploratory work and geological information about the physical characters, structural features and in-situ behaviour of the rocks as well as ground water conditions before construction commences. The importance of a detailed geological map of an underground layout cannot be overemphasised. Such a map not only should show the geologic structures and rocks encountered, but it also should serve as a lasting record to indicate the location of bad sections, which may require attention many years after the work is completed. The wisdom of undertaking a comprehensive geological study was proven (44) at the Chute-des-passes Project (Canada) where information obtained from geological investigation showed that it would be necessary to change the orientation and position of the Power house and draft tube manifold quite radically from the position and orientation selected for preliminary studies of the development (Fig.3a)

3.2 Classification of Rocks

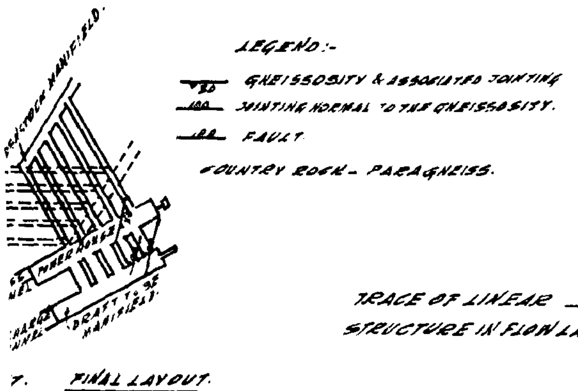
Rocks can be classified as (1) Consolidated rock and (2) unconsolidated rock. The physical characteristics are described below:

3.2.1 Consolidated Rocks:

Consolidated rocks are those in which the constituent particles or fragments are bound together strongly to produce an aggregate with moderate to considerable strength. Some consolidated rocks are massive or isotropic and the physical characteristics of these are same in all directions within the rock. There is no preferred orientation of the mineral particles and the ease of tunnelling in such rock is the same irrespective of the direction of the conduit. Igneous rocks such as certain types of granite, sedimentary rocks such as certain limestones, and metamorphic rocks such as certain marbles and quartzites come under this category.

Most rocks possess directional structures as a result of the parallel to sub parallel orientation of many or most of the mineral particles. Such rocks are anisotropic and show distinct variations in physical properties in different directions. Consideration of the nature of the anisotrophy of rocks is important in tunnelling as it has a direct bearing on the cost of driving the tunnel.

In igneous rocks layered or linear structures are commonly present. (Fig.3b) If the rock contains "platy" minerals such as tabular feldspar or mica, there is a tendency for these minerals to rotate into parallel or sub parallel positions during the intrusion and crystallisation of the rock. The resulting formation is called "flow layering" or "platy structure". Rocks having minerals of prismatic



7. FINAL LAYOUT.

800

IN FEET.

PLAN.

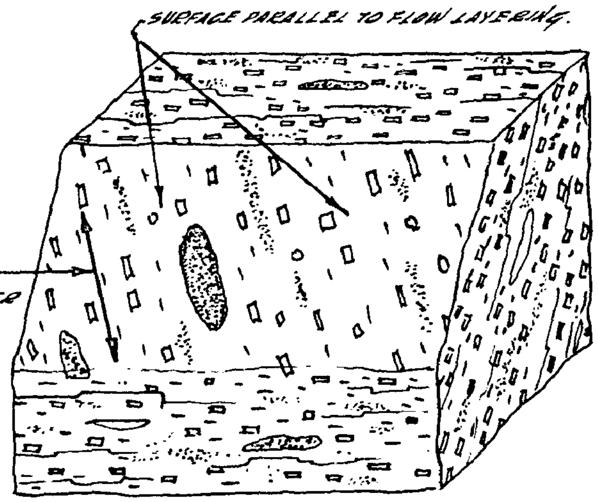
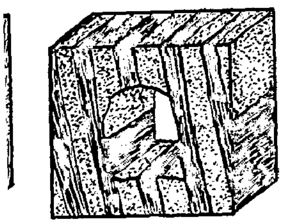


FIG: 3(b) BLOCK DIAGRAM SHOWING FLOW LAYERING & LINEAR STRUCTURE IN AN IGNEOUS ROCK, STIPPLED AREAS ARE INCLUSIONS OF FOREIGN ROCK.

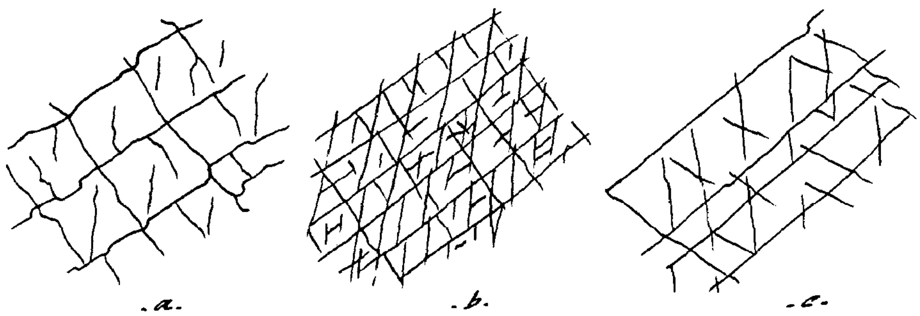


.b.

F. PARALLEL TO STRIKE.

MS SHOWING A TUNNEL IN

G FOLIATED ROCKS.



.a. .b. .c.

.a. ROUGH AND IRREGULAR, MINIMUM CAVING.

.b. SMOOTH SURFACED, MAXIMUM CAVING.

.c. SMOOTH AND ROUGH, CONSIDERABLE CAVING.

FIG. 3(d) TYPICAL JOINT PATTERNS, JOINT SURFACES.

or needle like habit tend to develop a unidirectional formation which is described as "linear structure" or "flow ~~layering~~ lineation". It is not uncommon to find both platy and linear structures in the same rock. Where flow lineation and flow layering are steeply inclined, lineation is of slight practical importance in the tunnel operation. However, the direction of the tunnel with respect to the flow layering merits careful consideration. For a tunnel driven across the flow layering, the face will advance relatively rapidly because in the blasting operation the rocks will break cleanly and easily away from the planes of weakness parallelling the flow layering. The walls, floor and roof tend to be rough and blocky. If the tunnel is parallel to the strike of steeply dipping flow layering the forward progress of the tunnel will be slowed because the tunnel is parallel to a direction of maximum strength and toughness in the rock. The walls of the tunnel will be relatively smooth; the floor and roof more or less irregular.

Layering in metamorphic rocks is called "foliation" and it is well developed in gneisses and schists. Foliated metamorphic rocks tend to split most easily along planes of parallel to the foliation. In foliated rocks the effect on the progress of tunnelling operation is more pronounced than the effect of layering in igneous rocks. A tunnel at right angles to the strike of the foliation in general is constructed much more rapidly than is one following the foliation, although the tunnel is likely to be much more blocky and irregular. A tunnel following the strike of steeply dipping foliated rocks tends to cavein from the roof. The tendency of the walls to cave increases as the dip of the foliation decreases.

Typical conditions are illustrated in Fig.3c which shows one tunnel cutting across steeply dipping foliated rocks, and another following the foliation of steeply inclined layers. The latter condition should be avoided if possible.

If a tunnel is driven through inclined layered rocks at right angles to their strike an additional factor should be considered i.e., the direction in which the tunnel is driven relative to the dip of the layers. The tunnel should be driven so that the layers will dip away from the heading so that blasting operations can be expedited because the cleavage in the rocks tends to produce an overhanging face rather than one which is sloping back toward the already completed section of the tunnel.

3.2.2 Unconsolidated rocks:

The range of composition, texture and structure in unconsolidated rocks is extreme. Texturally these rocks grade from the finest muds to coarse aggregate such as are found in the rock slides and bouldery glacial deposits. Unconsolidated rocks are characterised by the lack of interlocking contacts between adjacent grains or by the lack of a strong, binding cement or matrix. Some rocks, eg. some sandstones, have been converted into loose aggregates of mineral grains through the removal of the cement by dissolution.

In tunnel operations unconsolidated rocks generally are encountered at the portals, where the bed rock is blanketed with rock waste and alluvium. The difficulty of tunnelling through unconsolidated rocks consisting partly or largely of very fine grained or clayey materials increases in direct proportion to the amount of water present. Dry or slightly moist clayey aggregates stand fairly well in portal areas; and despite a slumping tendency, the walls and roof generally will stand long enough to permit support and lining to be set before caving starts.

3.3 FOLDS, JOINTS & FAULTS:

Defects in rocks such as folding, joining, faulting etc., tend to control the pattern of breakage, reduce stability of rocks and safety in tunnel excavation.

Rocks may be folded as tightly as pleats on a closed accordian or gently as waves on a balmy bay. The type of folding influences the excavation method (Fig.3e). Folding in stratified rocks require correct interpretation in order to predict the attitude of layers in different sectionf of the tunnel.

A tunnel through a bedded or schistose rock should be oriented in the direction providing maximum stability, ie., with the long axis at an obtuse or 90° angle to the strike of beds (Fig.3f). The same criterion can be used in orienting with respect to joint or fissure systems.

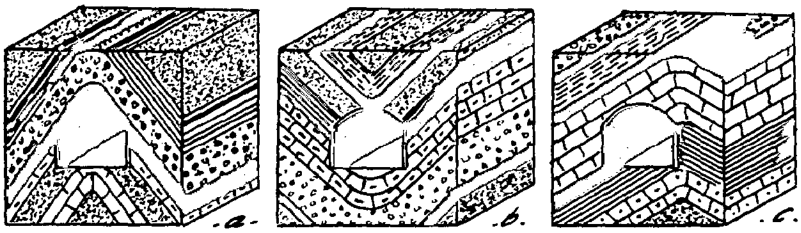


FIG: 3 (e) EXCAVATIONS AND ROCK FOLDS. PRINCIPLES OF ENGINEERING GEOLOGY AND GEOTECHNICS, D.P. KRYNINE & W.R. JUDD, COPYWRIGHT 1958, McGRAW-HILL BOOK COMPANY.

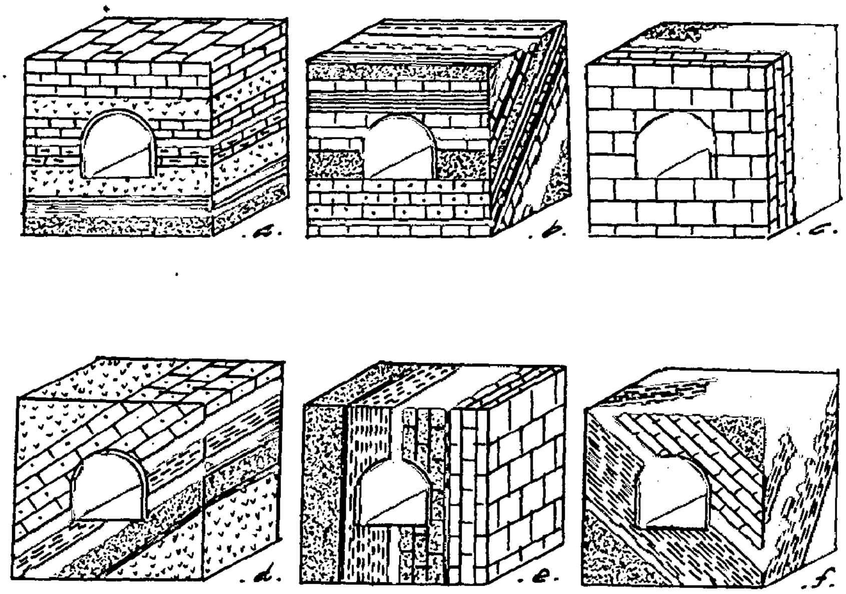


FIG: 3(f) EXCAVATIONS IN BEDDED ROCK. PRINCIPLES OF ENGINEERING GEOLOGY AND GEOTECHNIC.

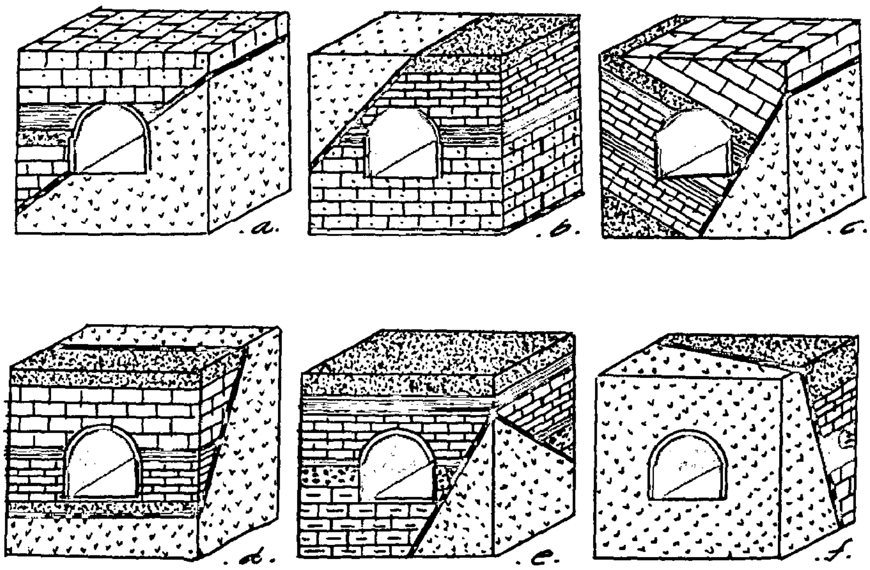


FIG: 3(g) EXCAVATIONS IN FAULTED ROCK. PRINCIPLES OF ENGINEERING GEOLOGY AND GEOTECHNIC.

The degree of folding also influences the stress distribution in tunnel walls.

Most rocks are characterised by incipient fractures and cleavages due to compressive stresses. Fracture in rocks along which there has been some differential movement is called a fault (Fig.3g). Faults are the common trouble and a major cause of bad caving in tunnels. If fault zones are unavoidable in the alignment and if they are predicted prior to the design, the conduit should be located at right angles or nearly so to the fault plane to minimise the amount of difficult operation.

Joints are natural divisional planes in rocks of all kinds in which there has not been any appreciable movement. Joint systems cut up rocks into blocks or sheets of various sizes. A tunnel in jointed rock should as far as possible be located so that its direction is at right angles to the most prominent, most steeply inclined joint system. This course should be chosen because in drilling and blasting the ease of breaking the rock in the heading away from joints is advantageous. A tunnel which follows a major joint system will have the danger of over head caving-in constantly present (Fig.3d).

Other factors such as alteration, permeability etc. are also important. Alteration is responsible to high temperatures, air slaking, and swelling and also noxious gases in tunnels. Coarse grained rocks alter more rapidly on hydration and swelling results. In such cases the tunnel cross section is reduced by the plastic flow. The plasticity increases with water content. No rock is completely impermeable. The effective remedy to control water seepage and to relieve pressure is of great importance in underground works.

3.4 INFLUENCE OF ROCKTYPES:

It is simple to construct a conduit through massive, hard and isotropic rocks as they can be excavated with limited overbreak without support. Common rocks belonging to this table are the granite rocks, basalt, marble and quartzite as well as massive beds of sandstone and limestone. Although granite is homogeneous the orientation of biotite mica flakes in case of biotite-granite produces structural patterns and this leads to irregularly roofed excavation. There is generally no pressure acting on the tunnel sides or on the roof in granite and other isotropic igneous rocks.

Basalt is quite fit for tunnelling and locating large cavities. Marbles and quartzites may be massive or bedded. Marble is easy to excavate. Although quartzite is isotropic, it is brittle and drilling causes shattering which results much overbreak and popping. Massive beds of sandstone and Limestones can be penetrated with some advantage as granite or basalt.

stratified formations are extremely anisotropic. The chief rock formations among stratified rock are Limestone, Dolomite and sandstone through which tunnelling is practicable. The excavation and stability of the stratified rocks are highly influenced by the trend and dip of the beds. When the beds are traversed with joints spaced farther apart than the width of the tunnel, the hard horizontal beds act as solid beams subject only to bending movement and this is less in hard Lime stones, sandstones and greater in Mudstones and Marl. If bending moment is less than the flexural strength of the rock, the roof is stable, or else the roof requires immediate support. Hornblende and Chlorite schists are considerably harder and stable whereas Talc, Mica and altered schists are very soft and earthy and tend to cave-in from the roof. If the tunnel is driven across the foliation in schistose rocks, the roof requires immediate support as the bending moment is greater than the flexural strength.

3.5 PLANNING FACTORS TO CHOICE OF SITES:

The important elements of an underground power house layout are shafts and tunnels for conveying water and for access and ventilation and cavities for housing generating machinery and accessories. The layout of the appurtenant works and conduit system essentially depends upon the location and alignment of the power house cavity with respect to the topographical and geological formation of the area. The largest span in the underground works will be for the power house cavity and it should be oriented in that direction which utilises to the greatest extent the rock's arching action. This occurs when the strike of foliations and the bedding planes is perpendicular to the power house axis. If the power house is so located, there is a tendency to insure that inferior rock is confined to the shortest dimension of the Power house. It is important to investigate the conditions of rock around the pressure shaft and tunnels in a layout. Pressure shafts normally enter the power house at right angles and if the powerhouse is so located that strike of the rock foliations is perpendicular to its axis, the strike is necessarily parallel to the pressure shaft axis. In these circumstances, it is to be expected that overbreak will exceed that which would have occurred had the penstocks have been driven normally to the strike.

Depth of rock cover is an important aspect in the layout. A thumb rule for the overburden (rock cover) above a power house excavation is that it should be twice the width of the excavation, with a minimum of 30 to 35 Ft. It is easy to specify hundreds of feet of rock cover. However care should be taken to verify that the rock is competent to sustain the increased crushing stress induced by excavation. This aspect has been dealt in detail under Chapter IV.

In the case of steel lined sloping pressure shafts, the participation of the rocks depends upon the rock cover provided and upon the characteristics of rocks. A minimum rock ~~x~~ cover of 50% of the design head is usually provided and this may be lowered depending upon the results of rock mechanic investigations. By going deeper, the design of the liner will be governed by the external ground water pressure especially in the upper section of the pressure shaft. From Geological considerations and cover requirements, the most economical layout dictates the keeping of the underground penstocks as close to the rock surface

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CHAPTER IV

ROCK MECHANICS IN THE DESIGN OF UNDERGROUND CAVITIES AND TUNNELS

4.1 GENERAL:

Underground rock, as a result of nature is under stress and the excavation of an opening causes a redistribution of stress and displacement of rock which tends towards a new condition of equilibrium (67, 16, 14, 28). Displacement of rock may involve sliding on planes of weakness, fracture or flow. To prevent displacement and to achieve permanent stability, artificial supports might be required. From considerations of economy, the primary objective should be to make the rock support by itself in a safe manner. The magnitude and direction of principal stress often dictate the orientation and shape of underground cavity. The shape best suited to the functional requirements and the natural stress conditions at site and the measures required to attain permanent stability have to be studied prior to final design of the cavity. These studies will produce considerable savings in cost during construction.

4.2 STRESSES IN A ROCKMASS:

Pressures due to the weight of superincumbent strata produce stresses in rock masses. Considering ideal conditions-rock is homogeneous, isotropic, perfectly elastic and nowhere do the stresses exceed the elastic limit - the vertical stress in the rock prior to the excavation of a cavity is exclusively due to gravity (67) given by $\sigma_y = W_r z$

where W_r is the density of rock and 'z' is the depth. If the rock were free to expand laterally, σ_v would be the only stress acting in the rock. If the lateral expansion had been prevented while the stress σ_v had been applied and the rock had remained elastic throughout that time, then horizontal stresses would be set up (67, 16). Fenner (1938) gives the solution for the horizontal stresses resulting from vertical loading in an elastic body as

$\sigma h_1 = \sigma h_2 = \frac{\mu}{1-\mu} \sigma v$ where σh_1 and σh_2 are the horizontal stresses, being Posson's ratio. At greater depth, hydrostatic condition can be assumed (Heim's Theory) so that $\mu = 0.5$ and and the horizontal stresses would be same as the vertical stresses. Rock material has generally been ascribed a Poisson's ratio of 0.2 to 0.3 which indicates horizontal stresses between 0.25 to 0.43 times the vertical stresses (50).

By virtue of the nature of rock masses, however, the application of a value for Poisson's ratio is inappropriate and the theoretical ratio of horizontal to vertical stress cannot apply and therefore the above relationship for computing stresses has little practical significance.

Terzaghi (67) has introduced an equation $N = \frac{\mu}{1-\mu}$ in which the assumptions are (i) the stratum under consideration has never carried any temporary load such as the weight of a sheet of ice or other deposits and subsequently removed by erosion and (ii) the temperature of the layers of the stratum has not changed. The dimensionless factor

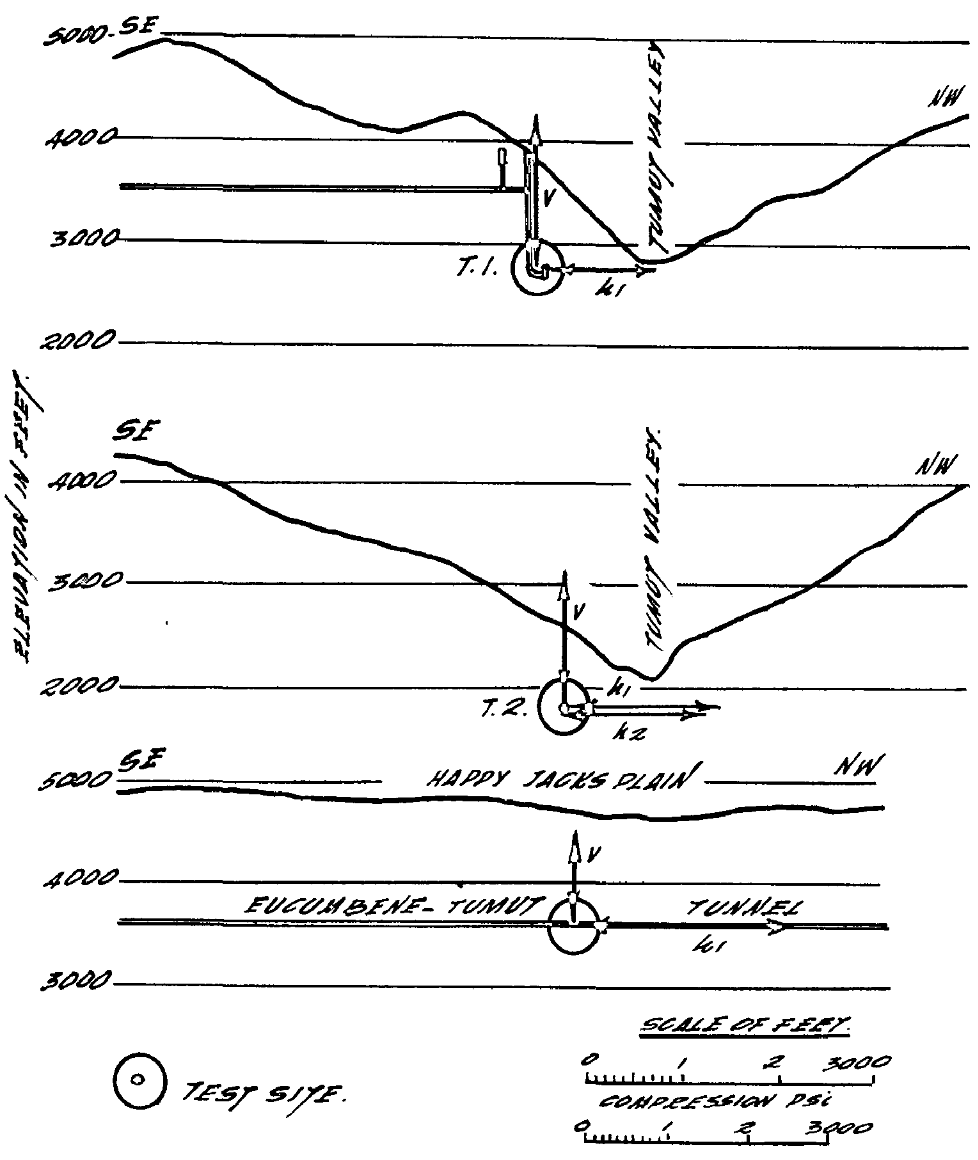


FIG: 4(a) NATURAL STRESSES AT THREE LOCALITIES IN THE SNOWY MOUNTAINS.

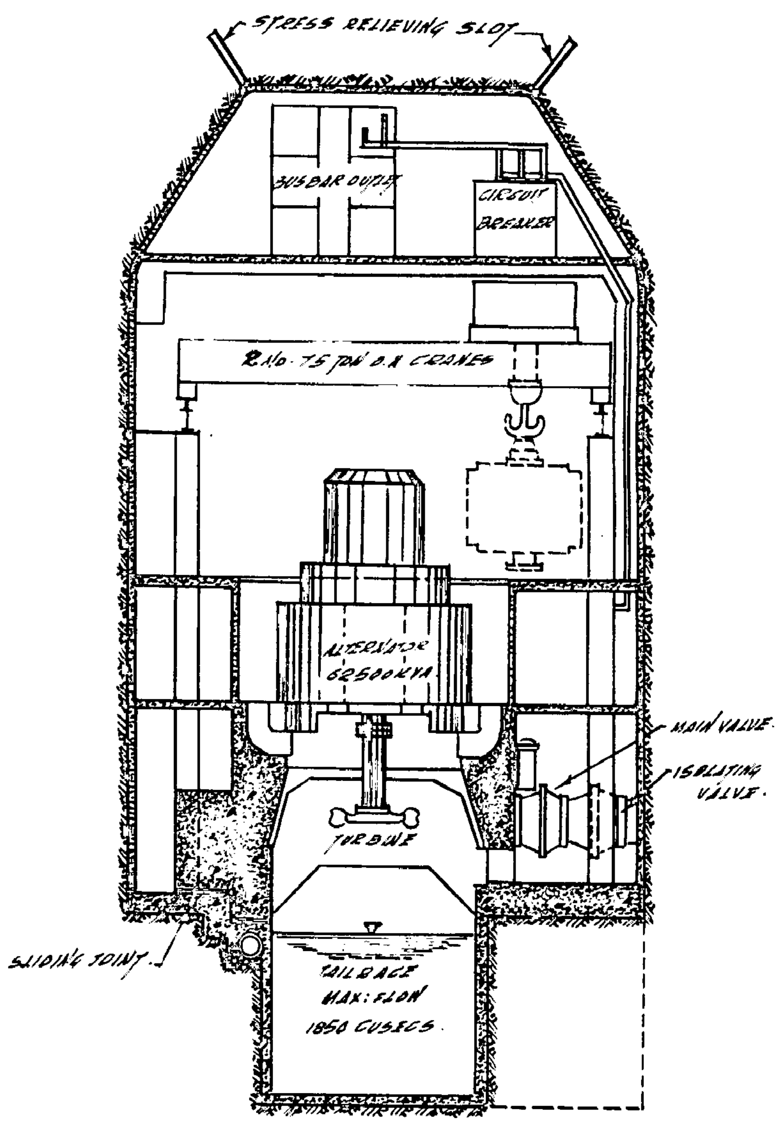
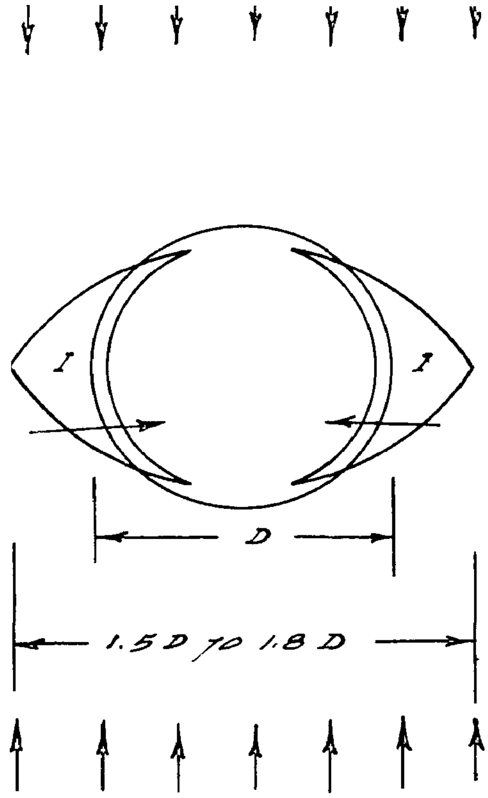


FIG: 4(b) POATINA POWER STATION - CROSS SECTION.

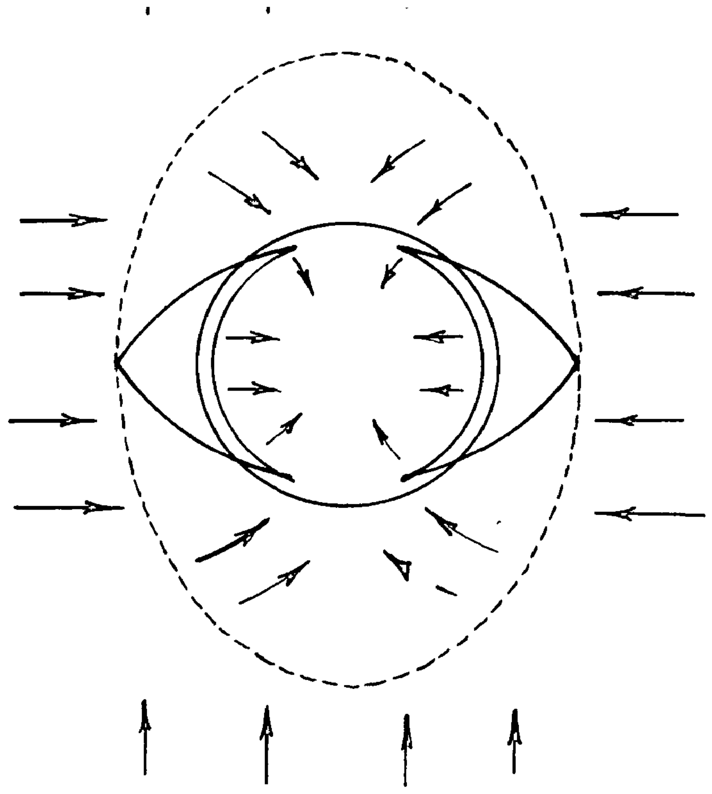
N depends on the geological history of the mass of rock and its value varies with depth and it may even have a different value, in different horizontal directions. If a geologically undisturbed stratum of sedimentary rock has temporarily carried a heavy overburden, its N value, according to Terzaghi, may range between $\frac{\mu}{1-\mu}$ and values close to unity. In folded and faulted metamorphic rocks, at great depth N may have any value compatible with the strength of the material. On account of the uncertainties, involved in the estimate of N value, it is always desirable to devise methods for computing N value on the basis of physical data.

According to the data contained in the Final Report on the Boulder Canyon Project (76), the initial horizontal stress in the rock surrounding the tunnel (located at a depth of about 150 Ft. below the bottom of canyon) is almost equal to three times the overburden pressure. This is attributed to the steep rise of the rock slopes on both sides of the canyon floor to a height of more than 1000 Ft. MOYE (48) in his investigations in the Eucumbene-Tumut tunnel through granite (fig.4a) has computed the natural compressive stress in the horizontal direction to be 2.6 times the stress in the vertical direction and vertical stress about 0.8 of the stress due to the weight of overlaying rock. The reason for the high horizontal pressure is attributed to the tectonic forces active in the region. The natural values of stresses at T1 and T2 Power stations (located in Granite Fig.4a) computed by Moye also showed similar features. The compression in the vertical direction was greater than that due to the weight of rock vertically above the power station sites. It is considered that these abnormally high values are perhaps due to the V-notch effect of the valley causing transfer of part of the stresses due to the weight of the steeply rising ground to the rock below adjacent lower ground. Tests on V-notch model of the valley demonstrated qualitatively that such stress concentrations are possible. Endersbee (16) in his investigations relating to Poatina Underground station site (located 500 ft. under ground and area comprising bedded x sand stones, silt zones and shales) has computed that the vertical stress field is greater than 1.6 times the weight of rock directly above the site. He suggests that there is shear transfer from the adjoining high ground which partly encircles the site. The horizontal stress computed has a value 3 to 4 times the overburden pressure and twice the measured vertical stress. The investigations of the U.S.B.R. in the George Power House penstock tunnel (located at a depth of 200 ft.) below a gentle ridge consisting granitic gneiss) led to the conclusion that the initial horizontal pressure was considerably smaller than the initial vertical pressure.

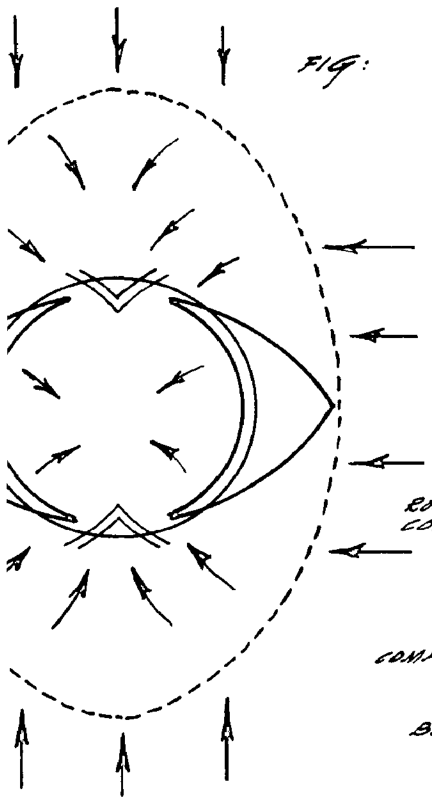
The above results demonstrate that in order to obtain the state of stress in rock at depth it is necessary to make actual measurements. The proximity of deeply cut valleys results in increased stress in adjacent lower ground. In addition it is evident that the past tectonic history



STAGE: I.



STAGE II.



STAGE III.

FIG: SKETCH OF MACHANICAL PROCESS AND SEQUENCE OF FAILURE AROUND A CAVITY BY STRESS REARRANGEMENT PRESSURE.

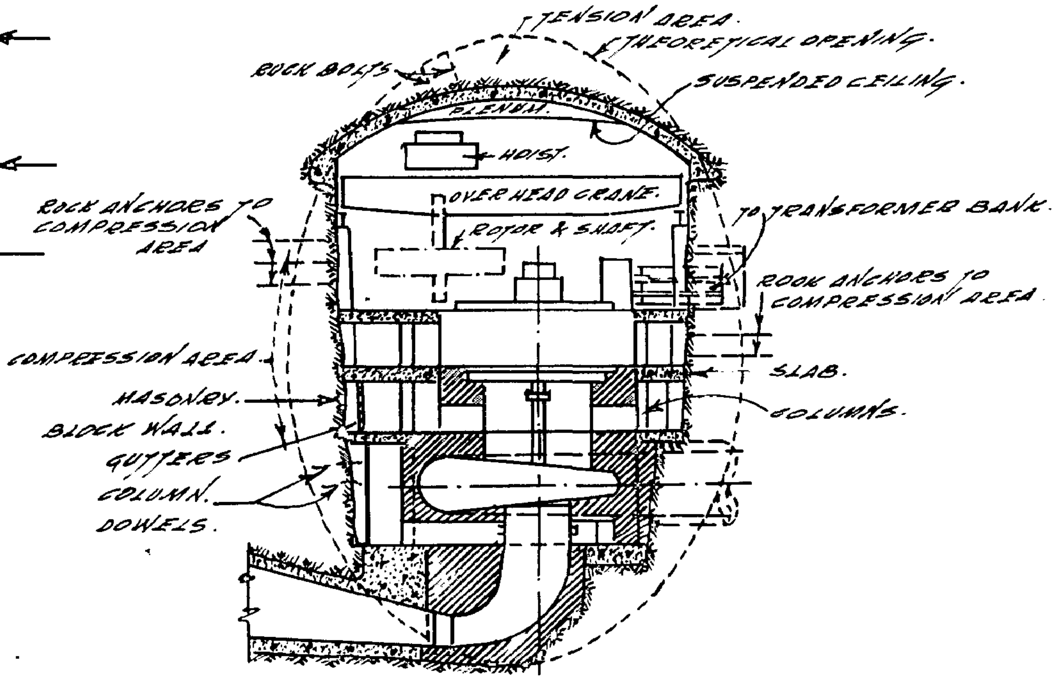


FIG: 4(C) PROPOSED EXCAVATION.

in the region and the present state of tectonic activity also may influence on the state of stress in a manner which cannot be reliably predicted. The initial state of stress has a major and pervading influence on the behaviour of the rock around underground openings. It is therefore considered that actual measurements of the natural state of stress, even though the results may be used with caution, owing to the errors inherent in such measurements, is a prime essential for the intelligent design and construction of underground works.

The actual state of stress following excavation of opening depends not only on the initial state of stress condition in the rock, but also on the shape of the opening, the dimensions of the opening and the geological conditions actually encountered. Many investigators, Terzaghi (67, 68), Mindlin (45) have investigated stress concentrations for circular and elliptical tunnels and spherical cavities. These results are useful as a guide for normal tunnel work but must be used with considerable judgement in the case of a large irregular opening such as a power station. Rabcewiez (60) describes that the stress re-arrangement is mechanical or progressive and generally occurs in three stages (Fig.4c) thus making an opening with elliptical shape. This curve would represent the upper limit of the potential rock fall into the opening. It would therefore be ideal if we can confine the equipment layout within this ellipse. Usually such an ellipse is drawn with a ratio of major to minor axis as 3:2. Assuming a Poisson's ratio of 0.25 for the rock, the stress distribution at the crown of the ellipse is ± zero (65). This condition at the crown of the ellipse will alter only if the maximum shear stress exceeds the shear strength of the rock. If a concrete arch is placed within this ellipse, it need be designed only to carry the weight of rock between the arch and the elliptical opening (83).

4.3 DESIGN CONSIDERATIONS OF UNDERGROUND CAVITIES:

Rock surrounding an underground opening will fail if the stress in the rock exceed the ultimate strength of rock. Failure criterion is based on maximum stress theory ie. rock fails in tension when the tensile stress exceeds the tensile strength of rock. If the tensile strength is small, the rock will fail in shear at a value of compressive stress equal to the compressive strength of rock. The strength and elastic properties vary widely from place to place and these should be determined by adopting good sampling procedures. Due to the uncertainties involved, large safety factors should be employed in the design work.

Rocks are classified into two groups viz. massive rock and horizontally bedded rock. Massive rock (ex.granite, diorite, basalt, quartzite etc.) are assumed to be elastic, homogeneous and isotropic. Horizontally bedded rocks include most of the sedimentary rocks and some metamorphic rocks.

These types of static stress fields are usually considered when designing underground openings (14) as illustrated in Fig.4d. The state of stress represented

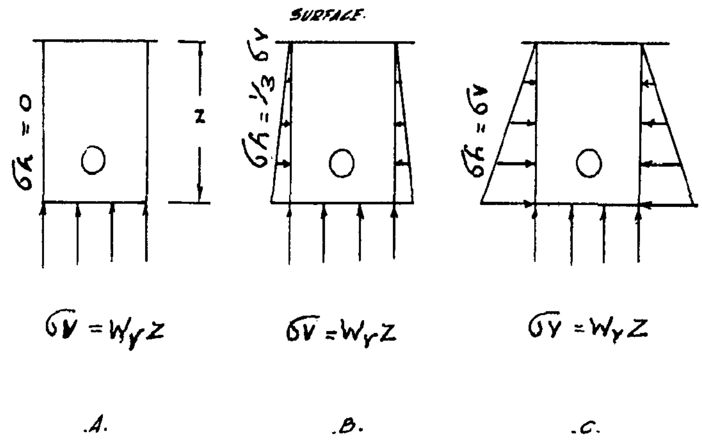
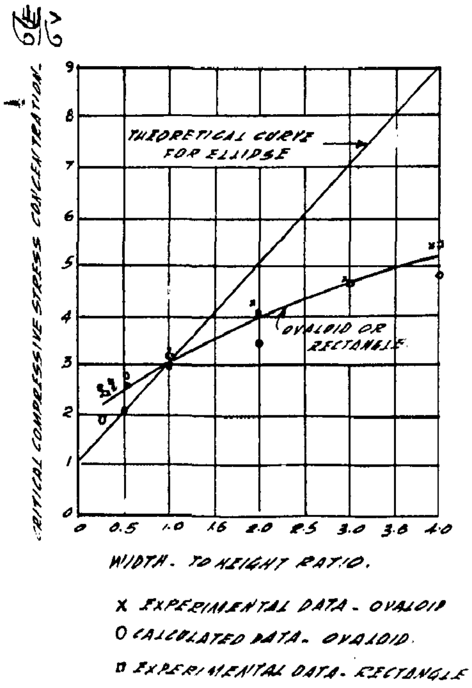


FIG: 4(d) THREE ASSUMED TYPES OF STRESS FIELDS.



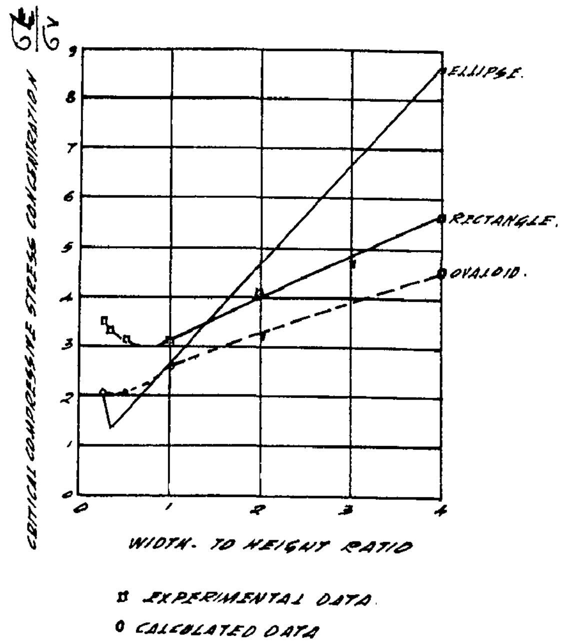


FIG: 4(e) CRITICAL COMPRESSIVE STRESS CONCENTRATION FOR TUNNELS OF VARIOUS CROSS SECTIONS. UNIDIRECTIONAL STRESS FIELD.

$\sigma v = W_{\gamma} Z$ $\sigma h = 0$

FIG: 4(f) CRITICAL COMPRESSIVE STRESS CONCENTRATION FOR TUNNELS OF VARIOUS CROSS SECTIONS, TWO DIRECTIONAL STRESS FIELD.

σh $\frac{1}{3}\sigma v$ $\sigma v : W_{\gamma} Z$

by type A would be expected to occur at shallow depths, near vertically free surfaces. Type B (when Poisson's ratio 0.25 is assumed - Terzaghi 67) would occur in geologically undisturbed rock over wide ranges of depth. Type c state of stress would occur at great depth in geogically disturbed rock where tectonic forces are active. In analysing the stress distribution, two directional stress field is only considered while the stress distritution along the length of the opening is assumed to be unform.

For an applied stress field of type A, the critical compressive stress concentration for various shaped openings as a function of width-to height ratio of the opening is given in Fig.4e. Ovaloids and rectangles with rounded corners are preferred cross section shapes if the width-to-height ratio is greater than 1 and elliptical section is preferred if the width-to-height ratio is less than 1. For type B applied stress field (Fig 4f) also ovaloidal or rectangular cross sections are preferred if the width-to-height ratio is greater than 1 and elliptical cross section preferred if the width-to-height ratio is less than 1. For type C applied stress field (Fig.4g) circle or ovaloid is the preferred cross section regardless of width-to-height ratio. The above conclusions are of Duvall (14).

Another problem is to consider the effect of having two or more openings underground which are parallel to each other and separated by a wall of rock. The stress fields around two or more parallel openings add together to give

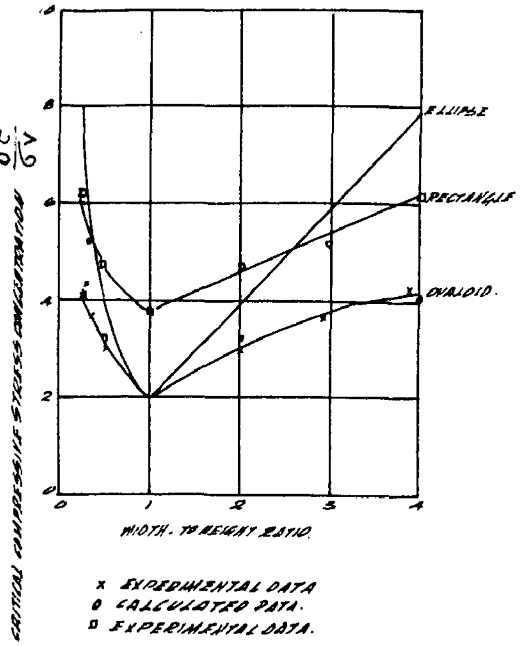


FIG: 4(g) CRITICAL COMPRESSIVE STRESS CONCENTRATION FOR TUNNELS OF VARIOUS CROSS SECTIONS, HYDROSTATIC STRESS FIELD. $\sigma_h = \sigma_v$. $\sigma_v = W_r Z$

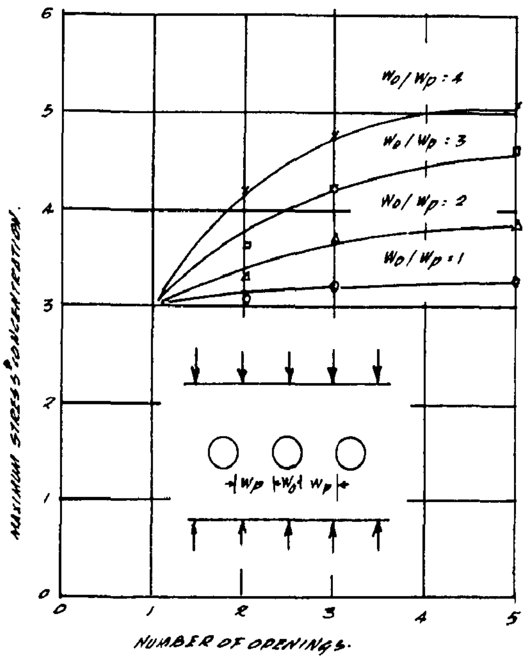


FIG: 4(h) MAXIMUM STRESS CONCENTRATION AS FUNCTION OF NUMBER OF OPENINGS AND OPENING-TO-PILLAR WIDTH RATIO.

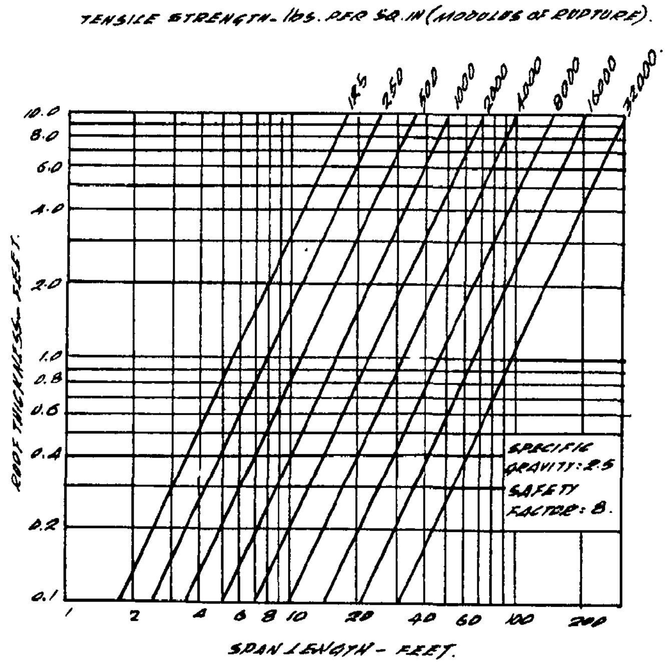


FIG: 4(i) SAFE SPAN LENGTH FOR VARIOUS ROOF THICKNESSES AND TENSILE STRENGTHS OF ROCK.

increased stress concentrations if the wall between the openings is less than one diameter of the opening. Fig.4h illustrates the case of equal size, equally spaced circular openings in which as the number of openings increases the maximum stress concentration increases rapidly at first and approaches an upper limit. The conclusion is that in designing underground openings, the spacing of rooms should be such that the pillar width is equal to or greater than the room width. For the T1 Power station photoelastic studies showed high stress concentrations when the transformer hall was located parallel to the machine chamber separated by a narrow pillar. To limit these stress concentrations, the transformer hall was finally located at right angles (Fig.6i) to the machine Chamber (48). At Serre-Poncon (18) the rock forming a wall between the machine chamber and transformer hall was brought under compression by providing tie rods of 120 Tons and thus increased safety was ensured (Fig.6a)

The stress concentration that occurs when two or more openings intersect should also be considered. Intersection at right angles is considered better than oblique ones for minimising stress concentrations.

In bedded formations, the boundary between different beds is an inherent plane of weakness and bed separation can occur. Therefore it is to be considered the additional stresses that would set up in the roof rock as a result of having a gravity-loaded slab of rock over a span equal to the room width. Fig. 41 shows the relation between slab thickness, span length and strength as given by beam theory (14). From these curves it can be said that un-supported underground openings in thinly bedded formations are not too satisfactory. If a power station is to be located in bedded formations underground, elaborate investigations have to be made to determine the size and shape of cavities. Poatina station is an example which is located in bedded formations consisting of sand stones and Mudstones. Investigations led to the conclusion that a semi-crular or semi-elliptical roof for the machine cavity would fail by shearing along bedding planes that intersected the surface near the crown. A rectangular shaped opening was disregarded since the roof stress near the walls exceeded the compressive strength of the rock and shear on the bedding planes at the corner exceeded the shear strength. A trapezoidal shape (unconventional) was considered best and the same was adopted (Fig.4b) Even for this shape the boundary compression in the roof near the haunches exceeded the compressive strength and the shear on the bedding planes near the haunch crown inter-section was greater than the shear strength. These high stresses were brought down to desired limits by stress relieving slots (Fig.4b).

4.4 ROCK STABILITY AROUND PRESSURE TUNNELS:

A pressure tunnel with a diameter 2R located at a depth H under horizontal rock surface would be perfectly safe without any lining if the water pressure P inside the tunnel

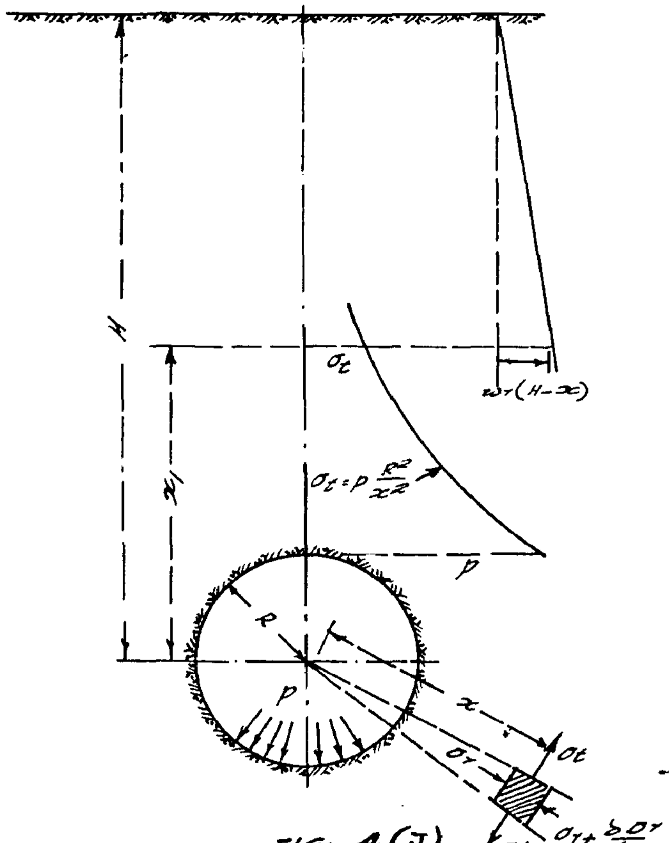


FIG: 4(J)

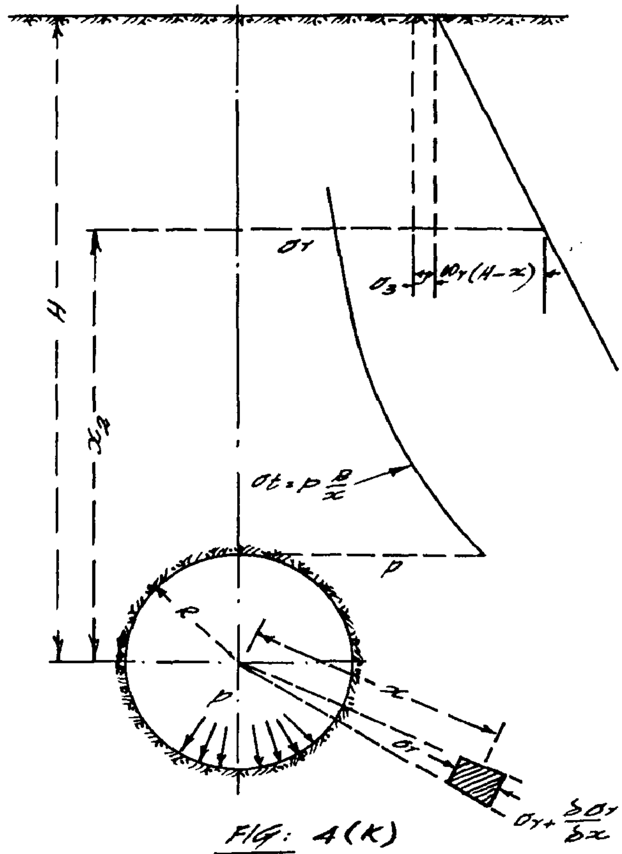


FIG: 4(K)

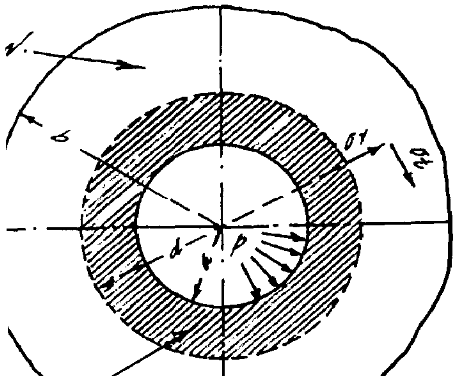


FIG: 4(l)

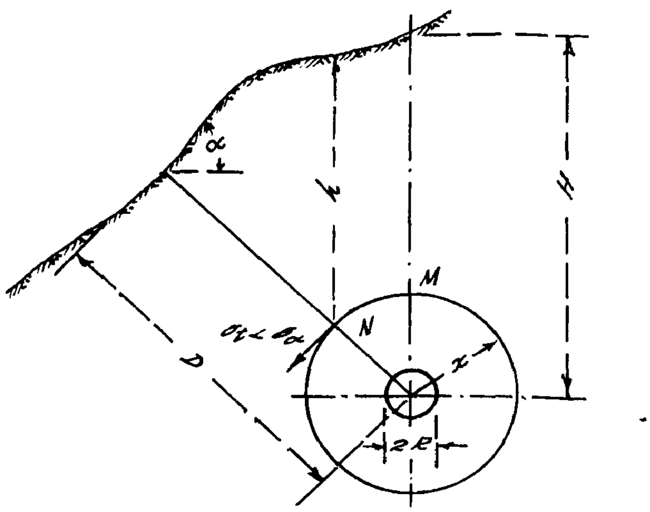


FIG. 4(m)

is less than the pressure P^1 produced by the rock's own weight. The hydrodynamic pressure inside the tunnel can thus be written as $P/W = \lambda H$ where λ is a factor and W = specific density of water and P/W and H are in feet or in metres. An accepted rule of thumb assumes $P/W = \frac{1}{4}H$ or $\lambda = \frac{1}{4}$. This rule of thumb is based on the idea that the weight of a slice of rock with a width B = 2R, a height H and a length 1 should be equal to or larger than the vertical depth on the same area 2R x 1 and with this condition a factor of safety of 5 is obtained. When rock density is assumed as 2.5 times the density of water than the limit value for λ would be $\frac{1}{2}$. Such a rule according to Jaegar (30) is too crude. He has further analysed this problem and showed that the rock cover over pressure tunnels mainly depended on the ratio of H/R of overburden H to tunnel radius R. Jaeger has also established that the minimum overburden required over a pressure tunnel is far greater in fissured rock than in sound rock.

According to Fig.4j at a depth (H - x), $\sigma_v = W_r (H - x) = P^1$ and horizontal stress $\sigma_h = K W_r (H - x)$. For hydrostatic stress distribution, K should be taken as unity when $\sigma_v = \sigma_{h_1, h_2} = P^1$. Again at a distance 'x' from the centre of the tunnel, filled with water at a pressure P, the circumferential tensile stress (70) is given by the equation:

$$\sigma_t = P(R/X_1)^2 = \sigma_r \text{ where } \sigma_r = \text{ radial compression.}$$

The balance of pressure at a depth $(H - x_1)$ can be assumed (neglecting rock's permissible tensile strength $\sigma 3$) so that $- \sigma t \leq \sigma h$ or

$$P\ (R/x_1)^2 < k\ W_r\ (H - x_1) \ldots\ldots\ldots\ldots\ldots (1)$$

By writing $x_1 = H/n$ and $P = \lambda WH$, then

$$\lambda \leq (H/R)^2\ k\ (Wr/w)\ (n - 1)/n^3 \ldots\ldots\ldots\ldots (2)$$

$$\text{or } \lambda \leq (H/R)^3\ m \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The above equation is valid provided the rock is nowhere fissured. The condition that at a depth $H - x$, no lifting of rock should occur yields.

$P\ (R/X)^2 < W_r\ (H-x)$, a condition less severe than equation (1).

For tunnels in fissured rock (Fig.4k), the relationship is

$$\lambda_2 \leq k(Wr/W)\ (H/R)\ (n - 1)/n^2 - \sigma 3/n\ WR \ldots\ldots (4)$$

assuming that at a height $x_2 = H/n$ above the tunnel centre line fissures caused by tensile circumferential stresses should not occur or $P(R/x_2) = \lambda_2\ nWR \leq kWr\ (H-x_2) + \sigma_3$

When a tunnel is steel lined, the pressure to be considered which is calculated from known methods (30).

Limit values of λ_P for the tunnels located in plastic rock (Fig.41) can also be established after deriving internal pressure equation based on elastic equations.

Assuming that $n = 3$, $Wr/W = 2.5$ and $k = 0.70$, values of λ, λ_2, λ_P are calculated for varying values H/R given as under:

H/R	λ	λ_2	λ_P
5	3.2	1.95	1.88
10	13	3.9	3.4
[illegible]	[illegible]	[illegible]	[illegible]

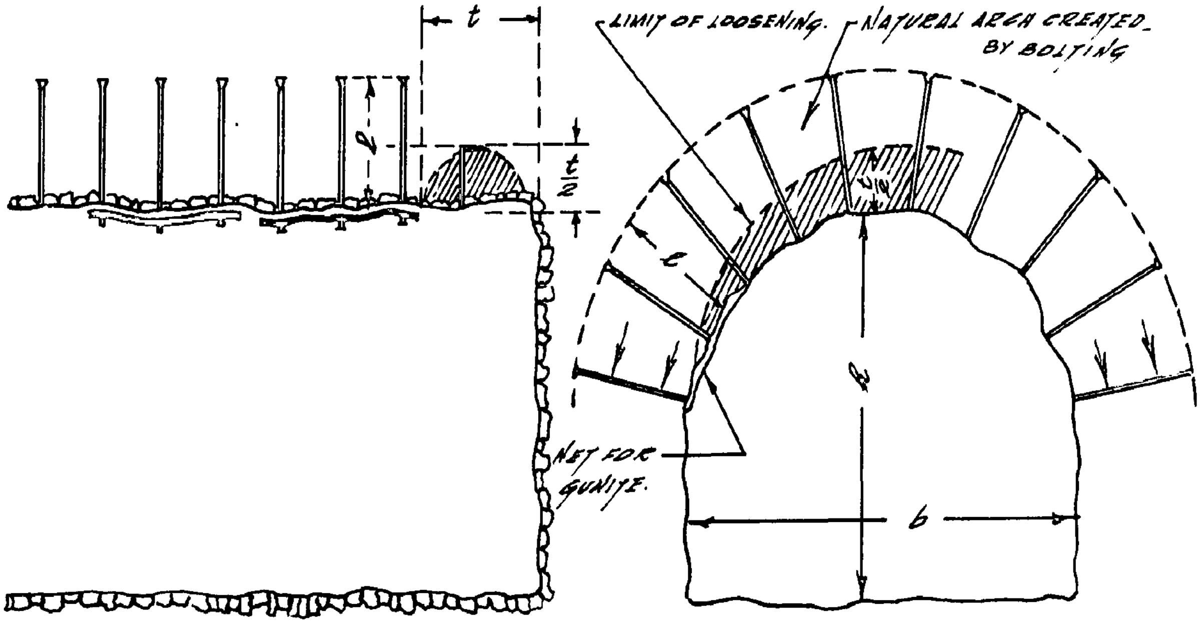
This table clearly shows the paramount importance of the rock quality and rock strength when determining the required overburden for protection of a pressure tunnel. Plastic rock is substantially less safe than fissured rock which in turn is less safe than solid massive rock.

When a pressure tunnel is located near an inclined slope (Fig. 4m), $\sigma v = W_r H$ and $\sigma h = k^1 \sigma v$. The constant k^1 will be substantially less than k, the value assumed when the rock surface is horizontal. With the above values, the net compression stress σa in a direction α which should show that $\sigma t \leq \sigma a$. stresses at point M vertically above the tunnel centre must be checked as they may be less favourable than the stresses at point N.

Jaeger has also shown that in fissured rock when infiltration occurs $\lambda_2 \simeq k \quad Wr/W$ and for values of λ_2 greater than that given by equation (4), the rock will be fissured. In such cases, if the lining is not watertight for that water pressure, water will penetrate into rock and percolate through it.

4.5 PERFORMANCE OF ROCK ABOUT CAVITIES:

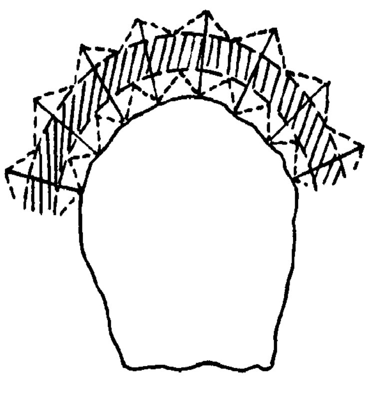
During the excavation of a cavity, a state of stress is reached around the cavity (Para 4.2) compatible with the strength of the rock. If failure occurs, the roof commonly assumes the shape of an arch. This fact indicates that the stress conditions in the rock above an arched roof are less injurious to the roof material than those above roofs of any other shape. Intact, isotropic rock may fail by "Popping"



t - LENGTH OF ONE ROUND.
l : LENGTH OF BOLT ⩾ DEPTH OF PRESUMED MAXIMUM LOOSENING
(AFTER RABEEWICZ - REF: ..59.)

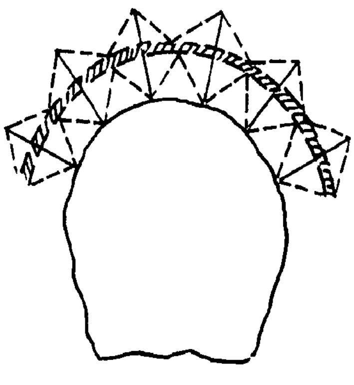
FIG: 4(R)

EFFECT OF ROCK BOLT ON DEPTH OF THE STRENGTHENED ZONE FOR A 20 FT: DIA: TUNNEL.
REF: NO: ....



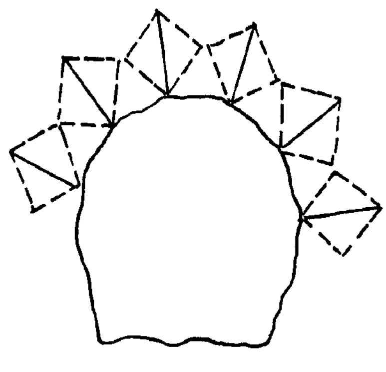
8 ROCK BOLTS 8 FT-
LONG SPACED 4 FT.
RESULTS WIDE ZONE OF UNIFORM
COMPRESSION (HATCHED AREA).

FIG: 4(O)



6 ROCK BOLTS 8 FT-
LONG SPACED 5 FT.
RESULTS NARROW ZONE OF
UNIFORM COMPRESSION. (HATCHED AREA).

FIG: 4(P)



6 ROCK BOLTS 8 FT.
LONG SPACED 6 FT.
RESULTS NO INTERACTION.

FIG: 4(Q)

(detachment of slab-like fragments from roof or wall) while jointed rock fails along joints and this process is called "stopping". Both these processes are associated with an increase of the ratio between height and the span of the cavity and change in the state of stress in the rock. The process of "stopping" or "Popping" is prevented by temporary supports subsequently replaced or supplemented by a permanent lining. Modern method is to give support by rock bolts which provides a positive force on the rock and provides a stressed membrane distinguishes it from steel or timber support. When used in appropriate patterns, the bolts create a principal compressive stress normal to the free surface of the excavation and this, in turn, creates a zone of rock which acts as a structural membrane capable of providing its own support (see Fig.4n, o, p, q) 'Popping' or 'stoping' can be prevented if the bolting is done immediately after the excavation. Rock bolts may be used for temporary support and permanent support. When it is employed for permanent support, grouted bolts are preferred. Modern trend is to use grouted bolts for the roofs of power house cavities. In certain stations, (Ex.Tumut 2 station located in Granite) in addition to grouted rock bolts, a reinforced concrete arch roof is also provided to prevent loosening of the closely jointed rock and possible minor rock falls. The method of rock bolting (54, 69, 34, 79) has been perfected in recent years and designers are now bold enough to locate underground cavities even in loosened and fractured rocks.

4.6 PRESSURE CONDUITS:

Pressure conduits may be unlined or lined with concrete or lined with concrete and steel. The need or otherwise of a concrete lining is often determined by economic rather than structural considerations in that a concrete lining reduces hydraulic losses. In some special cases, however, a concrete lining may be essential when the ~~concrete~~ ratio of the initial horizontal to vertical stress is low, tension can easily occur at the soffit in an unlined tunnel when water pressure is applied. Since rock at the soffit of a tunnel is often shattered during excavation, rock falls from roof can occur when the internal pressure is applied. To prevent this, concrete lining is sometimes provided. When a lining with concrete is employed, it is to be deisgned for external water pressure on dewatering the tunnel. Also additional stresses can be developed due to plastic flow in the surrounding rock (Tunnels in limestones, sandstones and shales). Methods of calculating these stresses has been dealt by several authorities (45, 56).

Steel lined shafts take into account the relative elasticity of steel lining, the concrete and the rock and participation of rock and steel in taking the internal water pressure. Among the several factors which can influence the decision as to whether a steel lining is required in any section, the most important being the need to prevent excessive leakage from the conduit at points where the rock cover might be insufficient to prevent rock heave.

The thickness of a a steel lining is small in relation to the diameter of the pressure conduit and the design is very often determined by external water pressure on the lining when the conduit is dewatered rather than by internal water pressure. This is especially true for conduits subject to maximum internal pressure heads less than 1000 Ft. (53). External water pressure can be reduced by various methods and these have been discussed in Chapter VI. Among the various methods employed in increasing the critical buckling stresses of the steel liner, the best known method is to provide longitudinal ribs on the external face of the pipe as at Vianden (73) ULLMANN (74) has demonstrated that the above arrangement offers more security than that provided by Amstutz's method. In the design of liner the maximum external pressure head that can exist outside the steel lining will not exceed the depth of cover over the steel lining.

Amstutz (2) and Vaughan (77) both have produced curves and formula for the design of steel liner for external water pressure. One of the assumptions made in using the Vaughan and Amstutz formulae is the thickness of the gap between the steel liner and the surrounding concrete. The size of the gap depends on site conditions and the construction techniques employed.

Formulae to determine the portion of the total load which will be taken by the surrounding rock and by a steel liner subject to internal pressure have been proposed by Jaeger (29) Vaughan (77), Bleifuss (4). It can be calculated if the moduli of elasticity of steel and of the rock are known, the elastic deformation of the liner in the radial direction being equated to the radial deformation of rock. The principal difficulty in this calculation is to assume the real value of the modulus of elasticity of the rock formation traversed by the pressure conduit.

Important factors to be considered in the design of steel liner are (i) gap between the steel liner and concrete (ii) Quality of rock and (iii) Rock cover. With suitable construction techniques the gap between the steel liner and concrete should be kept minimum possible. Rock cover can be calculated based on the assumptions contained in para 4.4. Due to the uncertainty which exists with regard to the rock properties it is generally assumed that rock cover over a pressure conduit (not steel lined) should never be less than 50% of the maximum internal pressure head. With this assumption, cracks in the rock above the soffit of the tunnel will not occur (53) if the ratio of the initial horizontal stress to the initial vertical stress is greater than 0.46. If this ratio is less than 0.46, cracks and consequent leakage can occur. But rock with a unit weight equal to 2.5 times the weight of water there still exists a factor of safety of 1.25 against heaving of the full depth of overlying rock.

In steel lined shafts, if the gap between the liner and concrete and the concrete and rock is zero, the rock cover can be reduced to fairly low values before a material increase in the stress occuring in the steel liner.

However, when the vertical cover is less than four times the diameter of the pressure conduit or when the pressure conduit approaches a rock face, as for example is the case at the upstream wall of an underground power house, it is desirable to design for a greater factor of safety than might be required for other sections of steel lined pressure conduit (53, 67). It happens that very few quantitative values for rock properties and in particular for the value of N for rock in a given locality are available, it is usually necessary to decide a minimum cover required over any given pressure conduit from experience in similar rock formations and from a study of the geological history associated with the rock formation in question.

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CHAPTER V

GENERATING MACHINERY AND OTHER EQUIPMENT FOR UNDERGROUND STATIONS

5.1 GENERAL

As in overground stations, all the types of turbines - Pelton, Francis and Kaplan - have been employed in underground stations. Deriaz type has been installed as at Culligran U.G. station (63). The selection of turbine is mainly governed by the head. However, increased importance is given in the selection of hydraulic and electrical equipment for underground stations to minimise construction cost and maintenance cost. Considerations in the selection of equipment are given as under.

5.2 TURBINES:

Remarkable results have been achieved in hydraulic turbine research and there is increase in size of the three main types and also permissible head that might be applied to each one of them. The increase in size had resulted in a reduction in the specific cost of manufacture and also an improvement in efficiency. In Sweden, the cost of turbine had remained unchanged since 1950, inspite of a general increase of 70% in costs (75). In other words, turbines have been made to supply more electricity than ever before in relation to the amount of material used in their construction.

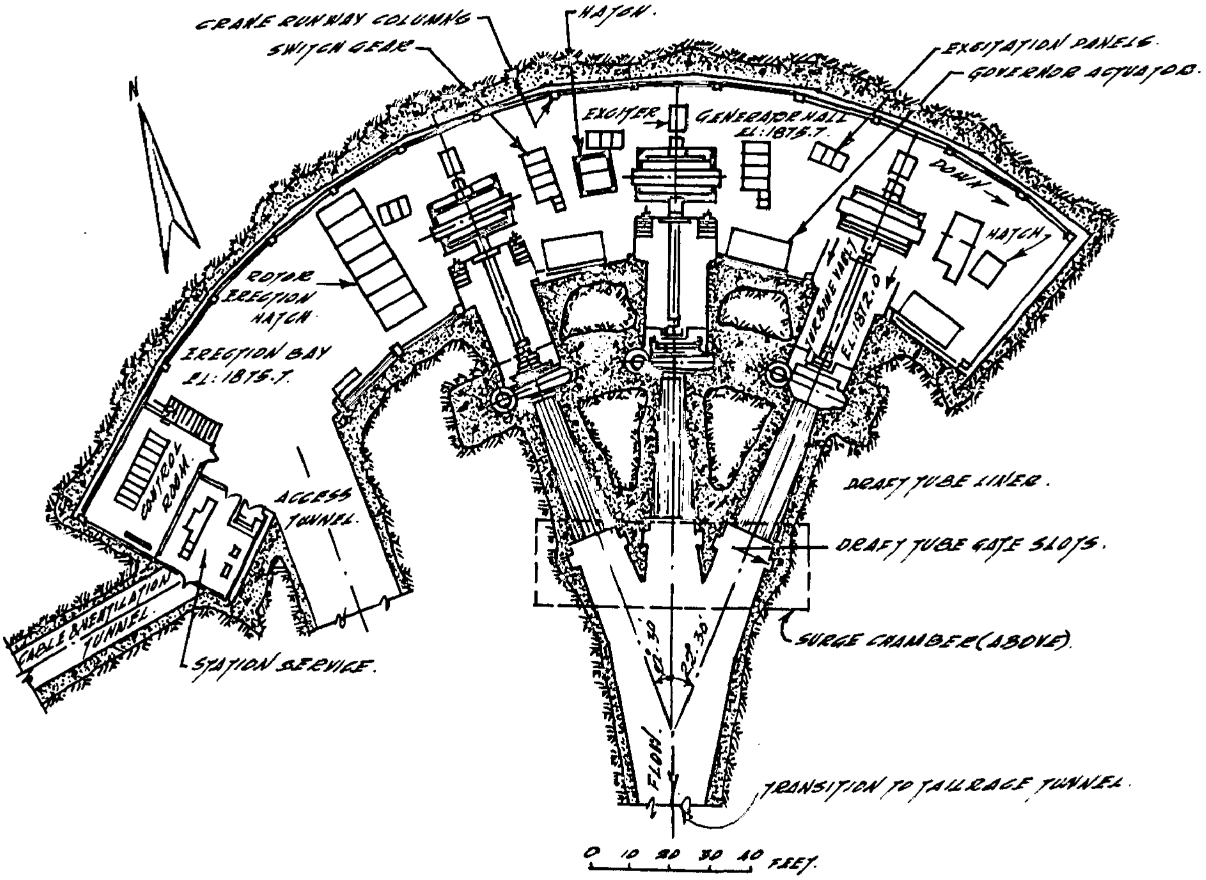
5.2.1 Increases in Output: The development achieved today enables a single runner Pelton turbine to be constructed for 250,000 H.P. (Mont Cenis Plant, France). Infiernillo U.G. station is equipped with 235,000 H.P. Francis turbines.

Francis Units producing 350,000 H.P. have been built in Russia and there is some talk of Russian and American projects for 4000,000 H.P. 600,000H.P. and even 500,000 kw (57) French designers have in view to produce 800,000 H.P. Francis turbines. How much further this development can go depend on runner casting, tooling and transport capacity problems. Individual Kaplan turbine units have now reached and passed the 100,000 H.P. mark. There are proposals to produce Kaplan units for 150,000 H.P. and 210,000 H.P.

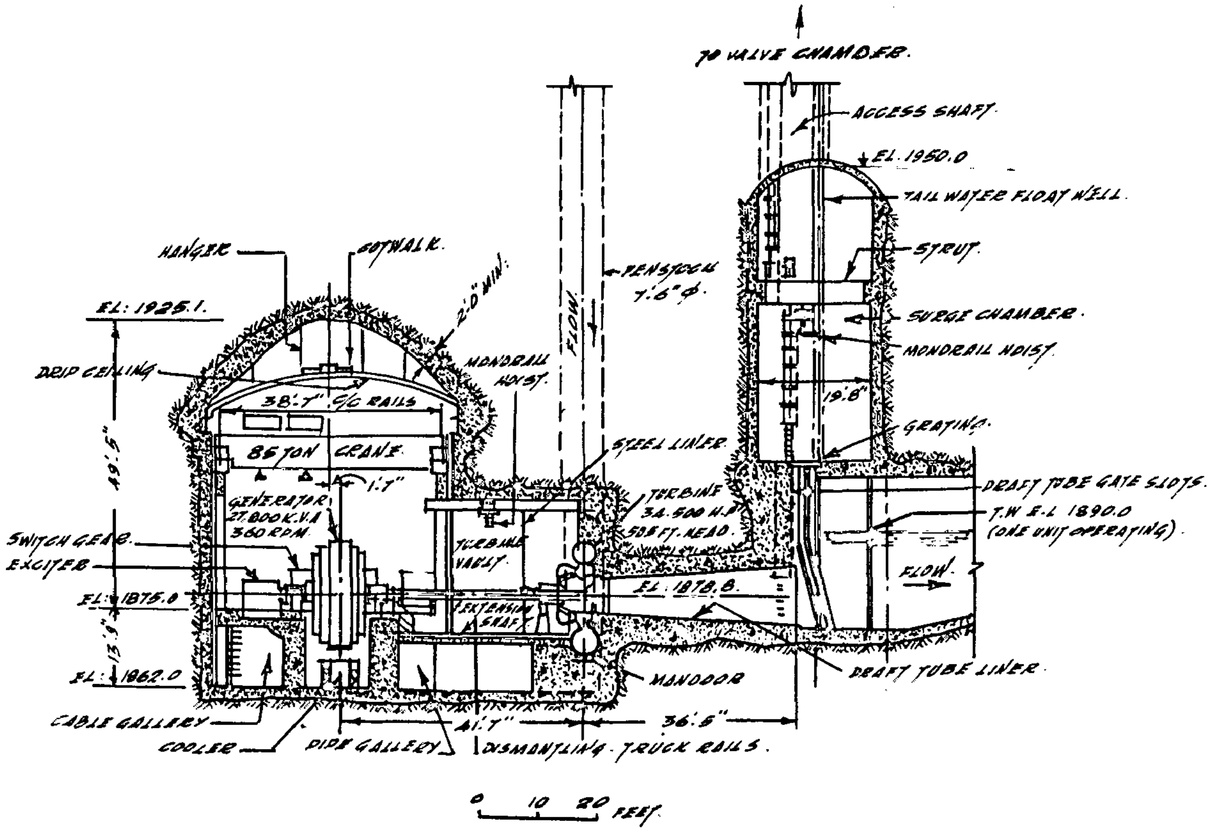
Pelton turbines are installed under very high heads whereas Francis turbines can be relied upon to utilize heads in the lower range of high heads. The range of application of the Francis turbine has been extending towards the higher head range (1800 Ft) during the last few years, which was formerly the sole preserve of the Pelton turbine. Kaplan and Francis turbines are competing in the 50 m to 80m head range much in the same way as Francis and Pelton turbine at the other end of the head scale.

There are certain limitations in selecting large capacity units for underground power stations. Depending upon the type of rock strata, the span of the Power House cavity is invariably limited and hence the unit size. Besides, this, transport limitations also impose restrictions in the selection of large unit size.

Before deciding upon the type to be installed (in the head range 1200 to 1800 Ft) Francis and Pelton turbines should be compared from several points such as cost of in-



POWER STATION.
PLAN.



POWER STATION.
TRANSVERSE SECTION.

FIG :- 5(a) AMBULAO PROJECT

stallation, operational considerations and aspects of repair and maintenance. Francis turbines are more economical because of the higher speed at which they can be run. Increase in speed reduces the dimensions of the unit and therefore less space requirement. Deep setting is possible in the a case of Francis turbines, whereas the setting height of the Pelton wheel ie. 0.60 to 3 metres is lost (49).

Economical studies have to be conducted to decide whether turbine setting (Francis or Pelton) in Underground stations should be horizontal or vertical. Horizontal settings has been adopted in number of U.G.stations in Italy. For U.G. pumped storage schemes, horizontal units are considered better. Horizontal shaft setting (reaction turbines) involves noteworthy advantages (49, 15). The width as well as the height of the machine hall can be reduced as compared with vertical setting, without the necessity of increasing the spacing of units. The volume of concrete required for the substructure is considerably less. Time of construction can be abbreviated as it is possible simultaneous installation of the generator and turbine. Besides a shorter construction period results also from the reduced volume of rock excavation and concrete work. Hydraulic losses are minimised since straight, axis conical type draft tube can be employed instead of the elbow type used with vertical setting (Fig.5a) Horizontal setting is not without drawbacks which can be summarised as below:

1) The overhead travelling crane in the machine

hall cannot be used for mounting the turbines and therefore separate lifting device is required. Therefore this arrangement is hardly suited for very large machines;

ii) Horizontal setting is not favourable to frequent use of the generating units as synchronous condensers for improving power factor and voltage regulation. These disadvantages were greatly outweighed by the advantages of horizontal shaft setting at Ambuklao Project where 3 Francis units each of 25 MW have been installed (15)

For high head developments, horizontal impulse turbines have been widely used in Europe. However, horizontal setting is not adopted when large units are to be installed. For the Haas Project (10) vertical shaft multi-nozzle Pelton turbine (92000 H.P. units, six nozzle, 400 rpm. and 2444 Ft head) was selected over horizontal shaft unit for a number of reasons: higher efficiency, fewer units, nozzles may be set closer to tailwater, higher speed and therefore smaller turbine and generator, effective energy dissipation into wheel pit walls and less water hammer hazard to the penstock from a single faulty nozzle operation because each nozzle controls a small part of the flow. Nicolaou (10) has drawn some specific conclusions by relating type of installation to volume of underground power station excavation. These are-.

i) For Pelton turbines, horizontal installations require from 15% to 40% less volume for power station excavation than the vertical installations;

ii) For Francis turbines, horizontal installations require slightly less volume for power station excavation than the vertical installations;

iii) For heads between 1000-1500 FT, Francis turbine installations require from 30% to 80% less volume for Power station excavation than the Pelton turbine installation.

iv) The range between 1000-1500 FT. has usually been considered as the province of Pelton turbine. However, with modern turbine design and with the savings in underground excavation at high heads, there has been a trend towards the adoption of Francis units for heads upto 1700 FT.

From the above, it is clear that selection of turbines and their setting in Underground stations deserve special consideration.

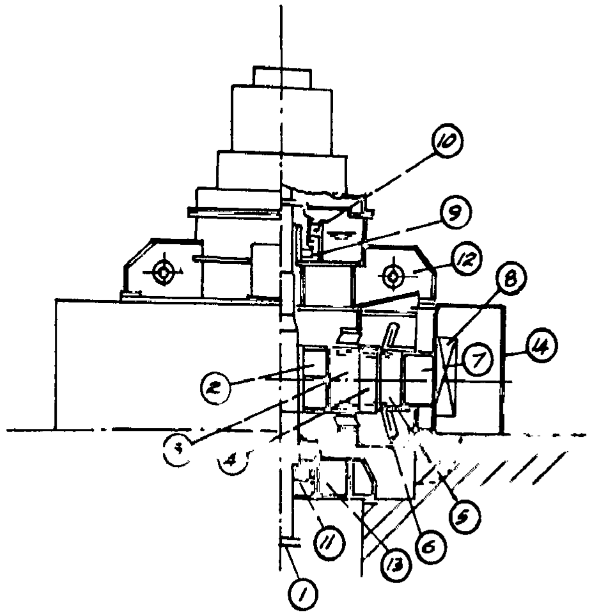
4.2.2 Increases in Runner Speed: Rotational and specific speeds are both steadily on the increase. By providing a large number of nozzles around a vertical Pelton runner, smaller nozzles can be used for a given output and head than in horizontal shaft machines which are restricted to two nozzles for each runner. Use of high quality material, improved hydraulic designs, better mountings, efficient bucket tips and edges - all these led to produce smaller and lighter buckets which can be set on a much smaller mean diameter, with the result that higher running speeds can now be achieved. In 1953, a twin-nozzle Pelton running at 428 rpm was required to develop 62000 HP under 700 Metre head. Today the solution for this would be a vertical single-nozzle turbine running at 600 rpm. In the case of runner design, a 24 bucket runner designed for a given specific speed in

1947 would only have 18 buckets today (57).

By increasing the specific speed of the units not only will there be an economy in the cost of the generating plant but also in the size of the powerhouse cavity (66). However, it will be necessary to ~~x~~ set the distributor centre line lower in order to avoid excess cavitation. Increase in speed will also result in lower WR^2 of the unit which, in turn will affect the speed rise of the machine and also the stability of the governing system. The lower setting of units results in some extra expenditure due to greater depth of excavation, but in underground installation where the entire excavation is already below the ground, there will not be any extra expenditure on this account. There will however be extra expenditure due to increased lengths of approach tunnel, tailrace tunnel and penstocks due to deeper setting of power house. These aspects have to be examined to study whether it would be desirable to increase the speed of the units from economic and operational considerations.

4.2.3 Dismantling Methods: Three methods have been adopted for dismantling the runner. These depend on the size of the power Unit. In the early days, dismantling was done from intermediate floor ie., between the turbine and alternator but later on preference was given to dismantling from below than to dismantling through the alternator. For dismantling from intermediate floor, a very long turbine shaft is required and this increases the vertical height of the cavity. Aldeadavila station (6 x 170,000 HP Francis Units) is designed along these lines (17). This arrangement is adopted for high head machines nowadays, for as the high speeds at which they run require comparatively small alternator rotors, the alternator casings are too small to allow the turbine parts to be withdrawn through the stator assembly. Dismantling from underneath the unit may not also be possible as the runner exit diameter is much smaller than the inlet diameter. Current practice in France is to dismantle the runner from underneath the unit as this method required much less time than by other methods. Dismantling through the alternator involves removal of the rotor and this method is not popular nowadays. Among the three methods mentioned above, dismantling from intermediate floor and from underneath the unit are prefered in underground layouts.

5.2.4 Turbine Valves: Two types of valves are in common use viz. Butterfly valves and spherical valves. The advantages claimed for Butterfly valves are a minimum of powerhouse space required for application, low mechanical friction, reliable operation and low cost. ~~These points easily override, reliable operation and low cost.~~ These points easily override the disadvantages and leakage and head loss and have made it by far the most popular under low and medium head. Compared to the Butterfly Valve, the objections to the spherical valve are the cost and complications of control necessarily associated with the moving seat. The advantages are compact mounting dimension, very low head loss and low mechanical friction. It has definite superiority in freedom



1. MAIN SHAFT.
2. ROTOR SPIDER.
3. ROTOR YOKE.
4. ROTOR POLE.
5. STATOR CORE.
6. STATOR WINDING.
7. STATOR FRAME.
8. AIR COOLER.
9. THRUST BEARING.
10. UPPER GUIDE BEARING.
11. LOWER GUIDE BEARING.
12. UPPER BRACKET.
13. LOWER BRACKET.
14. AIR DUCT.

FIG: 5(b) SECTION DRAWING OF VERTICAL SHAFT WATER TURBINE GENERATOR.

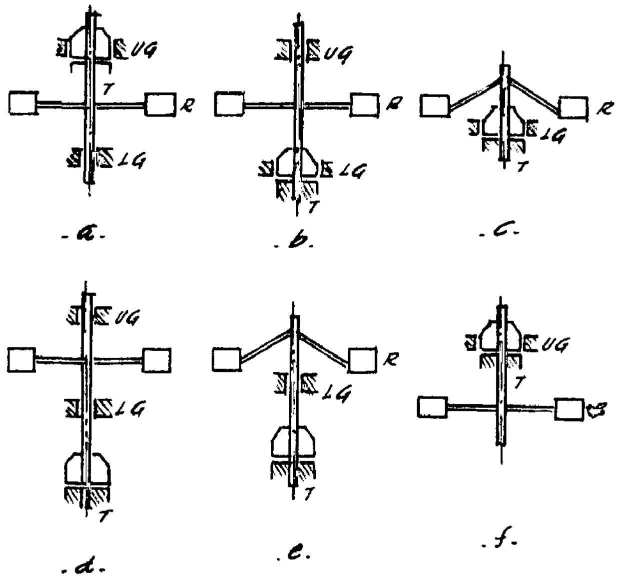


FIG: 5(c) VARIOUS KINDS OF TYPES ACCORDING TO LOCATION OF BEARINGS from leakage and this makes spherical valves more favourite for medium and high head application (49, 5). Beyond 300 ft. head, a spherical valve is most suited for installation as a turbine shut off valve.

5.2.5 <u>Servomotor</u>: Proper selection of servomotor reduces space requirement. The internally mounted servomotor for the jets in place of the usual externally situated servomotor is proving quite successful for Impulse turbines of medium and high output on account of the saving in space and the better hydraulic conditions created for the jet.

5.3 <u>GENERATOR</u>:

Designing large-scale generators of the horizontal shaft entail difficulties due to the deflection of the shaft because of the rotor's weight and to the large deformation of the stator frame because of the weight of the stator cone. However, if the vertical shaft type is constructed, these problems are eliminated and it is possible to adopt a split stator construction in view of the transport limitations (49, 21).

Vertical type water turbine generator is equipped with guide bearings and a thrust bearing. Figure 5 b & c shows typical models (a) is the standard model (b) is the semi-umbrella type in which the thrust bearing is located beneath the rotor, (c) is the umbrella type and (d) is the type in which the thrust bearing is located on the head cover of turbine which is a desirable location for Kaplan turbine. Type (e) is umbrella type of (d) model and almost all Kaplan turbines belong to the (d) and (e) types. Type (f) is the mono-block type in which there is no lower guide bearing. This type is mostly adopted in vertical shaft Pelton turbine generators (20). It is quite clear from the figure that Umbrella type generators are quite advantageous in lowering the height of the power house and minimising building costs. Umbrella type generators are about 15 to 20% lighter than those of the conventional type. The crane capacity can also be reduced when Umbrella type is used. The rotors of Umbrella type differ little from those of the conventional type. There is no need to extend the shaft above the rotor because there are no thrust bearings and guide bearings above the rotor. Only the spider is attached at the top of the shaft. Consequently the shaft length is greatly reduced and thus the height of the powerhouse is reduced. Since the thrust bearing is located beneath the rotor in order to simplify dis assembling of the generator, a construction is adopted in which the rotor can be taken out without dismantling the bearings. Since thrust bearing is below the rotor, inspection of the bearing is usually troublesome compared with the case of the conventional design in which it is placed above the rotor. Special provision can however be made for inspection without removing the rotor. The modern trend is to use umbrella type generators especially in underground stations. Examples are (i) Haas station equipped with 75,000 kva generators (10); (ii) Okutadami station with 180 MW generators (39).

5.3.1 <u>Generator Voltage:</u> A normal voltage rating of 11 K.V. for the generators is adopted in most of the stations. Voltage rating of even 18 KV has been adopted in certain stations. When large capacity generators are installed increase in voltage rating to 14 KV or 15 KV may lead to more balanced and reliable design for the windings and reduction in cost of the heavy current connections between the generator and the step-up transformer (25). In modern plants, a voltage rating of 13 to 15 KV for generator is usually seen.

5.4 <u>Transformers:</u>

Transformers may be located above ground or underground, in a separate cavity or within the main power house cavity. Economic considerations have been discussed in Chapter VI. Transformers may be single phase or three phase. Three phase units will be larger in size and heavy to handle. Underground stations are mostly equipped with single phase transformers due to transport limitations. In some recent underground stations heavy three phase transformers have been used. At crauchan (81), 275 KVA three phase transformers have been located in an underground cavity. In order to comply with a gross weight limit of 80 Tons on vehicles approaching the station and in view of the loading gauge limitations imposed by the access tunnel, each transformer after assembly and test in the works was dismantled and re-erected in its chamber. The problems involved in such a procedure are (1) to rebuild the core to ensure its characteristics and (ii) to regain the specified insulation strength.

When the transformers are located within the main powerhouse cavity, transformer noise is apt to be excessive. To reduce the noise to a predetermined value, some precaution is to be taken in the transformer construction and erection in addition to providing sound proofing coating for the walls. At Okutadami station (33), the precautions taken were: (i) Flux density was reduced about 10% below standard value; (ii) tank side plates were reinforced to suppress vibration of the tank; (iii) Transformer proper was placed on a spring device with vibration-proof rubber inserted between tank and base and (iv) the air gap between each phase was filled with a muffler and applied with a blind cover to eliminate the sound produced from that part.

selection of transformers is done with utmost care so that the dimensions and weight can be reduced. Water-cooled transformers are preferred for underground location so that the dimensions and weight are reduced. By providing cable connections both on the H.T. and L.T.side without bushings (external), the height is reduced. For the transformers at Okutadami U.G.station, a special type of bushing called Elephant bushing is used for transformers (33). It is of the indirect type in which the cable head is not connected directly to the transformer winding but a connection box is provided to separate it from the transformer by means of a wall bushing in oil. In this indirect system, all bushings are again in oil and size is greatly reduced and reliability is increased as compared with large bushing and cable head exposed in air.

5.5 CABLES:

The various methods for taking power cables to the switchyard located outside have been discussed in Chapter VI. Selection of cables and their erection assume importance when the station is located deep underground. For connections to the H.T.control equipment, pressure oil-filled or gas filled cables are used with proper gradients and pressure regulators. When cables are laid in steep gradients it is likely that high internal pressures may be built up at the bottom causing longitudinal extension of the sheathing. It would therefore be necessary to provide transverse and longitudinal reinforcement for the cables. At Cruachan (81) the transformers are connected to the switchyard by oil filled cables passing through a combined ventilation and cable shaft which rises 1080 Ft. vertically. The single core oil-filled cable (275 KVA) used has a reinforced lead sheath designed to withstand an internal pressure 45 lbs/sq.inch. Additional strength is imparted by Aluminium wire armouring and the entire cable is encased in PVC sheath. Special installation technique has been adopted when joint was made.

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CHAPTER - VI

DESIGN CRITERIA AND PRACTICES.

6.1 GENERAL

With the site of dam established, the scheme can be developed either as 'head' development or 'tail' development or 'intermediate' development. In every case, the power water may would play the most important part in the selection of the power house site. As a result of the advancement made in equipment arrangement, handling facilities, construction techniques and excavation methods, a wide variety of economical plant layouts have been developed and it is axiomatic today to say that the adoption of the shortest feasible waterway will along develop into the most economical project layout. Therefore the selection of sites can well be done with the thought that the power plant can be economically adopted to almost any feasible waterway. The greatest economy with the studies is achieved by the use of one of those five waterways given under the hydrodynamic classification of underground station layouts.

6.2 REQUIREMENTS OF UNDERGROUND LAYOUTS:

Underground plants comprise all the elements found in the ordinary types of developments with the power plant on the surface. The intakes, the pressure tunnel, the underground surge tank and the subterranean valve chamber are structures to be frequently encountered at surface stations as well. On the other hand, the sloping or vertical underground pressure shaft conveying water to the turbines, the underground cavity housing the generating machinery and auxiliary equipment and tailwater tunnel - occassionally with downstream surge chamber - are essentially characteristic of underground developments. Besides these, underground projects present certain problems associated with the transmission of power, ventilation and access and suitable provision has to be made in the layout. The design of the underground layout, prior to effecting entrance to the working area, is made in principle only. The general design criteria and practices followed for the selection of various sites are discussed below:

6.3 INTAKE:

Fundamentally there is no difference between the intake works for the classical type of power projects and the underground type. As in surface plants, intakes of underground plants can be located either within the body of the dam or upstream of the dam. The basic design requirement for an intake is to have a stream lined intake tube which will accelerate the water from the trash racks to the junction of the intake with the supply tunnel or pressure shaft, keeping the hydraulic losses to a minimum. This requires properly designed transition. If the intake is located a little upstream of the dam to suit the topographical and geological conditions, intake shafts or intake towers are generally adopted. In the case of shaft intakes, the entrance of shaft is located above the highest storage level of the reservoir. The pressure tunnel intersects the shafts by

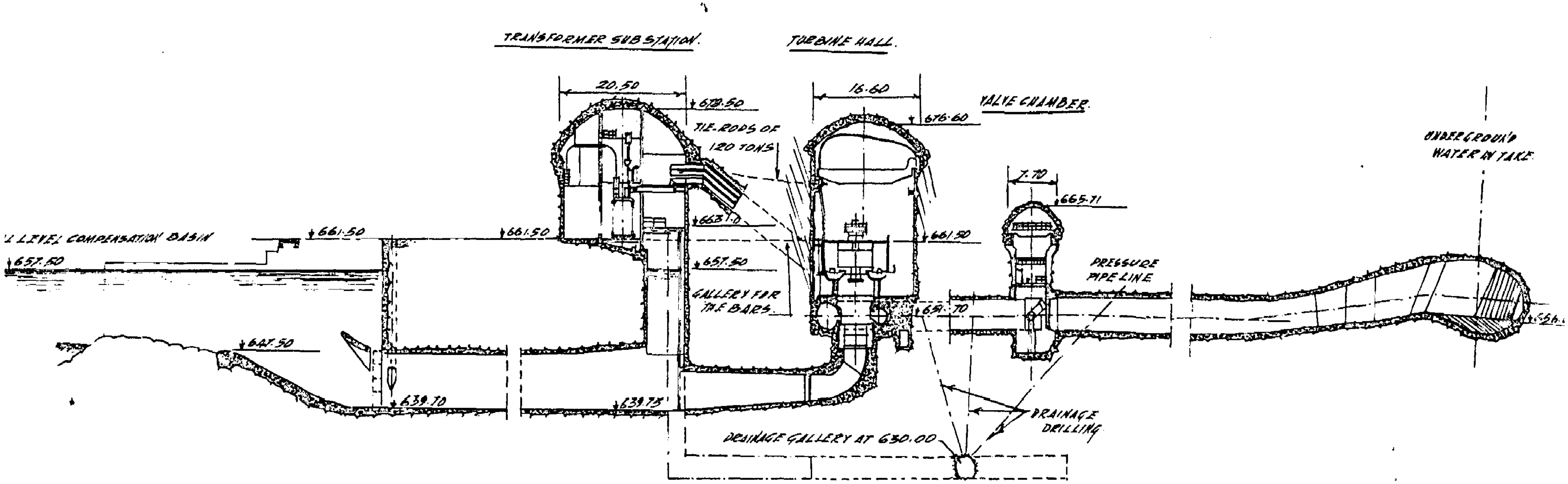


FIG: 6(a) LONGITUDINAL SECTION OF THE INSTALLATIONS CONNECTED WITH THE FALL.

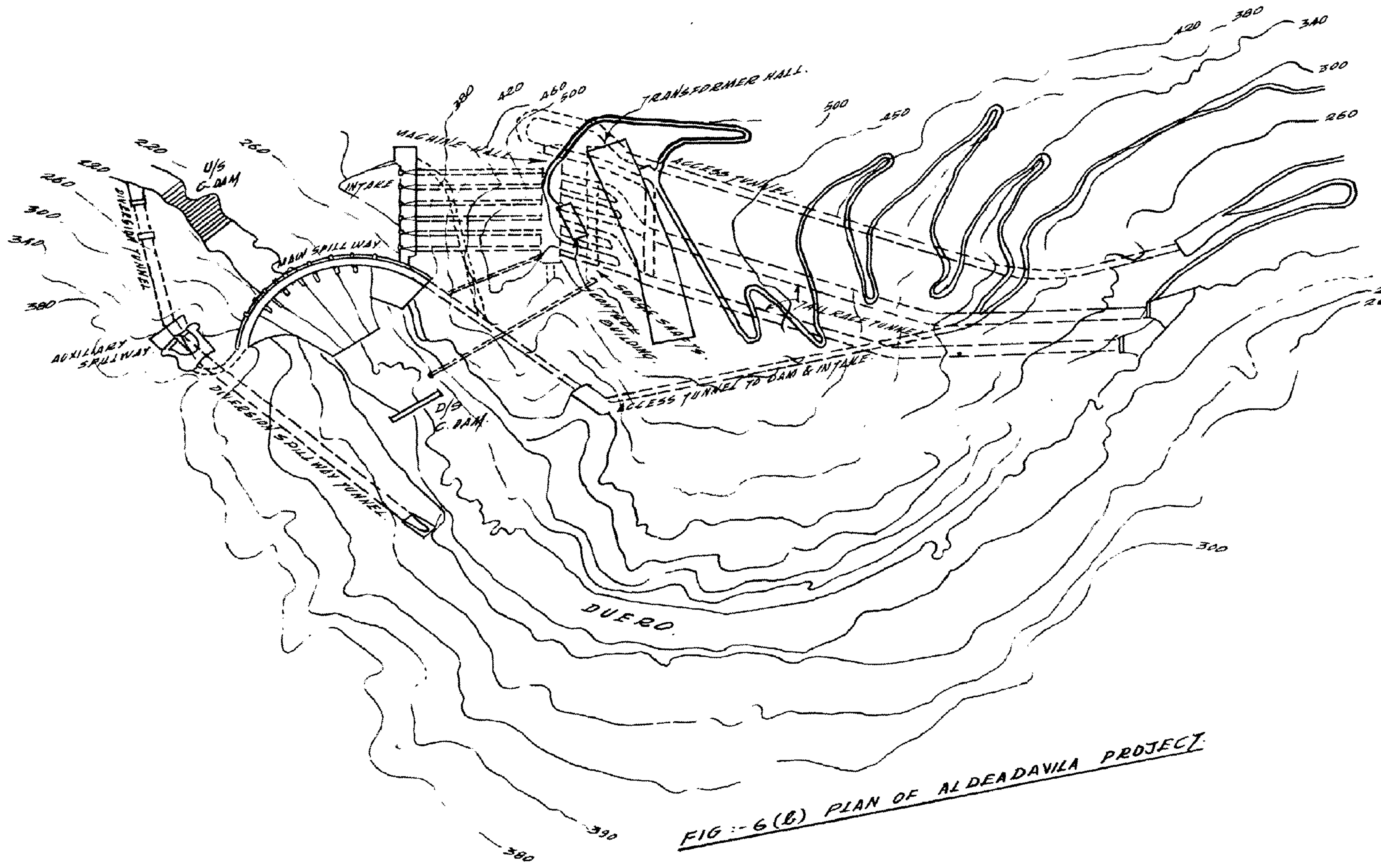


FIG :- 6(B) PLAN OF ALDEADAVILA PROJECT.

means of a closed pipe section incorporating the inlet valve. The shaft can be eliminated if the fluctuations in the reservoir is small and the conduit valve, together with the appurtenances, is located in a chamber directly accessible from the surface. In the case of intake towers, flow into the tower is generally controlled by a number of small gates instead of a single gage in intake shafts. High intake towers in earthquake regions are however not desirable. For the Ambukla underground project located in an earthquake region in Phillipines, a 380 ft.high intake tower reaching above the maximum reservoir level was initially planned. However, to safeguard the tower from possible earthquake shocks, the height of the tower was limited to 184 ft. below the full reservoir level thereby sacrificing intake accessibility and eliminating intake closure gates but achieving a considerable saving in project cost. In Serre-Poncon Underground development, the very large section of the intake (Fig.6a) gallery enabled the engineers to dispense with the employment of a surge chamber. For effecting minimum hydraulic losses it is desirable to align the pressure shafts (if geology permits) perpendicular to both the intake and the power plant centre line. An example being the intakes of Aldeadavila project (Spain) located in a vertical plane parallel to the centre line of the underground station (Fig.6b) stability of the hill slope near the intake area is a major consideration. Levels of intake should be fixed in such a way that under minimum draw down conditions air is not allowed to pass through the tunnel.

6.4 POWER TUNNEL:

Power tunnels are normally required for 'tail' developments and 'intermediate' developments. Mosonyi (49) classifies Power tunnels into three categories: (i) Low pressure tunnels with H lower than 5 metres (ii) Medium pressure tunnels with H from 5 metre to 100 metres and (iii) High pressure tunnels with H higher than 100 metres.

Design of power tunnel comprises (i) determination of tunnel diameter and grade; (ii) Assessment of rock loads likely to occur due to roof yield and provision of adequate lining; (iii) design of tunnel lining to withstand internal pressure and (iv) provision for drainage arrangement to relieve external hydro static pressures and elimination of seepage in the vicinity of important structures.

The tunnel diameter is primarily decided on the basis of economics and is a problem of balancing between revenue loss due to tunnel friction, interest charges on capital cost of tunnel, annual maintenance cost and capitalised abatement. The sum of all these costs must be a minimum for the most economical size of tunnel diameter. Other considerations for fixing diameter are (i) space requirements for equipment to be used in tunnelling, (ii) time factor considering difficulties that may be encountered in excavation of rock and (iii) availability of construction machinery. According to Mosonyi, minimum size of circular tunnel is at about 1.8m while rectangular cross sections should not be smaller than 2 x 1.8m

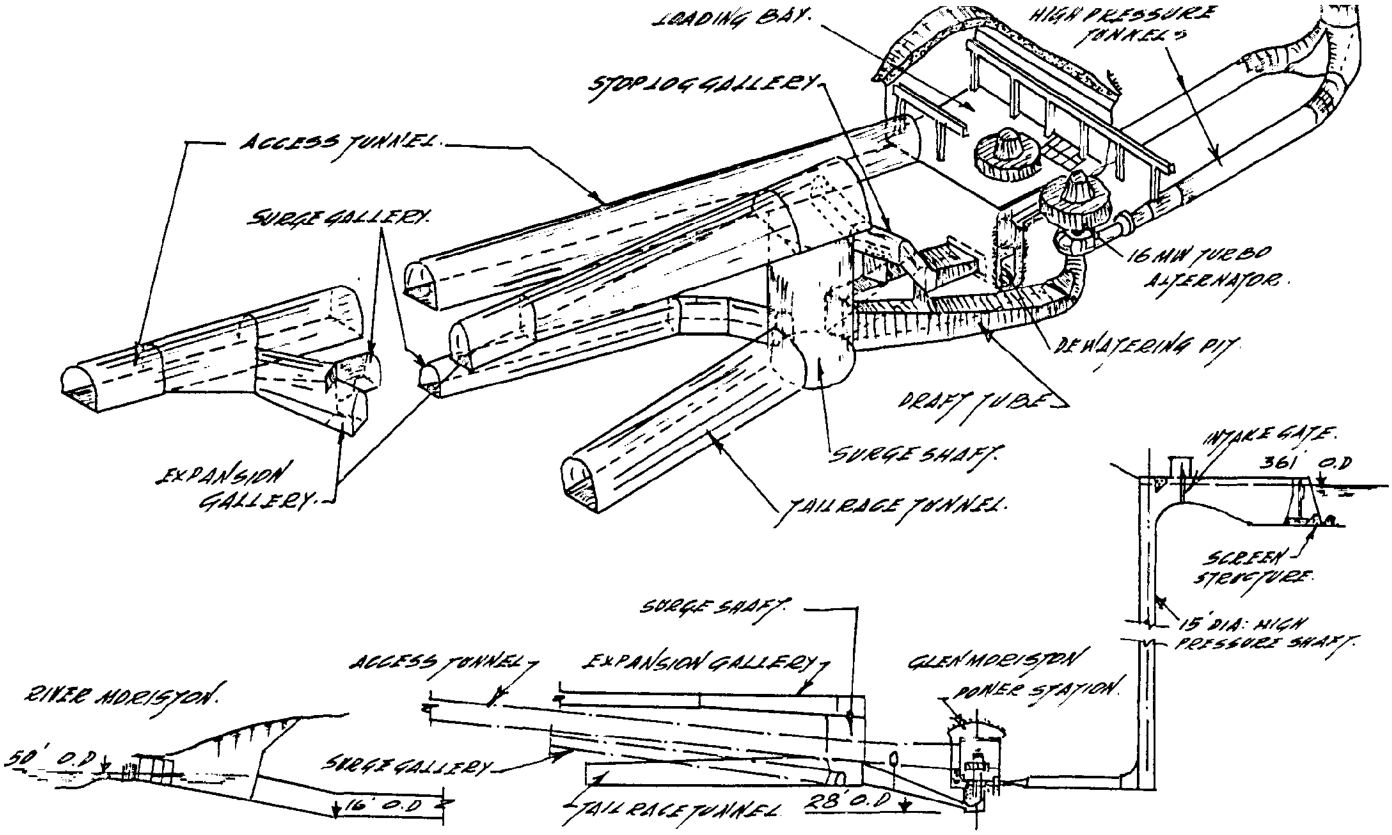
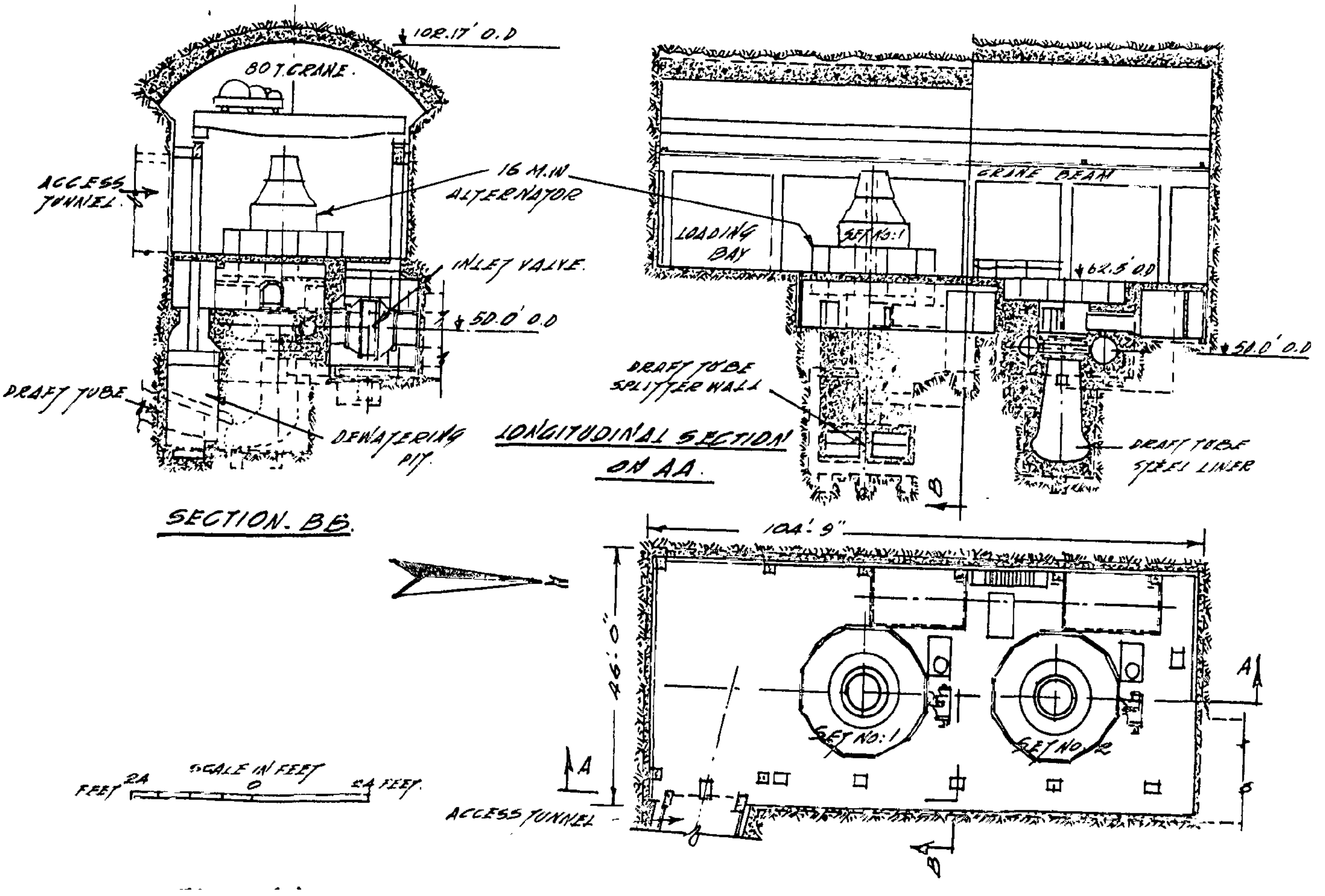


FIG: 6(c) GLENMORISTON STATION. (LONGITUDINAL SECTION OF WORKS)



Power tunnel upto 70 ft. diameter have been constructed in Granite in Norway. Velocity allowable is also a major consideration. Limit velocities suggested by H.Press are:

Very rough rock surface	1 to 2 m/sec.
Trimmed rock surface	1.5 to 3 m/sec.
Concrete surface.	2 to 4 m/sec.
Steel lined	2.5 to 7 m/sec.

From hydraulic considerations it is better to provide circular or horse-shoe shaped sections for power tunnels. The grade of the tunnel may be fixed after taking into consideration the minimum water level requirement in the surge chamber and after ensuring minimum rock cover in its alignment if possible.

Normally from point of view of minimum power loss, one tunnel is always to be preferred over two or more tunnels as it will give least friction losses for the same equivalent area of cross section. Where, however the magnitude of discharge to be handled requires an excessive value of tunnel diameter for the particular types of rock strata, it may be advisable to provide two or more tunnels and sizes of these can be determined from economic studies as mentioned earlier.

The decision as to whether the tunnel should be lined or unlined is purely a matter of economics. Some recent tunnels eg. Upper Vinstra of Norway and Glen moriston of Scottland (Fig. 6c) are mainly unlined. Only about 30% of the Kemano tunnel of Canada is lined. The higher coefficient of friction with an unlined tunnel may dictate a substantial increase in effective diameter which must be weighed against the cost of lining a smaller tunnel. Low pressure tunnels can be left unlined except for visiblefissures which may have to be sealed suitably. Lining might be a necessity if the tunnel is through fractured rock. A water tight lining is normally provided for tunnels operating under medium and high heads. With the operating heads exceeding a certain limit, all voids between the rock and the lining should carefully be eliminated and if this is not achieved, lining will be subjected to internal pressure and the required pressure thereof would be enormous. If the tunnel is lined or unlined or if the lining serves only for water sealing purposes, the permissible internal pressure is determined by the depth of over burden and by the quality of rock strata. Tunnel lining may be of concrete or with steel. When the tunnel is not subjected to a very large hydrostatic pressure (in the order of 10 atmospheres) reinforced concrete pipes, either prestressed or not, may be adopted. The alternative with steel piping is always favourable, especially in view of the higher safety of operation (41). If lining is designed to take only part of the internal pressure, the annular space between the lining and th rock is to be filled with concrete and grouted. In order to relieve external pressure exerted by seepage water when the tunnel is emptied suddenly, suitable drainage is to be provided.

A theoretically ideal tunnel has satisfactory hydrau ic characteristics under all working conditions, such as the hydraulic gradient must be sufficient to give the required discharge without causing excessive velocities or excessive losses and is located on the shortest straight line. The tunnel should be under sufficient rock cover - a rule of thumb is 80% of static head of water. Consideration should also be given to the location of intake and exit portals and availability of intermediate access points for construction activity.

6.5 PRESSURE SHAFT:

In 'head' developments, mostly, the power plant will be located in abutments. For such location, separate pressure conduits for each turbine are ordinarily used (37, 83, 49). This arrangement would prove economical since manifolds and guard valves can be eliminated (83). On the other hand, the high head or tail developments and intermediate developments normally have one or more pressure conduits from the upstream surge chamber to the power station and these conduits branching off to the individual turbines just upstream of the power station chamber.

Pressure shafts can be either vertical or sloping. Vertical shafts offer the shortest route for conveying water to the turbines. Although the tailrace tunnel becomes longer, a saving in cost is still attainable since specific costs of the tailrace tunnel are comparatively lower than the pressure conduit if the latter must be lined. An economic analysis should then obtain the minimum total cost of the two opposing elements viz. the length of the pressure shaft on one side and the length of the tailrace, access and other conduits to power house on the other. Such an analysis will be the basis for determining the value of the penstock slope (41). Other benefits with shorter conduits are: (i) the more favourable turbine regulation (ii) the omission of relief valves as at Glen Moriston and Ceannacroc stations (62) and even the upper surge tank (Ex.Poatina station). (iii) the construction of very steep shafts is more cumbersome than that of vertical ones. Shafts, unless of excessive depth, should preferably be constructed vertically (49) if the above mentioned advantages can be attained. Vertical shafts have been constructed to: (i) 255m shafts (steel lined) at Hanabanilla Project, Cuba (39); (ii) 660 m deep R.C.lined shaft at the San Giacomo Power station, Italy (iii) 410 m deep shaft with freely supported penstock at the Villa Santa Maria Power Station (Italy). When horizontal setting is adopted for turbines vertical shafts would be more economical in the case of low head stations as at Guayabo Power station in South America. As such no limit can therefore be given and each x case has to be investigated independently.

In the case of sloping shafts, the economical angle is determined theoretically from considerations such as (i) Quality of the rock and geology of the terrain (ii) Area of the conduit and (iii) Construction method and (iv) slope of terrain. Removal of muck through shafts inclined at less than 35^{0} is considered expensive while in shafts steeper than 45^{0} the additional cost of protection against damages inflicted by the spoil emerging from the portal would be high (49). Pressure shafts at the Maggia System, Italy, are constructed to a slope of 67 to 70% corresponding to an angle of 34 to 35°. A slope steeper than this, sloping at 85%, ie. almost 40.5° is adopted at Santa Masseuza Power Station for reasons of convenient removal of spoil. For economic reasons, the slope of the pressure shaft section of Haas Project is kept at 65° (10). In Norway, generally slopes vary between 36° and 39° yet even 45 to 46° shafts have been constructed (49). In Italy, the slope generally adopted has a value from 70 to 85% (41). A Slope as flat as 32° was adopted for the Vianden Pressure Shaft in Luxembourg for economic reasons though there was some excavation problems (73). For the Clachan Station, a slope of 39° is adopted for a variety of reasons viz. (i) the inclination was just a little in excess of the natural angle of repose of rock spoil so that the spoil was more easily removed (ii) the inclination was almost normal to the planes of cleavage in rock so that overbreak is less during excavation and (iii) Along the pressure shaft, the weight of over burden measured vertically above the shaft was sufficient to balance the water pressure at any point (29 - discussion by Mr.J.A.Banks). Sometimes construction adits provided for pressure shaft excavation are subsequently used as permanent drainage galleries. With this provision, the assumed diagram of external water pressure can be interrupted at the elevation of the drainage gallery and from this point downward the pressure starts to build up again from zero and taking this aspect in the designs, savings can be effected in the cost of lining. Economical construction adits can be provided only if the sloping shaft is pushed towards the slope of the hill. The above method of relieving external pressure was successfully applied in the design for the 48° Kemano pressure shafts (36, 49) and 45° Koyna Pressure shaft (40).

The choice between vertical shaft and inclined shaft is purely based on the overall economics of the project. For example in Gando Power Project, a study was made of plans for vertical and inclined shafts. Results of comparison did not show much difference in construction costs. However inclined shaft was adopted for the reason that the 470 m vertical shaft would take about 4 to 5 months more time than the 745 m inclined type (64).

NUMBER OF SHAFTS:

In determining the number of shafts, economical considerations require, in general, the smallest possible number of shafts. The excessive diameter and thickness of the steel lining required in case of large demands for turbines and high heads, however, impose restrictions upon the size of the shafts, consequently necessitating the increase of their number. Degree of weldability is an important factor in this respect. Since the thickness of the steel lining depends, under identical rock conditions, on the diameter and the head, the maximum practicable size of the pressure shaft can be characterised by a given value of its Power capacity (49). The power capacity obtainable by a single pressure shaft is rated as per today at some 600,000 H.P. (Kemano).

DIAMETER OF SHAFT:

There is no general rule in determining the effective cross section of the shaft. Permissible velocities in exposed penstocks 3 m to 8 m will apply to pressure shafts also. The clean diameter of shaft is determined individually for each case by economic analysis. Increasing the section involves higher construction costs. For computing economical diameter of steel lined vertical shafts at depths from 30 m to 80 m G. Isakksson gives the following relationship $d = Q^{0.40}$(m) where Q is the maximum discharge in cumecs. According to Mosonyi, the above formula can only be applied to preliminary estimates and to conditions similar to those encountered in Sweden (49). The economical shaft diameter decreases with increasing heads. With increasing wall thicknesses the diameter of the steel lining is preferably reduced in oneor two occasionally three stages.

DESIGN CONSIDERATIONS:

Pressure shafts can be lined or unlined. The criteria in omitting lining are:

i) The rock should be able to resist expected pressure;

ii) rock through which the shaft passes should be sound in order to eliminate seepage;

iii) The weight of rock cover should be considerably in excess of the hydrostatic pressure.

Regarding point (iii) experimental data after

R. Heggstad regarding pressure shafts in Norway - 11 out of 12 pressure unlined shafts, the weight of rock cover is 1.7 times the value of hydrostatic pressure. For the 300 metre pressure unlined shaft of Tafjord III station the rock cover exerts a 2.8 fold counter pressure. Sometimes pressure shafts of low head and medium head station are lined for no other reason than to prevent seepage. In poor quality rocks, even low and medium pressures shall be borne entirely by the lining which can be reinforced concrete or steel lining. For high pressure shafts reinforced concrete or steel lining embedded in concrete or prestress concrete lining are provided. The French claim that prestressed concrete liners for pressure shafts are better than steel liners because prestress concrete liners are stiffer (Water Power 1950).

Placing of steel penstocks in the tunnel may be carried out in two ways; (i) the penstock pipe is laid on supporting saddles inside the tunnel and (ii) the interspace between the penstock and the rock is filled with concrete. A combination of penstock laid on the mountain slope and steel lined pressure shaft is sometimes adopted as in Haas Project (10) and Huinco Station (61) Refer.Fig.6m.

Though the length of conduit can be reduced, the free penstock in an inclined or vertical shaft is a less frequently applied solution as the penstock is to be designed for the entire hydrostatic pressure including water hammer. Disposal of seepage water is also a major problem. Some examples of penstocks laid in tunnels are: Santa Giustna (41) Gando Project

(64) Fig.6t Salfane station (head 1474 m) and Villa Santa Maria in Italy. The 2.1 m dia. 410m penstock of Villa Santa Maria station is surrounded by a 3.55 m diameter concrete lined service shaft with a winding stair in the annular space leading down to bottom and the shaft is made accesible by horizontal adits. With the interspace filled with concrete, it is possible to transfer to the rock part of the load acting inside the penstock. By proper attention to design, taking full advantage of the strength of the surrounding rock, selection of optimum grouting pressure and of the economic form of lining, the overall cost can be minimised. Assuming low pressure grouting is fully effective in filling all gaps between steel liner and the concrete encasement and between the encasement and the rock, the % of internal hydrostatic pressure in the penstock which will be carried by the rock can be calculated according to theories of VAUGHAN (77); Jaeger (29); Bliefus (4); Amstutz (2).

In rock of poor quality, only a small, in some cases, practically negligible part of the internal pressure can be transmitted to the rock by a grouted steel lining. Consequently the saving in steel attainable thereby would be insignificant. In rock of good quality, a major portion or even the entire internal pressure can be transmitted to the rock when the steel lining acts only as a water sealing membrane. According to R.Heggstad in sound rocks characterised by deformation modulus between 80,000 to 150,000 kg/sq.cm. 65 to 75% of the internal pressure can be transmitted to rock provided all gaps are carefully grounted.

In the design of steel liners, both external and internal pressures must be considered (18, 13, 36, 77, 4, 43 and 2). External pressures occur either during grouting operation or on dewatering after a prolonged period of use. Often it is the external rather than the internal pressure that determines the thickness of the liner. The decision between a lining of adequate strength and a thinner one supported freely in a tunnel is governed by the relative economics of the two solutions. The problem of estimation of maximum probable external pressure is a most delicate one in the design of pressure conduits.

In the case of vertical pressure shafts the ground water pressure can be almost as high as the hydrostatic value corresponding to the head water level (49). However, with the alignment of the inclined pressure shaft situated below a sloping terrain, the external pressure for any point along the shaft can obviously be not higher than the water column corresponding to the local rock cover. Actual ground water pressure remain in general well below the theoretical upper limit mentioned above. Consequently, pressure shafts located almost parallel to the sloping terrain are not likely to be exposed to excessive outside water loads and the pressure is not likely to increase downward along the shaft (49). Amstutz has proposed an adequate method for calculating permissible external pressure upon embedded steel lining (2).

The maximum probable water pressure will be perhaps with the assumption of complete saturation of rock cover upto terrain level. No theoretical method of general validity can be given for computation of external water load. The degree to which the theoretical upper limit characterised by the depth of rock overburden should be reduced is to be determined for each individual case with a thorough knowledge of the tectomic and stratigraphic formation of the mountain as well as of strength properties of the rock and this aspect has already been discussed in Chapter IV.

The design calculation for the steel plate linings depend on the form of the anchorage, if any. Theories of Issemnn-Pilarski (which are generalisation of the theory of Allievi for pipes) and the formula of Von Mises, Foppl and Amstutz (2) have been used. A detailed account of these theories is beyond the scope of this dissertation.

A method of protection against external water pressure is to impart gweater stiffness to the liningor to have it anchored into the rock (49). Reinforcing hoops or ribs were adopted at Isere-Arc and elsewhere. Electricite-de-France has introduced "hedge hog spine" anchorage which consists of studs welded to the place and this type is used at Rossens (72). It is found that the weight of steel being added to the steel plates for the hedgehog spine anchorage is very small (29). To relieve external water pressure some authorities prefer drainage pipes running parallel to the penstock (Bitto station in Italy). Others provide an inspection gallery adjacent to the penstock as at Gerlos in Austria. Intermediate drainage adit has been provided at Kemano and Koyna. The defect in providing drainage pipes running parallel to sloping shafts is that these pipes get clogges sometimes or becomes sealed when pressure grouting is applied. When these are large, the stress distribution on the lining is no longer symmetrical (29).

To increase the stability of the steel lining against buckling, grouting is quite popular. According to Amstutz and Jullard, the prestress induced in the steel lining by grouting tends to increase the safety against buckling. There is difference of opinion among authorities regarding grouting pressure to be adopted. swiss and Italian Engineers seem to agree with high pressure grouting at $P^1 = 1.5$ P (P^1 is grouting pressure and P is internal water pressure). German Engineers propose $P^1 = 2$ P.

Filling the interspace between the steel liner and rock with concrete is a major problem. According to Guthrie Brown, one of the best means of dealing with the deposition of concrete around steel pipes was to modify the cross section of the rock to be excavated around steel pipes from circular to pear shaped so that better access is obtained for placing concrete.

It is clear from the foregoing discussions that there is difference in opinion among various authorities in the methods adopted for the design and construction of lined and unlined pressure shafts. Though marked advances have been made in this field during the last century the detemmination of the distribution of load between the liner and the rock mass has not been satisfactorily solved. The prime reason for this, may be insufficient fundamental knowledge available about the elements involved.

SPECIAL CONSIDERATIONS IN THE LAYOUT OF PRESSURE SHAFT:

If one or two shafts with their manifolds and guard valves prove more economical for a multiunit plant located in the abutment, it will be better to locate the manifold and guard valves as high as possible on the waterway (Salamonde station). With this arrangement, reduction in cost follows from the smaller operating head for which the valves and manifold must be designed (83).

Special consideration is given in the layout of conduit adjacent to an underground plant for reducing tunnelling costs. The pressure shaft of a multiunit station should preferably be parallel to the longitudinal axis of the machine hall (41, 49). It is usually economical to terminate this shaft in a manifold, the shaft of a multiunit station forming branches from this manifold. (Fig.6d). According to Marcello (41), the most advisable arrangement is to place the distributing conduit to each turbine at such an angle in the space so as to reach the machines with flatter angles and shorter length of piping. Smaller angles lead to longer connecting pipes in some cases to larger Caverns (for instance when the meergency valves are located in the machine hall itself.) Therefore the angle of the manifold branches affects the overall construction costs. From structural considerations it would be preferable to align the distributing pipe at right angles

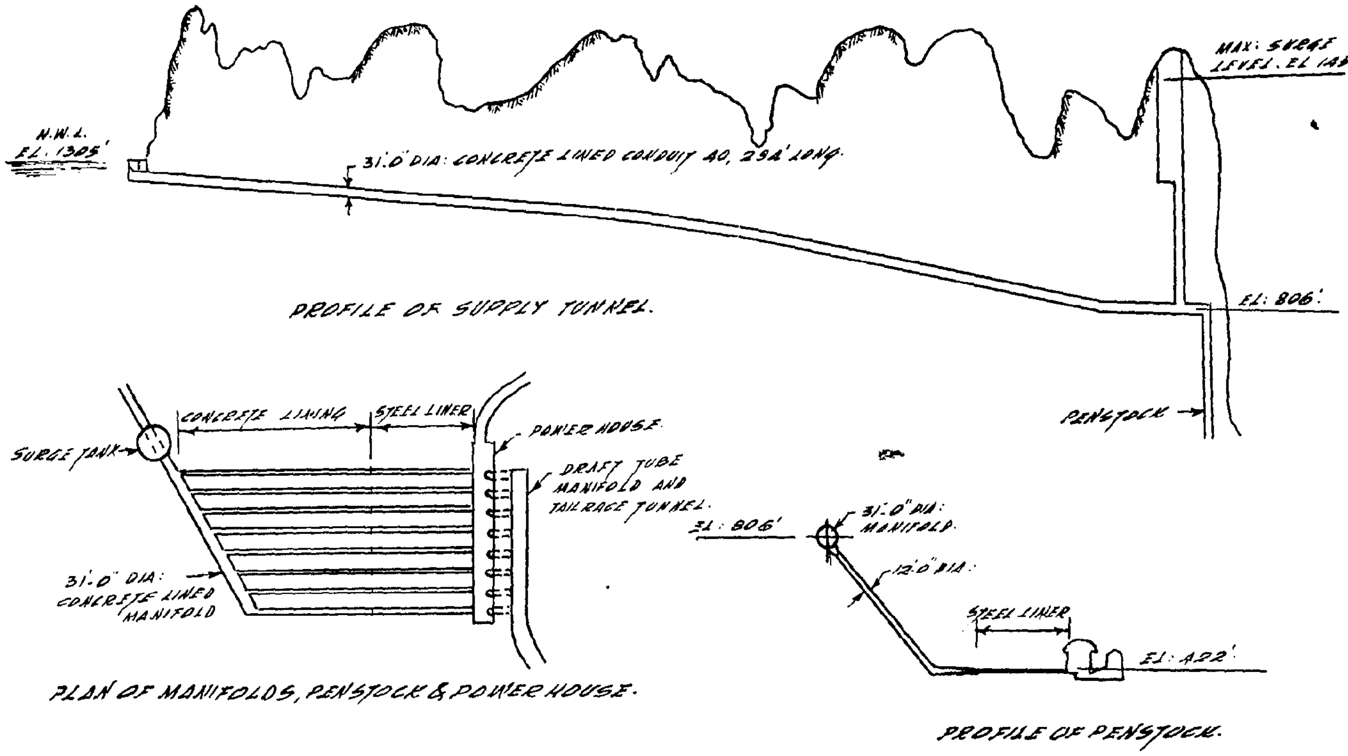


FIG: 6(d) PLAN AND PROFILES OF BERSIMIS No: 1 DEVELOPMENT.

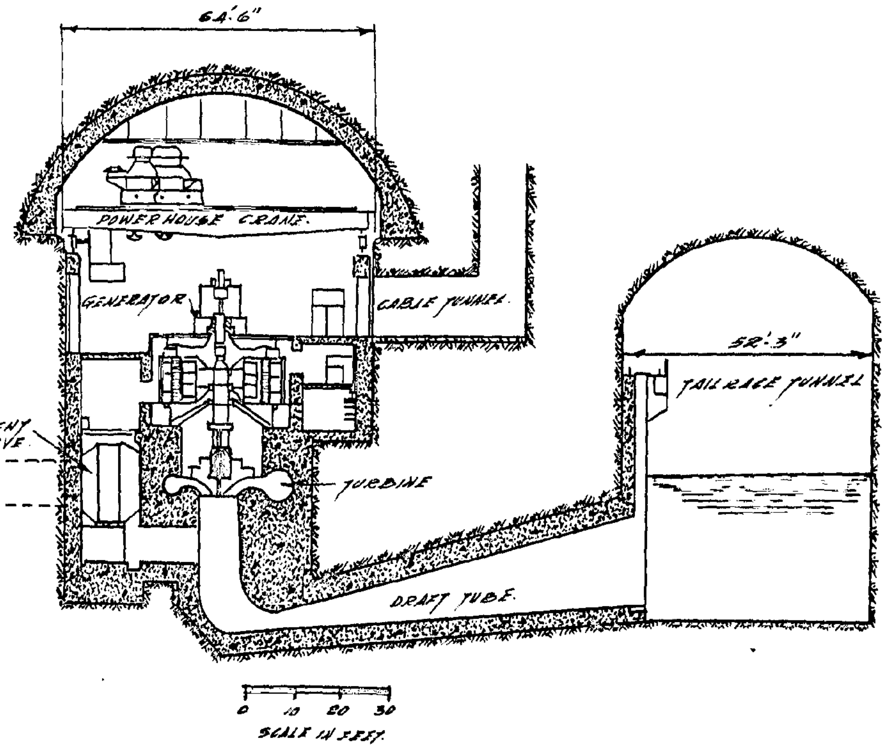


FIG: 6(d) CROSS SECTION OF BERSIMIS No: 1 POWER HOUSE AND TAILRACE TUNNEL.

to the powerhouse cavern. In the case of horizontal axis units, right angle becomes a necessity for example, Villa Santa Maria station in Italy and Stalden station (Fig.6c) and others. Whenever the orientation of the machine hall fixed by other considerations happens to be within 30° to 90° to the direction of the penstocks (in many cases) it is evident that the connecting pipes will come at right angle with take off from the main shaft at 30° to 45° (Fig.6d & 6f).

The hydraulic losses greatly depend on the take off angle. With reference to the extensive loboratory work by Professor Thoma in Munich, losses increase with the angle, slowly upto 60° and much faster up. Take off angle generally varies from 45° to 90°. From velocity considerations, two arrangements may be used. One is that the velocity is the same everywhere, the diameter of pipes varying with the flow and the other with the main feeder having a constant section. The use of a constant manifold diameter reduces tunnelling costs and the standard transition from manifold minises cost of form work. The choice of the optimum layout depends on economical studies taking into account of head losses and the civil costs and backed up by practical considerations.

6.6 SURGE CHAMBER

The pressure tunnel upstream of the surge tank is not usually designed to withstand high pressure. A surge tank located at the head of the pressure shaft will be the best protection against water hammer penetrating the tunnel. A surge tank located as close as possible to the power station makes

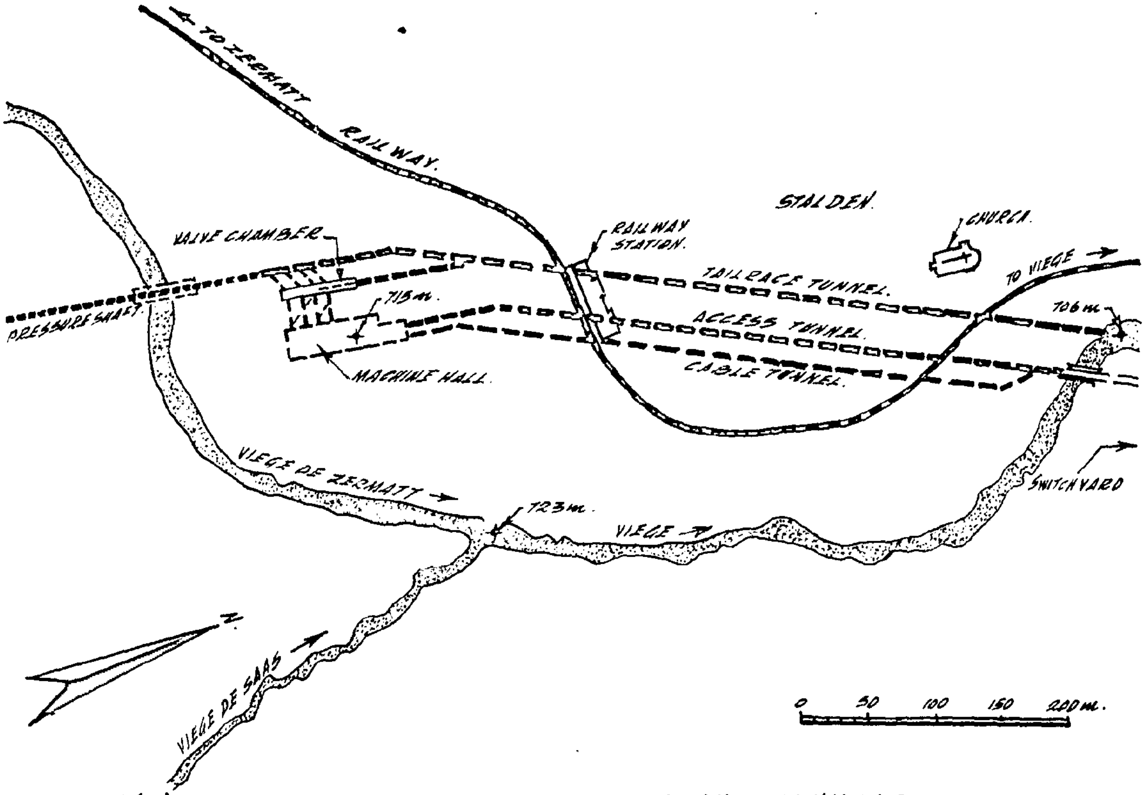


FIG: 6(e) SITE MAP OF STALDEN STATION AND TUNNELS.

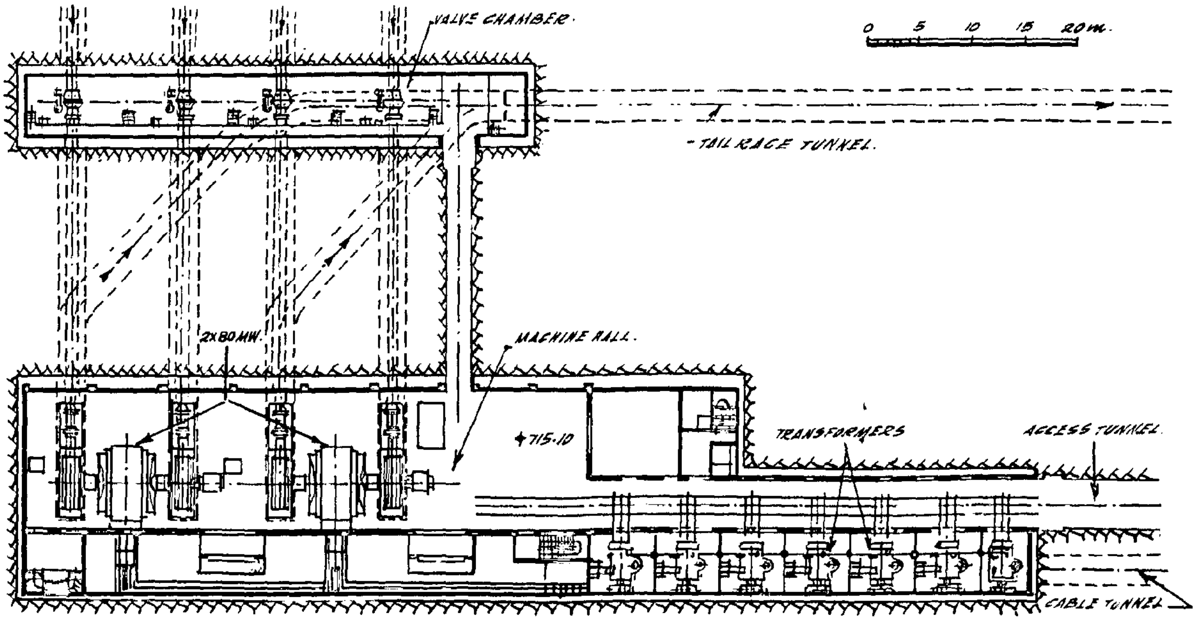


FIG: 6(e) PLAN OF STALDEN UNDERGROUND STATION.

to the powerhouse cavern. In the case of horizontal axis units, right angle becomes a necessity for example, Villa Santa Maria station in Italy and Stalden station (Fig.6c) and others. Whenever the orientation of the machine hall fixed by other considerations happens to be within 30° to 90° to the direction of the penstocks (in many cases) it is evident that the connecting pipes will come at right angle with take off from the main shaft at 30° to 45° (Fig.6d & 6f).

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6.6 <u>SURGE CHAMBER</u>

The pressure tunnel upstream of the surge tank is not usually designed to withstand high pressure. A surge tank located at the head of the pressure shaft will be the best protection against water hammer penetrating the tunnel. A surge tank located as close as possible to the power station makes possible a reduction in the length and thickness of the costly high pressure shaft and also permits stable regulation of the turbine which may become impossible with a longer pipe line (27). Surge tanks are usually associated with conventional hydroelectric developments incorporating surface power houses. Underground power station also require surge tanks, and these generally give rise to new and difficult hydrodynamic problems of their own.

Surge chambers in the pressure conduits are not ordinarily required with low to medium underground developments (37). Thus, the so called 'head' developments do not involve surge chambers on the pressure conduits. The long pressure tunnels and penstocks associated with high head or so called 'tail' underground plants normally involve surge chambers on the pressure line. In the case of 'intermediate' development, surge tanks will be required both at upsteam and downstream of the power station.

In some instances the tailrace tunnel will be a free flow tunnel with the possibility of the translatory waves being formed on the surface of the free flowing water in response to variations of the discharge aorequired by the changes of electrical load. There is then no departure from the conventional hydrodynamic system as in above ground condition. In some instances the tunnel downstream of the turbine may be under pressure wholly or intermittently or partially. May be the tunnel is always under pressure for conditions of steady flow as well as for unsteady flow. In certain cases

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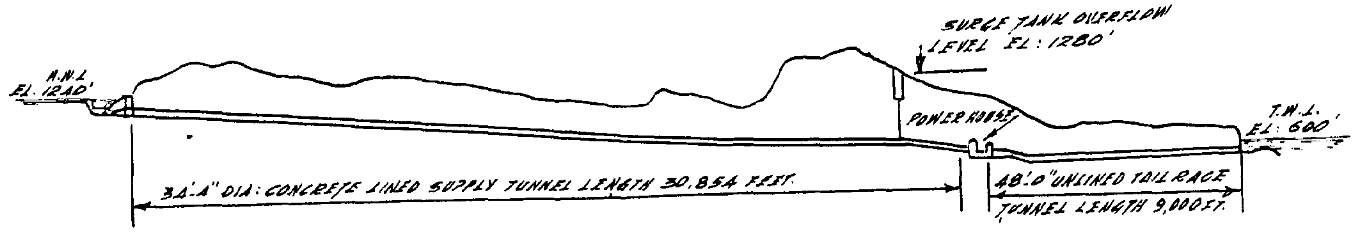
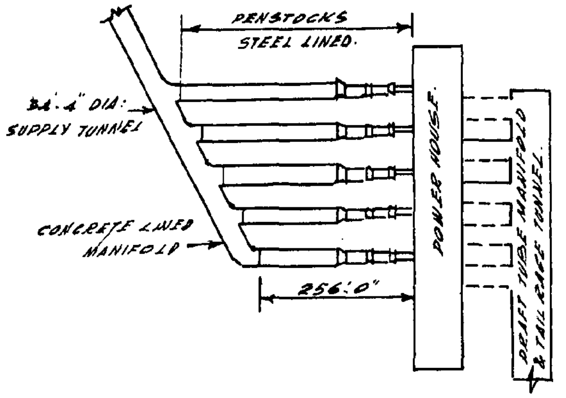
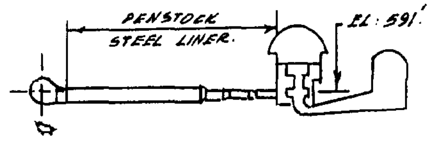




PLAN OF MANIFOLDS, PENSTOCKS & POWER HOUSE.



PROFILE OF PENSTOCK.

FIG: 6(+) PLAN AND PROFILES OF CHUTE-DES-PASSES DEVELOPMENT.

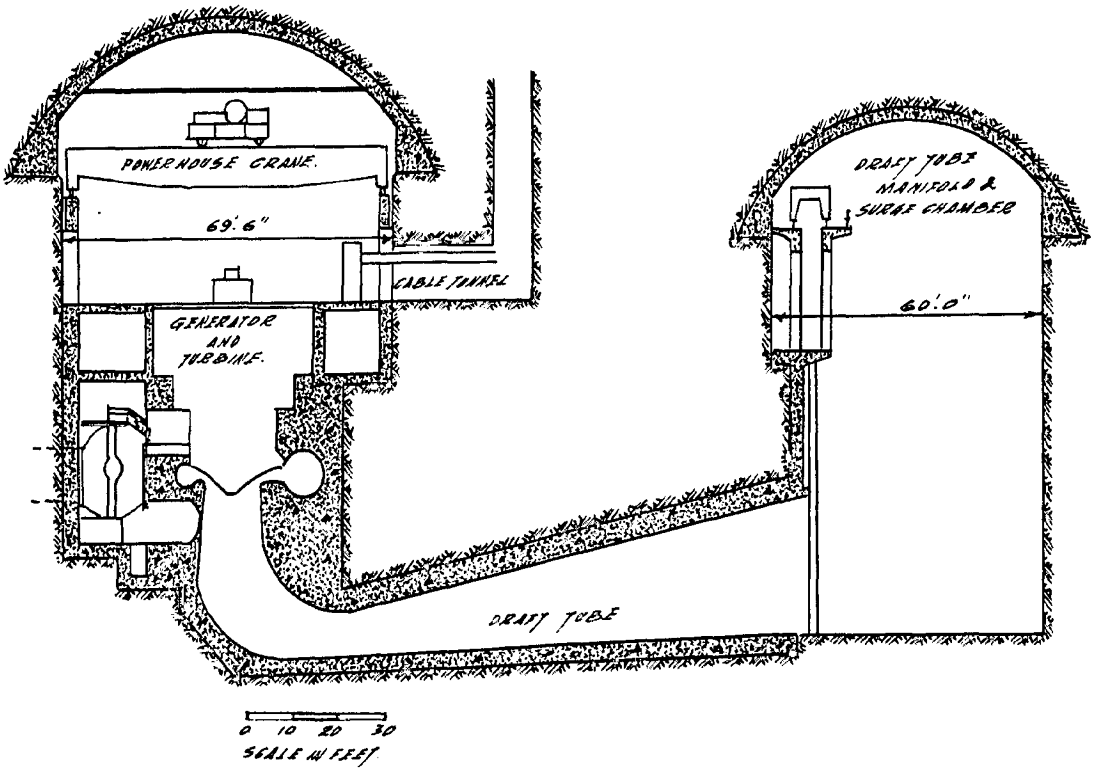


FIG: 6(+) CROSS SECTION OF CHUTE-DES-PASSES POWER HOUSE & DRAFT TUBE MANIFOLD.

the tailrace tunnel may be normally a free flow one and works under pressure when the downstream level rises (flood) or when the discharge in the tunnel varies rapidly. In most of these instances a proper surge chamber has to be incorporated on the downstream side of the turbine so as to minimise the water hammer and pressure variation and to make the turbine governing practicable.

In underground developments and overground developments, design aspects of the upstream surge tank are identical. Downstream surge tanks can also be investigated in a manner similar to that followed in the case of upstream ones. Systems with downstream and upstream surge tanks are applied in connection with reaction turbines (Francis turbines, less frequently propeller turbines), reaction turbines being only followed by a conduit discharging under pressure and hydrodynamic condition of such a system will be different. The design of a surge tank is a subject by itself and therefore only consideration with regard to type, shape, location and arrangement with the surge tank related to underground development is discussed below:

An analysis shows that combined systems of surge chambers with two or more shafts, having restricted orifices, expansion chambers, overflows and reservoir chambers have been constructed for upstream location of tanks (27). In all cases the main aim is to reduce the volume of excavation while assuring stable governing conditions. According to Jeagar, a throttled surge tank with lateral orifice is a better

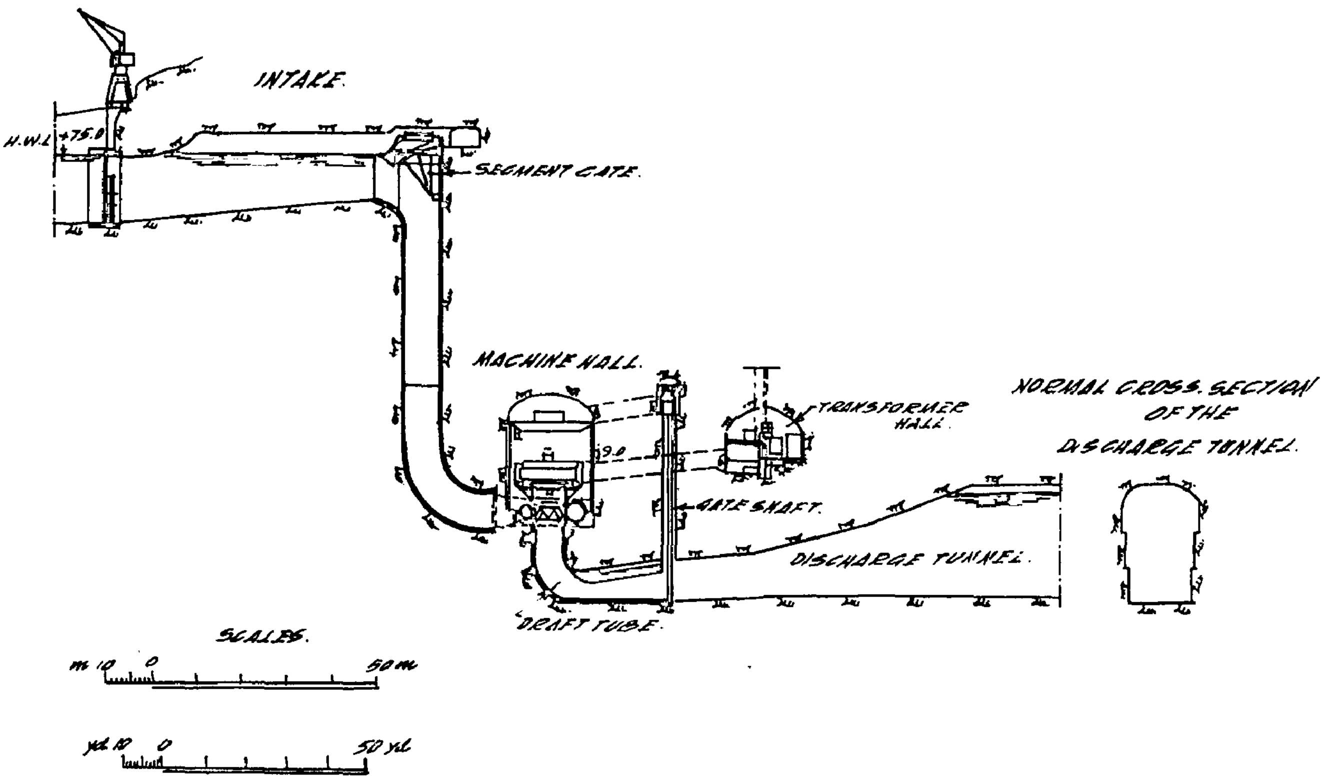


FIG: 6(8) VERTICAL SECTION THROUGH THE POWER STATION.

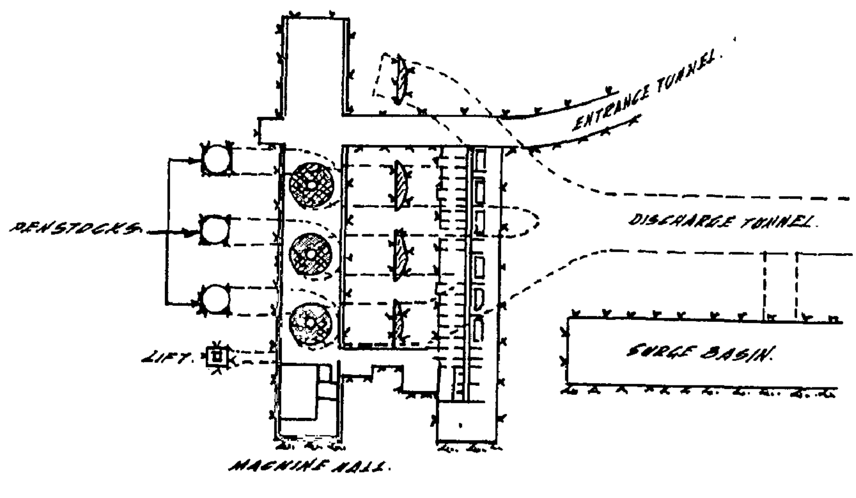


FIG: 6(8) HORIZONTAL SECTION THROUGH THE POWER STATION.

STJONORRFORS STATION.

arrangement than providing a surge tank with restriction just above the pressure tunnel, as in Randens U.G.power house of Isere-Arc. This arrangement have definite advantages, the chief being that the construction of the tank does not interfere with the construction of the main tunnel. Moreover, the water levels in the tank are allowed to fall lower than with a more conventional design of tank with a restricted orifice. Though it is ideal to keep the surge tank as close to the power house as possible, it might be necessary to locate the surge tank far upstream of the penstock head due to economical reasons. The surge tank of Chute-des-passes (42) located 2800 ft. upstream of the penstock manifold is particularly illustrative (Fig.6f). In the above station the draft tube manifold which acts as a surge tank for the discharge tunnel is approximately 7 times of Thoma area. similar arrangement is adopted for the Besimis (Fig.6d) station also (1). Draft tube gates can be suitably arranged in the downstream chambers as in Chute-des-Passes and Bersimis stations. The necessity of a downstream surge tank can, of course, be eliminated (49) by ensuring free flow conditions in the tailrace tunnel (Ex. stonorrfors and Ceannacroc arrangements - figures 6g and 6h. This may prove expensive if long distances are involved. The tunnel should accommodate the high surge waves following gate opening. The section required would be excessively large so that it may be found more economical to construct, in case of long distances, a pressure tunnel with a surge tank just downstream of the power station (62) as at Glenmoriston, Tumut I

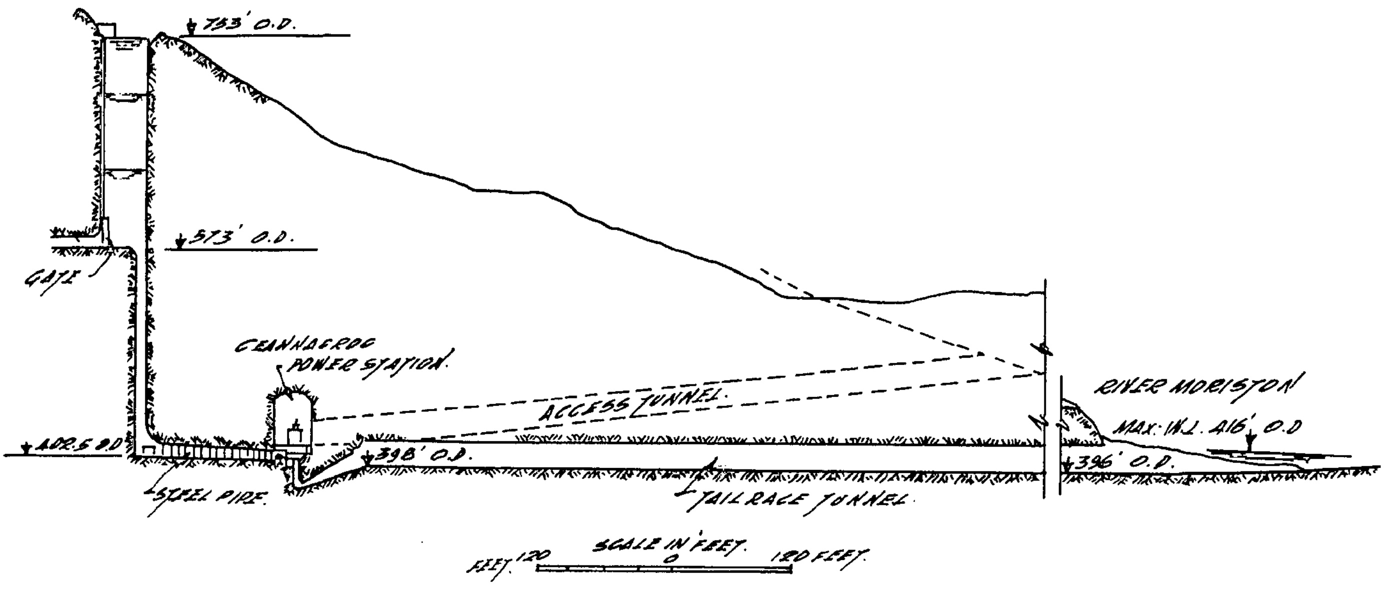


FIG: 6(h) CEANNACROC STATION- (LONGITUDINAL SECTION OF WORKS).

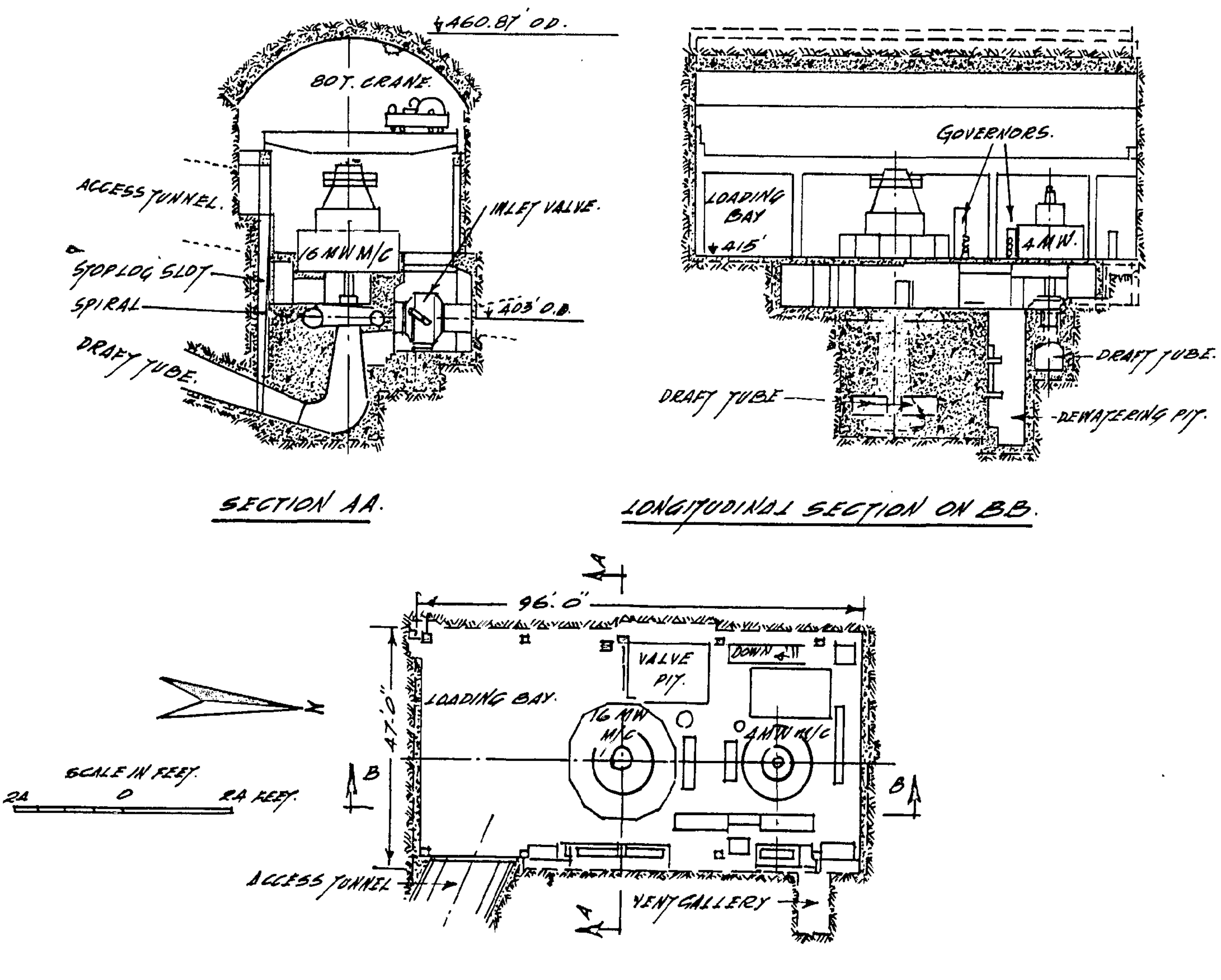


FIG: 6(h) DETAILS OF CEANNACROC STATION.

and II Station (Figs. 6e & 6i & 6j) and others. The downstream surge chamber may be designed arbitrarily as far as the shape is concerned. It may be shaft like or a basin of ample dimension. The chamber connected to several units should be designed as a coherent cavern if a single tailwater tunnel is constructed. The surge chamber is in such cases a long gallery extending before the draft tube ports and sicharging into a single tailwater tunnel as at Verbano station. A short entrance section of the tailwater tunnel blasted to a significantly enlarged diameter has also been used to act as surge chamber (Isre-Arc-Fig.6u). In case of two or more tailwater tunnels, the number of independent surge chambers must be increased correspondingly; each of the surge chambers comprises the ports of all the draft tubes served by a common tailwater tunnel (49). For the Aldeadavila (Fig.6b) immediately downstream of the machine hall are four surge chambers connected in pairs, each pair communicating with the draft tubes of three machines and for each pair of surge chambers a separate tailrace tunnel returns the discharge to the river (80). With this arrangement the station is divided hydraulically into two halves each comprising three machines so that it is possible to dewater either half of the station including tailrace tunnel while other half continues in operation (see Fig.6b & 6k). A single or at the most two tailwater tunnels is used in general practice (49).

From the hydraulic aspect, the downstream surge tank may be designed as a simple shaft with expansion galleries at top and bottom as at Glenmoriston station (Fig.6c) or as a

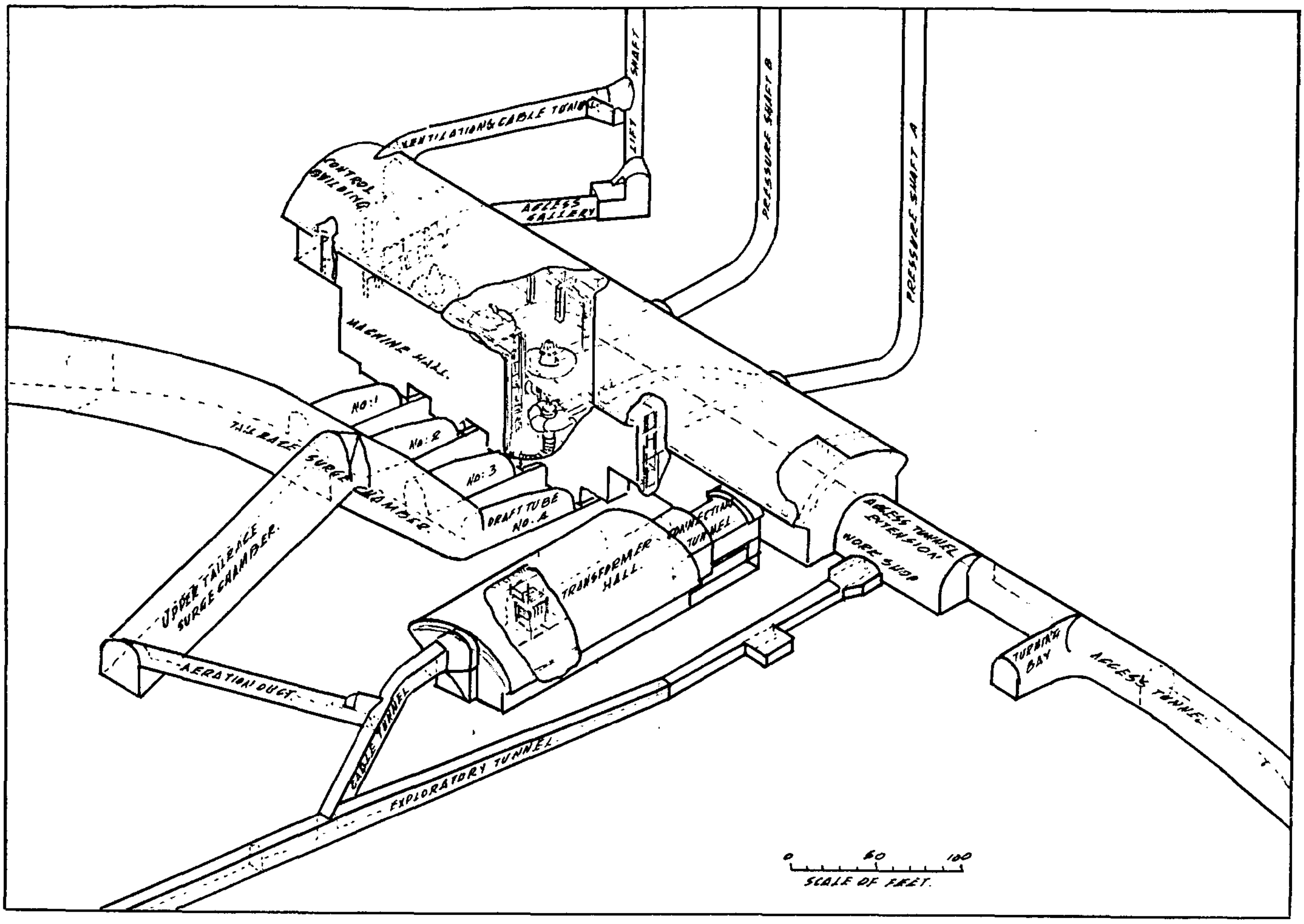


FIG: 6(i) TUMUT I POWER STATION - ISOMETRIC DRAWING.

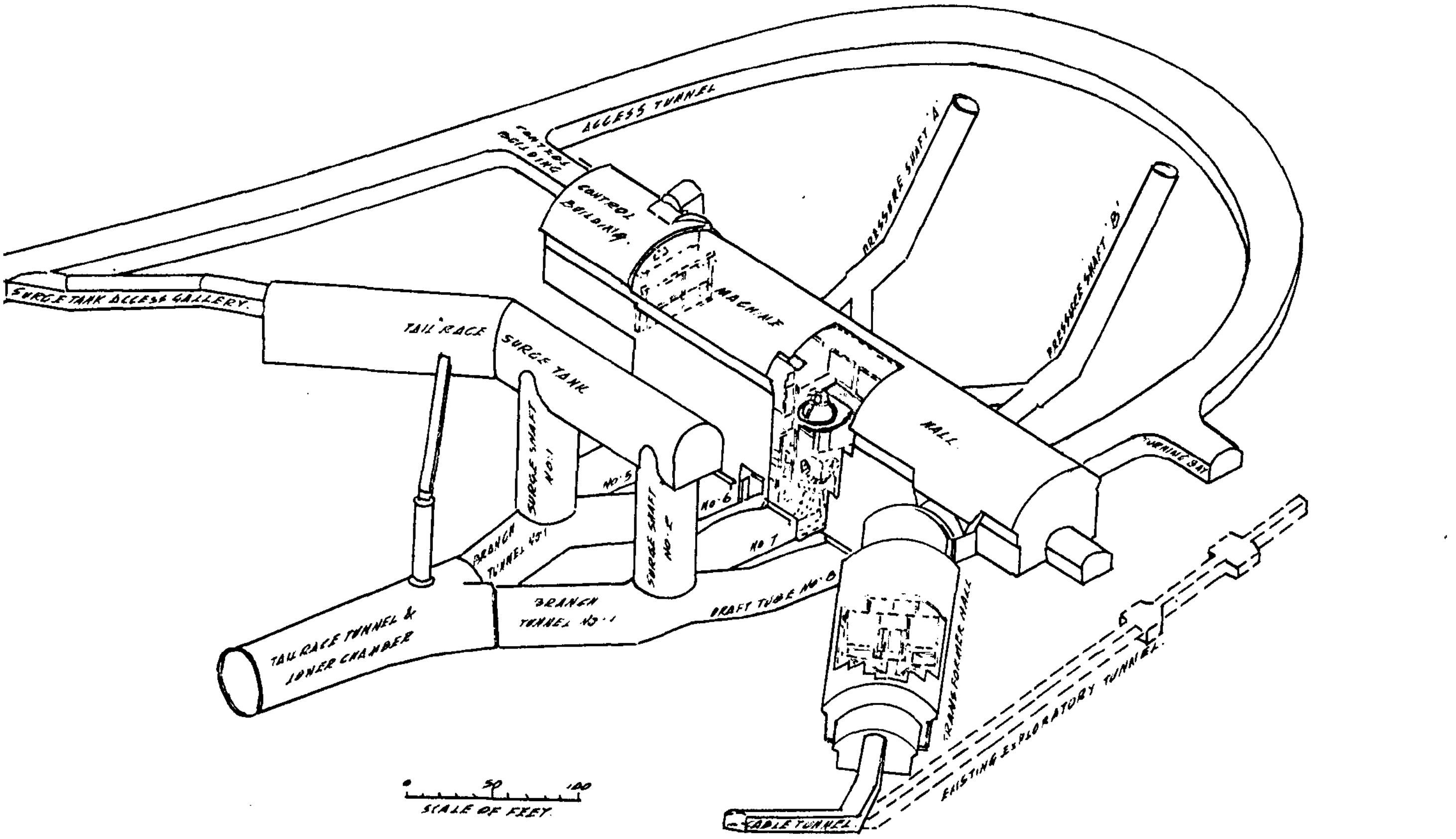
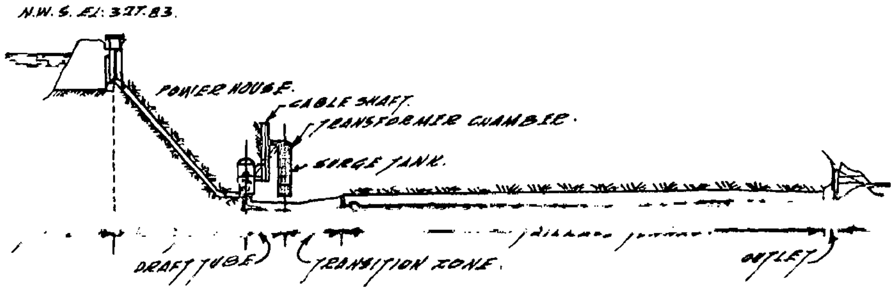


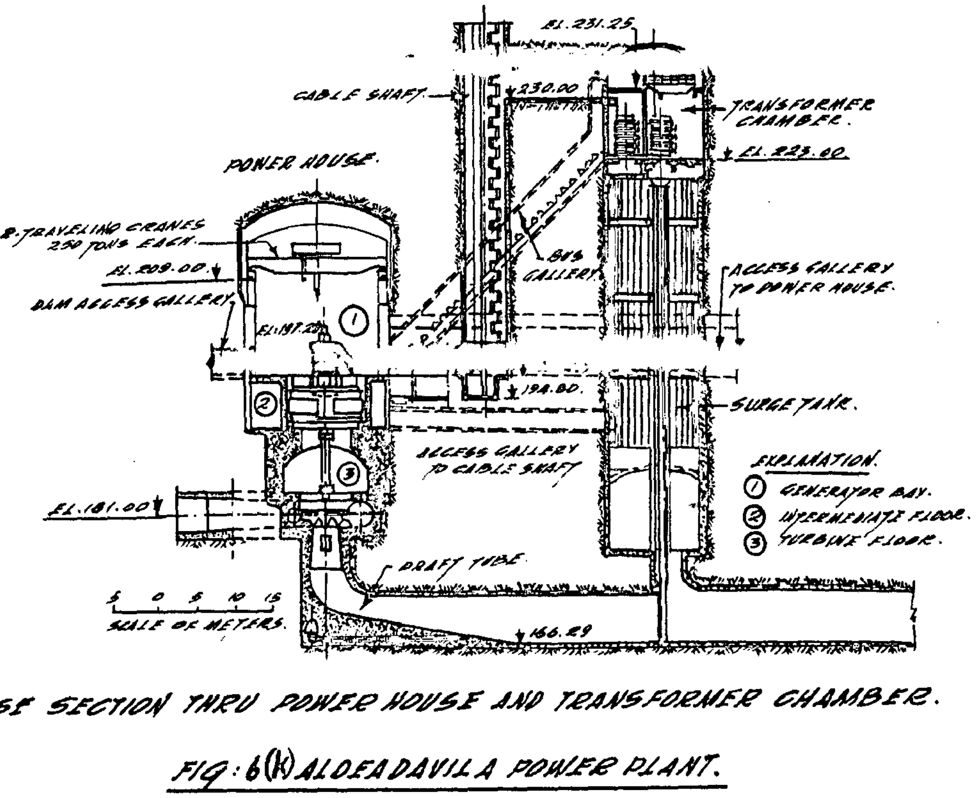
FIG: 6(j) TUMUT 2 POWER STATION - ISOMETRIC DRAWING.



LONGITUDINAL PROFILE THRU POWER HOUSE.

50 0 50 100
SCALE OF METERS.

FIG: 6(K) LONGITUDINAL PROFILE THROUGH POWER TUNNELS AND POWER HOUSE.
ALDEADAVILA DAM AND POWER PLANT.



TRANSVERSE SECTION THRU POWER HOUSE AND TRANSFORMER CHAMBER.

FIG: 6(K) ALDEADAVILA POWER PLANT.

simple shaft as at Tumut 2 (fig.6j) or shaft having restricted orifices. Simple chambers are used in most cases, yet for increased dampening of oscillations restricted-orifice chambers are applied (Pionnay station, Switzerland).

Sometimes surge chambers are arranged within the same excavation as that for the transformer hall (Serre-Poncon station and Aldeadavila Ref. Figures 6a & 6k). With this arrangement, the number of underground cavities can be limited.

Surge chambers should be provided with amply dimensioned air vents. The air duct may be combined for reasons of economy (49) with one of the service tunnels frequently with the main access shaft or tunnel (see Figure ~~##~~ 6i).

6.7 PENSTOCK VALVES & TURBINE VALVES:

Valves are usually installed at two places in the penstock. One is at the upstream end of the penstock called control valve or penstock valve immediately after the surge tank, while the second is at the downstream end of the shaft or at the unit penstock, immediately ahead of the turbine called inlet valve or turbine valve. Dewatering of penstocks is ensured by the penstock valve which serves at the same time as the emergency valve in case of penstock rupture. The turbine valve is to close the penstock when the turbine is not operated. In installations where one penstock serves two or more turbine units, individual valve for each turbine is invariably provided for facilitating maintenance work on one unit. Provision of a control valve for each turbine is therefore absolutely necessary (49). Penstock valves are sometimes replaced by gates located in the surge shaft. Both these devices can be push button controlled for remote operation from the control room. In the case of penstock valves, however, the provision of a separate set of stop logs is essential for the maintenance of valves itself. When gates are installed, stop logs can be avoided as maintenance can be done even during operation. However during maintenance of gates, there will be no safeguard against a penstock rupture. As such stop logs would also be required. Provision of gate and stop log in the surge shaft are diffulult and may present problems in the design of surge tank. It therefore appears, desirable to provide valves in a separate valve house and only one set of gates or stop logs as standby arrangement in the surge tank. The choice between gates and stop logs is depended on economics and ease of operation. Gates are generally preferred as stop logs have to be provided for the entire height. In the case of low-to-medium head developments, control of flow through the pressure conduit is provided by various types of intake gates but several types of valves are sometimes utilised. In the case of high head developments a valve is ordinarily located at the upstream end of the pressure shaft as at Kemano (38) and Haas (10). Sometimes pensstock valves are eliminated completely and reliance is placed on automatic closing devices at the tunnel intake as at Fionnay and Aldeadavila (80). Ample provision for automatic air inlet must be made just downstream from such valves or closing devices to avoid collapsing of the penstock liners due to vacuum. Valves

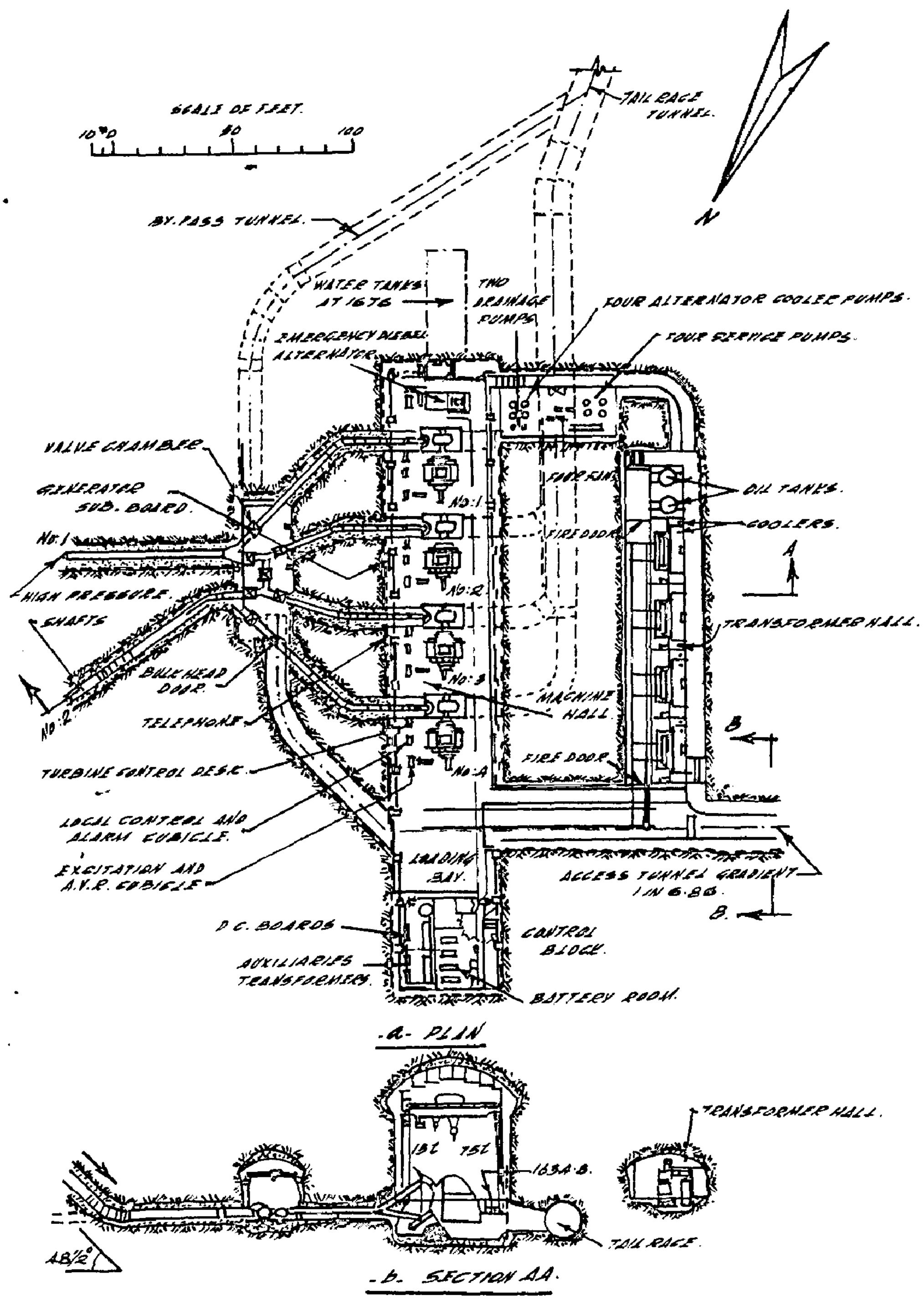


FIG:6(L) JOR POWER STATION.   PLAN AND CROSS SECTION.

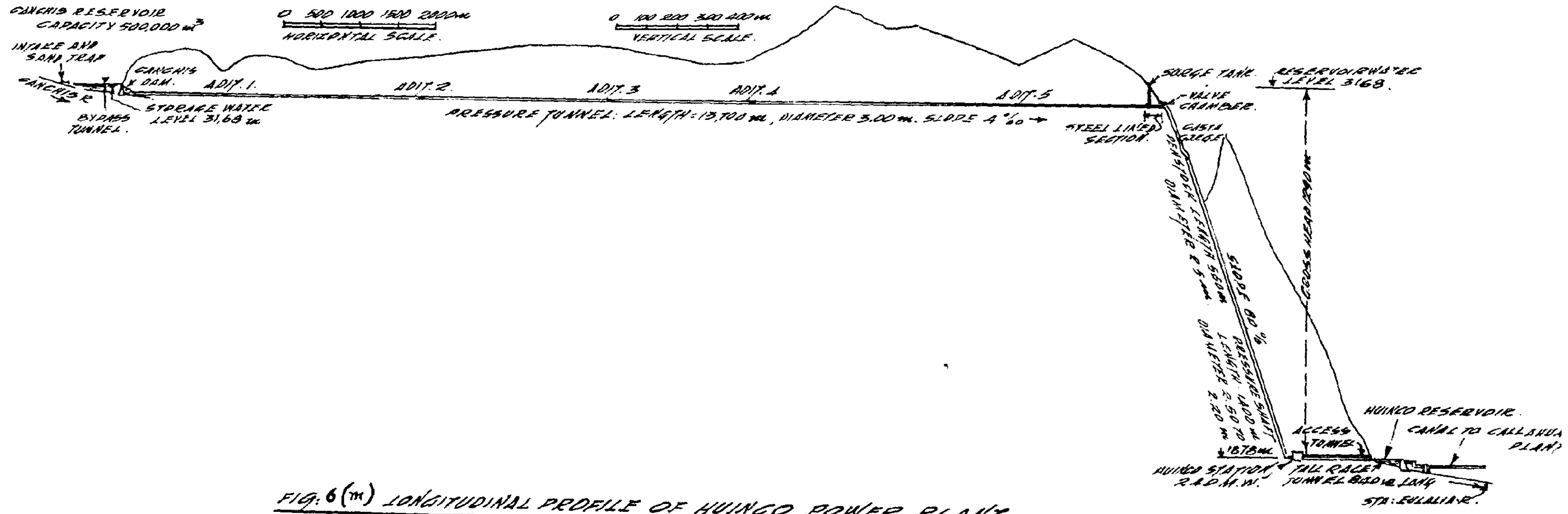


FIG: 6(m) LONGITUDINAL PROFILE OF HUINCO POWER PLANT.

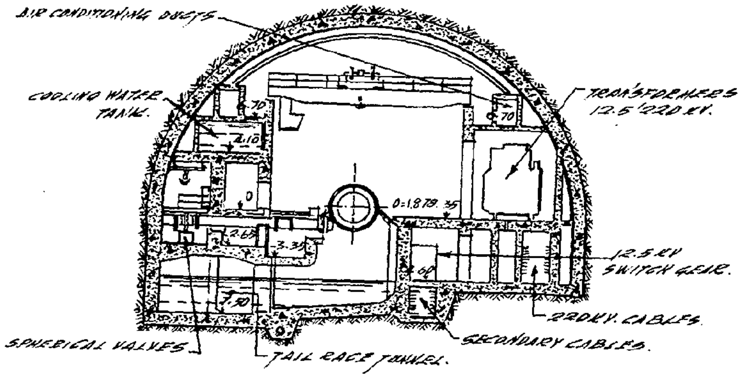


FIG: 6(m) SECTION THROUGH HUINCO MACHINE HALL

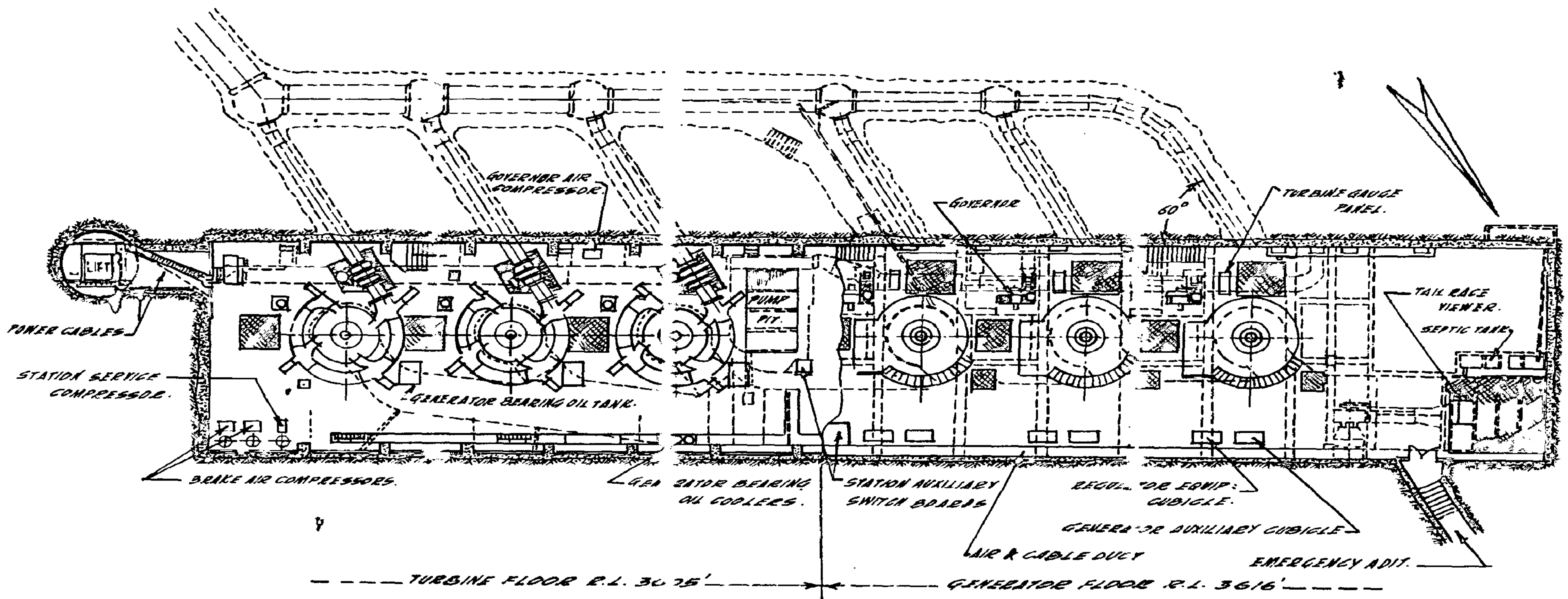


FIG: 6(N) PLAN SHOWING GENERAL ARRANGEMENT OF McKay CREEK UNDERGROUND POWER STATION.

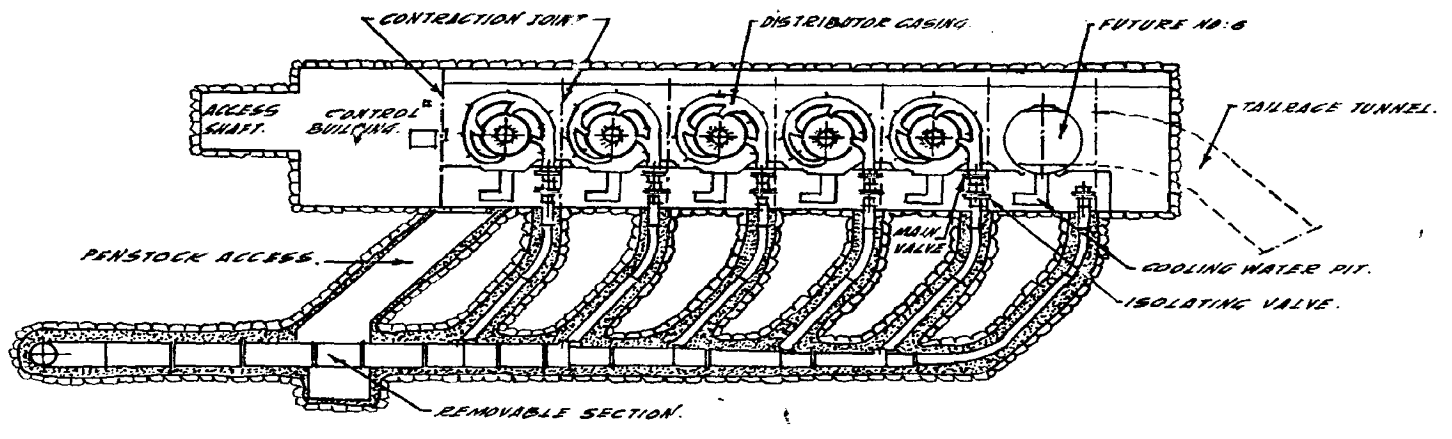


FIG: 6(O) PLAN SECTION ON DISTRIBUTORS AND PENSTOCKS.

(POATINA UNDERGROUND POWER STATION)

at the penstock head are generally located in a separate gallery. Removal of these valves is done usually by cranes. A hook anchored in the rock and a pulley block is used for the removal of f valves at Randens station.

Unit valves are ordinarily installed on the branch connections to the individual turbines. There may be single or duplicate valves as at Innertkirchen.

The valves may be located in a separate cavern as at Jor station (12) Fig.6l and Stalden station (82) Fig.6e and Huinco station (61) Fig.6m and others; in a passage between the penstock tunnel and the powerhouse; or in the power house proper as at Mckay Creek station. Fig.6n Bersimis 1 (1) Fig.6d and elsewhere. Actual location is governed to a substantial extent by the nature of the rock formation. Placement of the valves in a separate chamber reduces the necessary width of the chamber required for the generating units but does necessitate a separate crane and access passage. In the case of high head and medium head developments, where serious consideration must be given to the possibility of a ruptured pressure shaft, the valve chamber is ordinarily provided with its own discharge tunnel connecting with the tail race as in Jor Power station (Fig. 6l) and others. In the event of a full closure of an underground station located in the upper reaches of a river, by-pass arrangement would be required to ensure supply of water for down stream requirements. Bypass valves-cum-energy dissipators can be installed on individual units so that they automatically open by the movement of servomotor pistons as the guide vane closes. Such an arrangement is very costly. In Gando Project a needle valve is provided at the lowest portion of the penstock to discharge irrigation water by dissipating the high pressure energy of 400 m in the event of failure of turbines (64).

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6.8 POWER HOUSE & APPURTENANCES:

6.8.1 GENERAL:

The dimensions of the underground cavity for housing the generating machinery greatly depend on the size and number of generating sets and station arrangement. As in the case of surface station, the turbines may be Pelton or Francis and of vertical or horizontal type (see Chapter IV). For low heads Kaplan turbines are employed. The basis upon which rests the choice between the different types of turbines is essentially the same as for ordinary aboveground stations, but increased importance is attached to the volume of rock excavation. The tendency towards a more economical utilisation of space is even more pronounced than at the surface arrangement. Having fixed the type as well as the shaft setting, the section of the Power House cavity can be designed on a tentative basis and the same may have to be modified when more particulars about the generating machinery are got from the suppliers. For the underground location of the station, communication galleries, shafts and tunnels for (i) access and transportation; (ii) ventilation; (iii) accommodating cables and buses and (iv) disposal of flood water have to be constructed between the power house and the surface as well as between individual caverns. To reduce the construction costs, these passages should be united as far as possible. The design criteria and principles followed to meet the above requirements are discussed below:

6.8.2 UNDERGROUND POWER HOUSE:

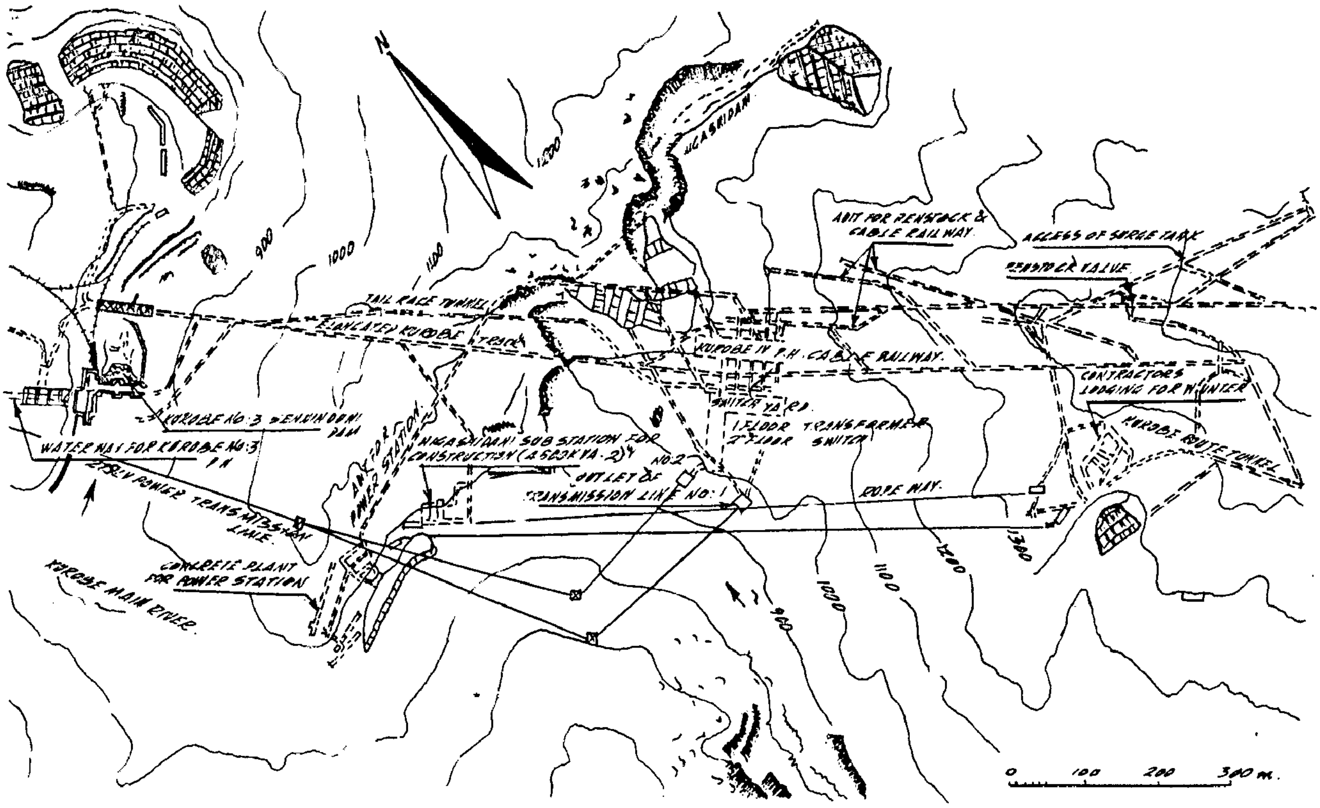


FIG: 6(b) GENERAL PLAN AROUND POWER STATION.

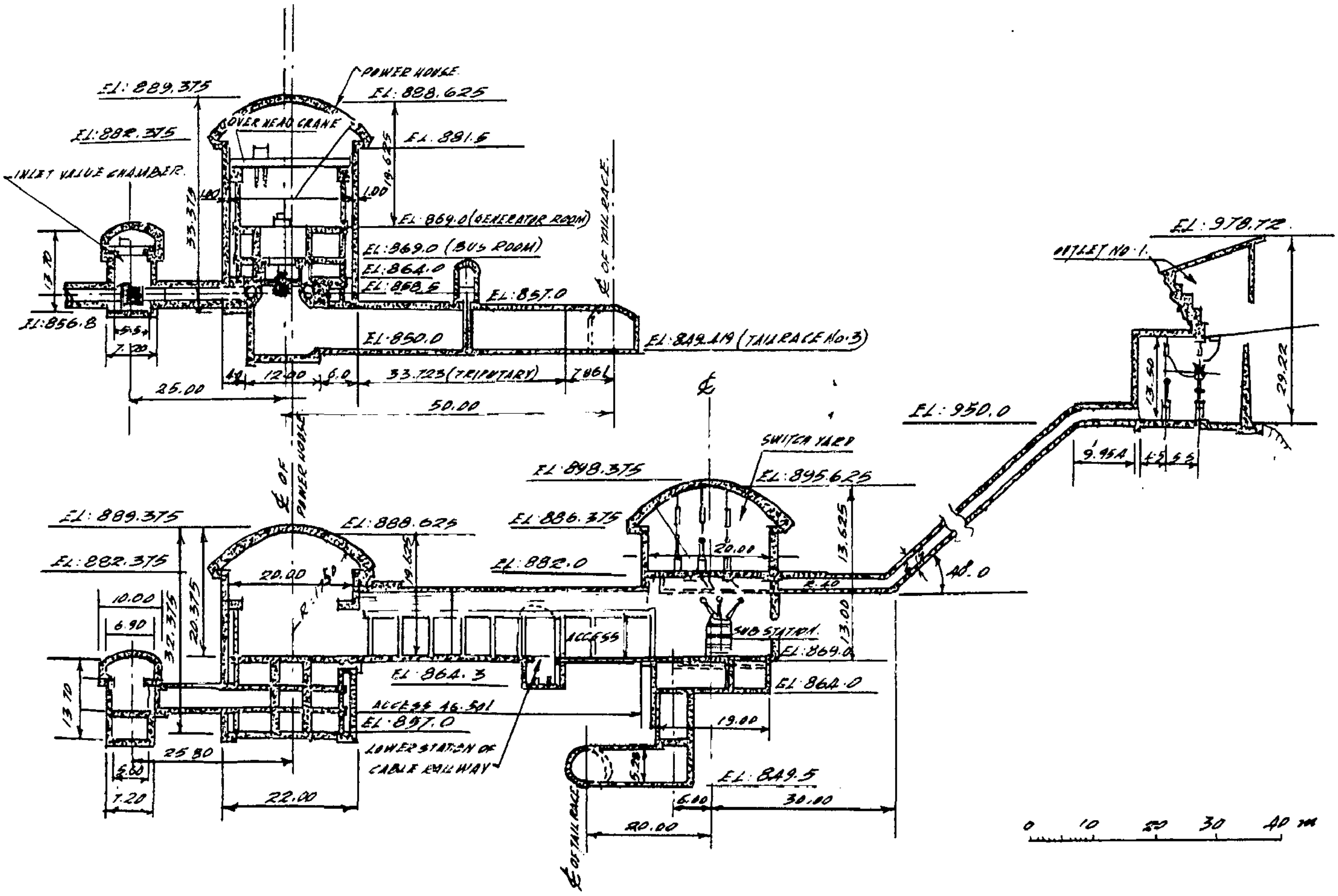


FIG: 6(b) CROSS SECTION OF POWER HOUSE.

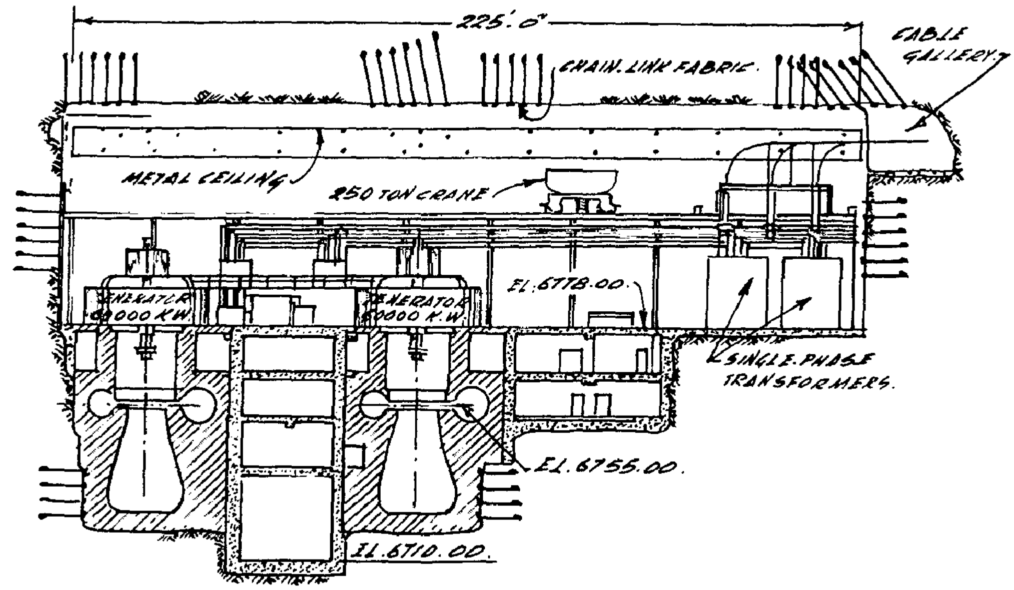


FIG: 6(q) LONGITUDINAL SECTION OF MORROW POINT UNDERGROUND POWER PLANT SHOWS FLOOR SLABS WHICH SERVE AS STRUTS OR BRACES BETWEEN THE ROCK WALLS. NO EXPANSION JOINTS ARE TO BE USED IN THE PLANT SINCE THE MINOR TEMPARATURE CHANGES WILL NOT PRODUCE EXCESSIVE EXPANSION.

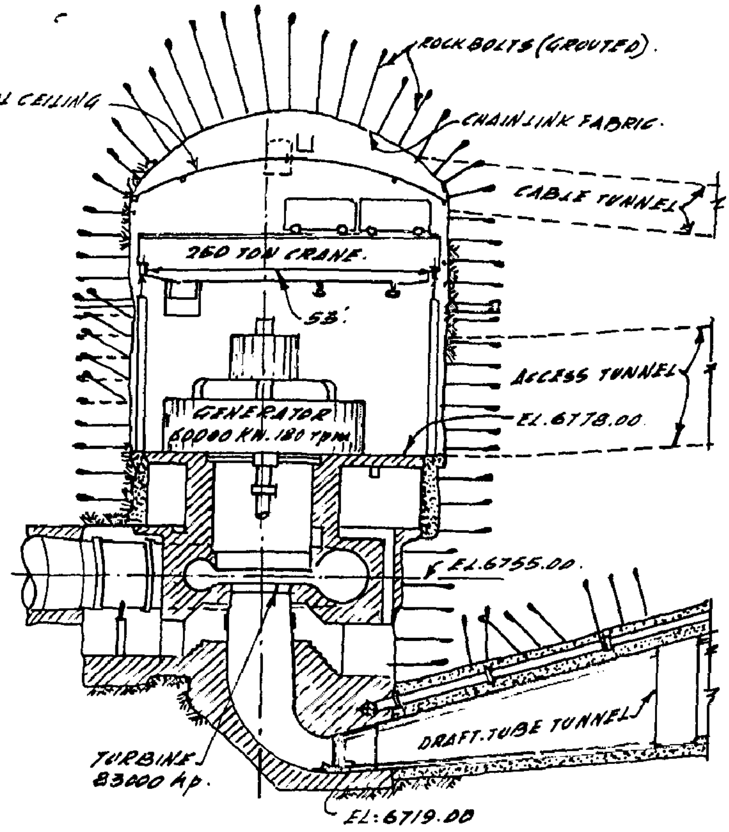
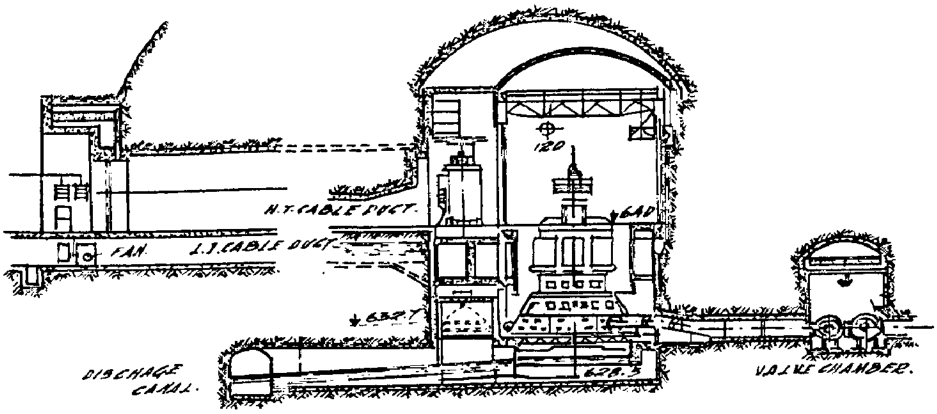
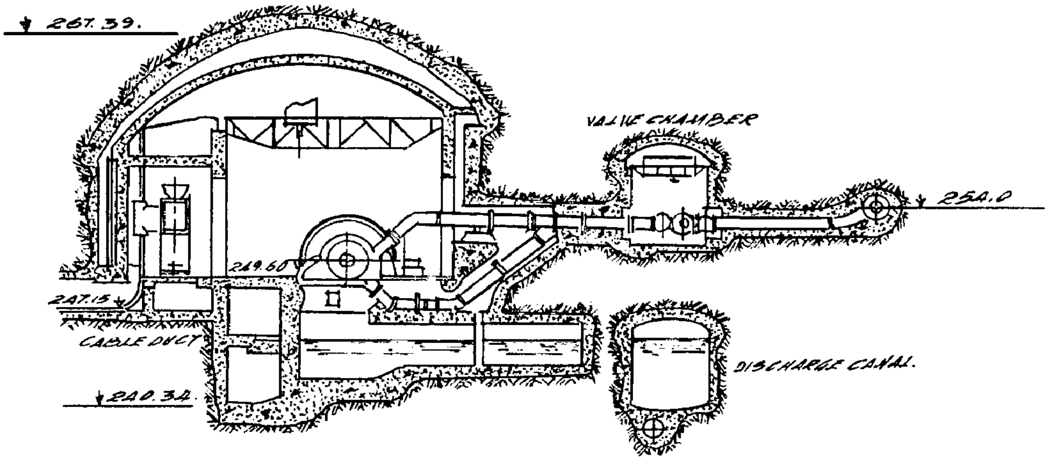


FIG 6(q) TRANSVERSE SECTION OF MORROW POINT POWER PLANT SHOWS ROCK BOLTING AND METAL CEILING.

is constructed to accommodate the turbines, generators and ancillary equipment. Sometimes the turbine valves and transformers are also accommodated within the power house cavity or in independent cavities. Sometimes the entire power station equipment and substation equipment are located underground as at Tafjord III station in Norway Kurobegawa No.4 Station, Japan (Ref:32 Fig.6p). The decision to accommodate the substation equipment also in underground caverns was taken to prevent damage from snow piles during the extremely inclement weather in winter when the mercury drops to as low as -20°. Depending upon the geology and type of rock at site, the span of the underground cavity is limited and in such cases independent caverns are constructed for housing turbine valves, generating machinery and transformers as at Jor station (Fig.6l) Koyna, the Serre Poncon (Fig.6a) and others. When a cavity with a large span can be constructed, the cross section is divided into three parts, the machine hall in the middle flanked on either side by a hydraulic (valves) and electric section (transformers) as at Huinco station (Ref: 61 Fig.6m) and Peccia (Fig.6r) and others. In some stations, the valves are located inside the main cavern and transformers are located (i) overground as at Mackay Creek station (Ref.22 Fig.6n), Bersimis and Chute-des-Passes and others or (ii) in an extension of the station as at Morrow Point Power Plant (Fig.6v). Another arrangement is to accommodate the valves in a separate cavity and to house the transformers within the power house cavity as at Stalden Station (Fig.6e) Innertkirchen (Fig.6r) and others.



INNERTKIRCHEN. (672 m)



SANTA MASSENZA. (590/460 m).

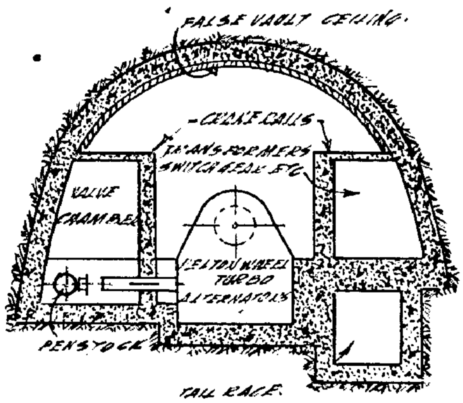


FIG: 6 (r)

PECCIA. (426 m).

6.8.2 TURBINE VALVES--LOCATION:

As discussed earlier, the valves upstream of the turbine can be located inside the main cavity or in a separate cavity upstream of the Power station. The latter arrangement is mostly retained for high head schemes. Increased safety against flooding is ensured by the construction of a separate gallery for valves, since only the latter become flooded in case of occasional valve failure. The complete filling of the valve gallery is prevented by means of a by-pass gallery to the surface or to the tailrace as at Jor station (Fig.61). When both the supply conduit and discharge conduit are arranged at the same side of the power house as at Stalden station and Huinco station (Figures 6e & 6m) the valve gallery is directly connected to the tailrace tunnel and thus the By-pass tunnel is omitted entirely. According to Mosonyi (49), this arrangement is especially advantageous in case of a free penstock located in a service tunnel. For medium head stations with Francis installation, increased attention is to be paid for the location of the bypass tunnels from valve cavity to the tailrace tunnel as this will invariably be a pressure tunnel. In such installations the valves are located atabout generator floor level or at a higher level in a cavity in the pressure shaft to facilitate connection of the by-pass to the tailrace pressure tunnel (Salamonde Station). When access to the valve cavity located at generator floor level is provided from the power house cavity, a water tight separation door protects the power station against flooding in the event of damage to the valves or penstocks.

When the turbine valves of high head installations are located inside the power house cavern connection galleries are provided for discharging the flood water into the free flow tailrace tunnel. The tailrace in such installation is aligned along the longitudinal axis of the station as in Poatina and the Huinco stations (see figures 6o & 6m). In Poatina station, emergency drains have been provided from valve pits to the tailrace, which is under the turbines and on the long axis of the station, thus making a subsidiary excavation unnecessary (47). With Francis turbine installations, however, special problems are involved when valves are also located within the main cavity. The valve pit is separated from the generating machinery by a concrete wall and flood protection is provided by submersible drainage pumps brought into operation by water level relays. With such arrangement as in Tumut II, the whole power plant is to be shut down in the event of the water level continuing to rise.

From the above discussion, it is clear that the valves may be located in a separate chamber or in the power house proper. Actual location is governed to a substantial extent by the nature of the rock formation. In the case of separate cavity, it will be necessary to instal a separate crane for handling the valves and additional gallery or shaft for access. On the other hand, a separate cavity sometimes afford additional access for pressure shaft excavation. The present day trend in underground power station layouts is to install the valves in the main cavity itself, as it becomes possible to economise on the total underground excavations and it is also possible to use the power house crane for its erection and subsequent maintenance.

6.8.3 TRANSFORMERS & ELECTRICAL CONNECTION WITH OUTSIDE:

In underground stations, the main transformers are variously located; (a) inside the main underground cavity either in an extension of the cavity as at Stalden station or opposite the generators as in Huinco Station, Innertkirchen, Santa Massenza, Morrow point Station and others; (b) in separate caverns as in Tumut I and Tumut II stations (Fig.61 & 63), Jor station and others; (c) outside above ground as in Mackey creek, Chute-des-Passes, Bersimis I and others. There is an increasing tendency to place the transofrmers underground (37). The choice of the location depends largely on the distance between the machines and the high voltage line take-off. If this is about 200 yards or more the underground location is preferred both as regards capital cost and losses (19). A simple rule of thumb used by Italian Engineers is that, if the length of the low-tension transmission cables is more than 250 to 300 metres, the arrangements with underground transformers is cheaper. The advantage of one alternative with respect to the other should be studied case by case with reference to the capacity and number of the machines besides the length of the connection between the machines and the transformers. Comparative study should be made between outdoor and indoor installation costs, besides the cost of connections and their losses, making also allowance for the less effective operation of a generator with increased overall reactance in the generator bus bar system and the effect on the static and dynamic stability of the net work. Whether the transformers are underground or overground, dissassembly of the transformers has to be carried out in a room separate from the machine hall. Consideration should be given to higher cost of underground disassembly room and relevant fire protection installations which are more expensive for underground than for surface plants (41). Some authorities prefer to locate the transformers at ground level to avoid hazard associated with large volumess of transformer oil loacted within or directly adjacent to the power station cavity. This arrangement results in long, low tension leads. In the Bersimis and Chute-des-Passes developments, these leads are 400 to 500 Ft. long and studies indicated that long low voltage leads are economic solution when the excavation for underground transformers is taken into account (44).

The span of the cavity is not increased when the transformers are located in an extension of the powerhouse cavity. Added advantage is that the powerhouse Crane can be utilised for the erection and maintenance of the transformers. The transformers can be placed on rock and therefore cost of foundation will be reduced. The transformers are arranged in separate compartments and fire protection doors are provided. Oil spilling from the transformers in case of explosion can suitably be dealt by constructing drainage pits and collection chambers.

When the transformers are located opposite to the units, the width of the cavity is increased beyond that required for the generating units and this arrangement necessitates a divider wall to minimize the fire hazard. Though the power house crane can be used for erection and subsequent handling of the transformers, the deck on which the transformers are to be placed will be costly. Besides, drainage arrangement for the oil spilling from the transformers will be difficult and costly.

When the width of the powerhouse cavity is limited by rock conditions, construction of an additional cavern for the transformers may be preferable inspite of the excess cost of excavation. Their separation may be beneficial as far as safety requirements are concerned.

The underground location of 200KV or even higher voltage transformers is extremely expensive yet popular now-a-days. The 300 KV transformers at Kemano is accommodated within the power house cavern. Single phase transformers of 380 KV at Harspranget and three phase transformers of 275KV at Cruachan (Fig.6s) both located in separate caverns are particularly interesting features.

Connection between the generators and their transformers can be either bus or cable depending on the economic solution in the particular case. When separate cavern is constructed for transformers, a cable or bus gallery (Fig.6k) should be excavated between the machine hall and transformer

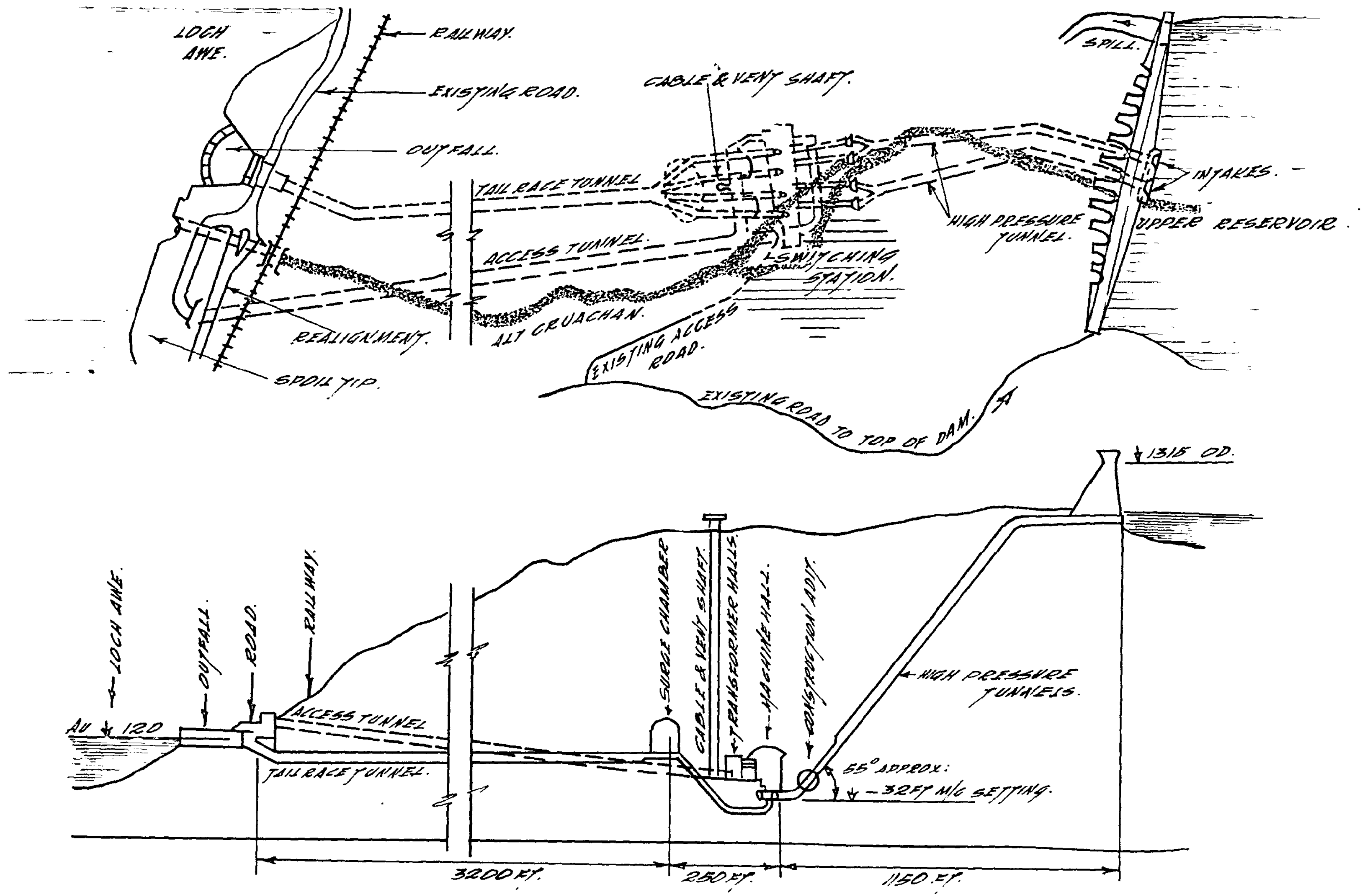


FIG: 6(5) PLAN VIEW & PROFILE OF THE CRUACHAN SECTION.

chamber for the low voltage electrical connection. Low voltage connection is sometimes taken through the interspace between the false ceiling and the power house roof. These arrangements are often offset by the problems of providing high voltage connection between the transformers and the open switchyard. Depending upon the overground location of the switchyard, the high tension cables are taken through shafts of sloping tunnels or through the access tunnel or tailrace tunnel. With special installation techniques employed, the 275 KVA oil filled cables have been taken through a 15' diameter vertical shaft of 1080 Ft. at Crauchen. (81 - see Fig.6m). Similar arrangement is seen at Mckay Creek station (22) and at Aldeadavila station having a 1040 Ft. cable shaft (17 - see Fig. 6k) and others. In all the above cases these shafts are put to multipurpose use such as for accommodating Power and control cables, lift arrangement, stair way & ventilation ducts. Sometimes a passage is left for lowering the machine parts to the power house cavity.

Sometimes power and control cables are taken through the access tunnel as at Jor station (11). At Huinco station (61) the 220 KV oil filled cables are laid on asbestos-cement trays in a duct running along the 800 metre long main access tunnel. In certain stations, cables have been taken through free flow tailrace tunnels. Sloping tunnels have been constructed (3 and 82) solely for the purpose of taking cables to the switchyard at Morrowpoint plant (230 KV cables through a 10' x 10' and 190 ft. long tunnel) and Stalden station ($5.9m^2$

in section and 470 metres long) and in many other stations. In some swedish and other stations sloping tunnels have been constructed for ventilation which serve as cableway for outgoing power cables. For periodical inspection and maintenance work sloping tunnels for cables are constructed with mild slope. Cable tunnels through sound rock are invariably unlined.

The choice between a tunnel or shaft is purely based on economic studies.

6.8.4 POWER HOUSE INTERNAL LAYOUT:

Palo and Marks (51) and Doland (13) have established basic principles for plant layout in overground stations. Though these concepts are generally followed, a more intelligent approach is aimed for equipment layout in underground stations to reduce excavation.

As in overground plants, the unit bay layout, the most expensive feature, is predicted on five primary items considered in the following order (83)

i) The plant water way (turbine, draft tube and penstock construction;

ii) The generator;

iii) The main power transformer;

iv) The overhead cranes or equipment needed for installation and maintenance; and

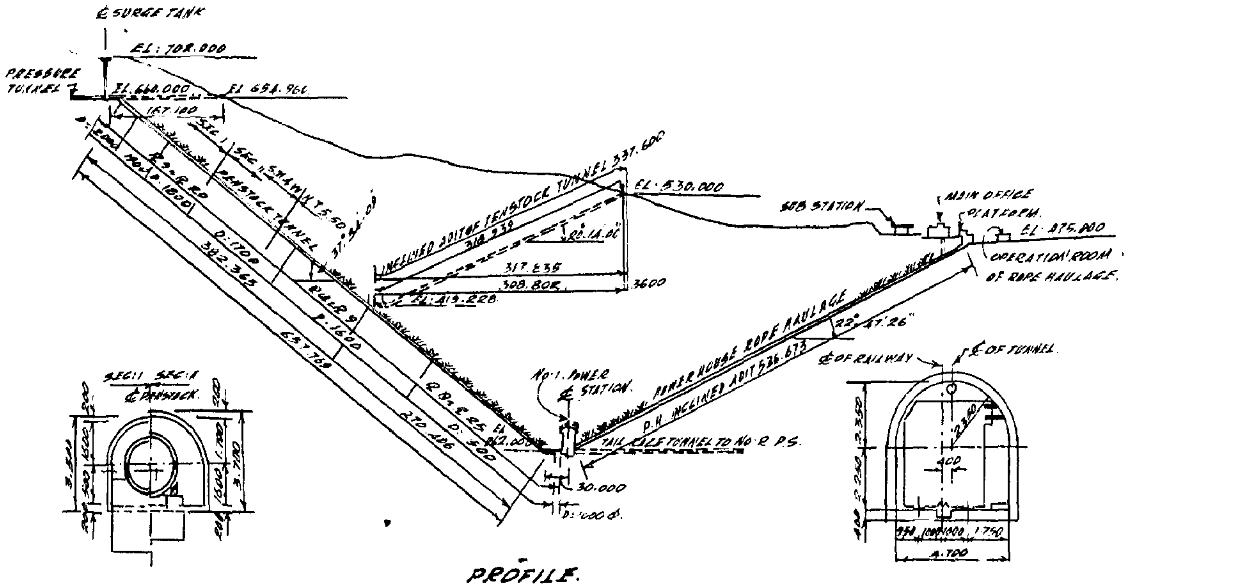
v) The maximum tailwater.

The first four items only is considered in alloting space within a unit bay for maximum economy. The last item is important only in thecase of schemes with Pelton installation.

The size of the unit varies with the area of the required waterway since, for a given plant capacity, the head and discharge remain the same (83); For example, a 300,000 kw plant with four 75,000 kw units will require less overall length than one with six 50,000 kw units. Though the width and height will be slightly larger for the four unit 75,000kw plant, the plant cost will be normally less. Plant size, therefore, should be largest feasible after considering limitations in transport and penstock sizes.

As discussed in Chapter V, vertical setting of turbines is preferred when large size plants are employed. With vertical setting, the plant waterway and the limits of substructure can be easily sketched (Fig.4c). The desired access to the turbine pit and support for the generator establish the generator elevation. At this stage, the generator rotor and shaft are located in a position that would permit moving it to and from the service area for installation and maintenance, the overhead crane is located. Variations in the generator (Umbrella or suspended), the transformer (single phase or three phase) whether inside or outside, turbine valves, whether inside or outside are then considered and the sketch modified suitably. According to Wolf (83), the Unit bay arrived at thus would never require a revision to accommodate the auxiliary equipment when the space requirements of the first four items are met. With the section roughed out, an ellipse of no stress is drawn outlining the proposed excavation. Assuming that the rock has no appreciable tensile strength this curve would represent the uppxer limit of the potential rock fall into the opening. The concrete arch for the roof is then drawn to permit operation of the overhead crane. Roughly the arch is designed to carry the weight of rock between the arch and the elliptical opening which may require revision after model studies.

The adoption of a vertical plant makes possible several alternative power station layouts, and there appears to be no universal trend in this respect. Many authorities favour the double floor design, where the top floor is at generator level and above which the exciters only (Aldeadavila, Bersimis, Poatina stations) and sometimes both the generators and exciters project (Morrowpoint plant and others). In this case most of the auxiliary plant is placed on the lower turbine floor from which access is gained to the turbine pits. Instruments and control panels are generally preferred on the upper floor. The weight of the generator and the hydraulic thrust of its turbine can be taken through the turbine fixed stay vanes to the foundations. Alternatively it can be taken through concrete or steel columns around the outside of the turbine casing. The latter arrangement permits better access to the turbine moving parts for maintenance but may be restricted to medium and high head machines (19). The upper floor can, if desired be completely (Gando Power Project - see.Fig.6t) or partly (Serve-Poncon station - Fig 6a) eliminated. If however only the floor at the turbine level is provided, the dimensions of the cavity usually have to be



PROFILE.

CROSS SECTION OF PENSTOCK TUNNEL.

CROSS SECTION OF POWER HOUSE INCLINED ADIT

FIG: 6(t) PROFILE OF No: 1 POWER STATION, SURGE TANK, PENSTOCK & SURFACE BUILDING.

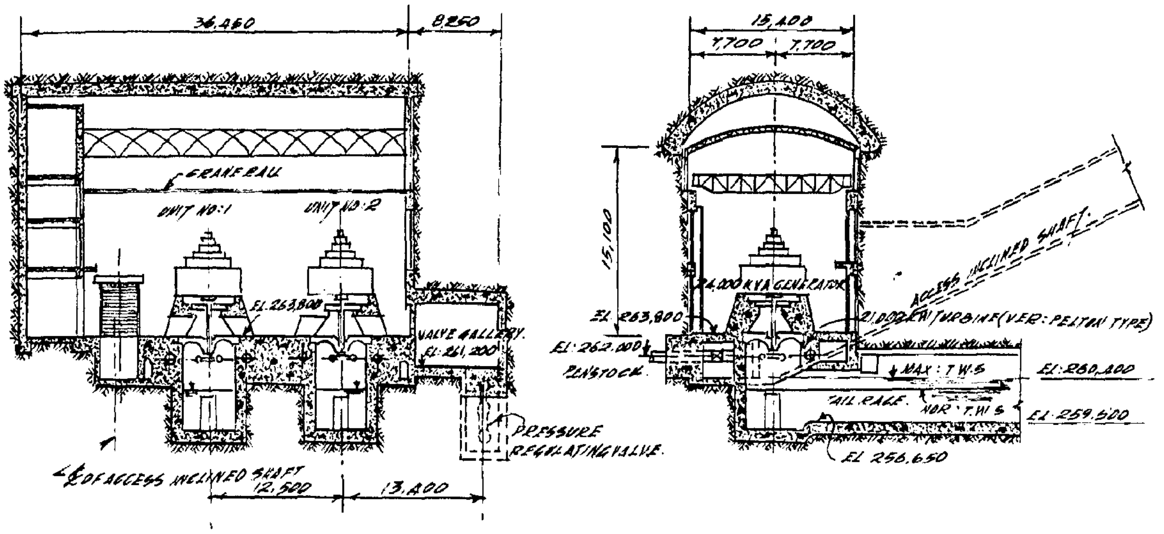


FIG: 6(t) LONGITUDINAL AND CROSS SECTIONS OF No: 1 POWER HOUSE.

GANDO POWER PROJECT.

greater (25).

With horizontal setting, the station requires only one floor upon which the auxiliaries are mounted and from which all the operation and maintenance work is carried out. With such setting, the machine emerge in general almost entirely above the machine hall floor. Thus, at the side opposite to the tailrace tunnel, there generally remains sufficient room for cable gallery in the sub-structure (Huinco station - Fig.6m. Room is similarly available under the floor between individual machine units. The auxiliary equipment is housed in the machine hall or elsewhere. When the span of the cavity is to be limited, auxiliary equipment can be arranged in cells excavated in the rock between and beyond the legs of the frames supporting the crane rail.

Conventional straight line layout is generally adopted for the power house cavity. Several alternative plant arrangements are possible as the plant is located underground. A curved underground layout has been adopted for the Ambuklao (15) power station due to geologic reasons. In curved layouts, the crane arrangements would be more costly, but may well be offset by reduced costs from shorter penstocks.

The longitudinal length of the station is decided upon after providing space for service and control bays. Before allotting space for this, consideration should be given for the expansion-contraction joints. In a conventional surface plant unit bay, expansion-contraction joints are provided at each side of the unit framing system while in underground stations this can be omitted (Fig.6v). When provided in multiunit layouts these need not be kept in a straight line, but may be offset if desired. Further, the cross walls will not be needed to resist tail race water loads and so, they can be omitted. Elimination of one of the cross walls will give saving in length of 3 to 5Ft. for every pair of units. The above provisions will permit a closer unit spacing and shorter the length of the station as compared with its surface counterpart (83).

Having fixed the unit ~~x~~ bay layout, the remaining layout features and their location are subject to a variety of economical arrangement. The service or erection bay is usually located at one end of the unit bay in a multiunit plant. When two stage construction is desired, two service bays at either ends are sometimes provided with provision to convert one among these as control room. Alternatively a layout with a central erection bay flanked by unit bays is preferred for two stage construction. In all cases the width of the service bay should necessarily be identical with the unit bays to permit movement of the powerhouse cranes over the erection area. According to an old rule of thumb, the length of the service bay is twice the length of the unit bay. In modern plants, the service bay is only 1.25 times the unit bay length. In final designs, the service area fixed thus is checked against the area required for rotor assembling or storing upper guide bearing bracket and access for truck-trailer unit needed for delivering the component parts of the generating machinery.

The control bay or control room is usually arranged as a separate feature of a multiunit layout. It may be located overground close to the transformers and the high voltage switching yard in low and medium stations (for Ex.Aldeadavila at one end of the unit bays) in a separate cavity near the centre of the unit bays or on the roof over unit bays. The central location with respect to the units is preferred by some authorities. The size of the control bay is dictated by the space required for the main control boards and access to them.

Some special techniques are employed for reducing the height and width of the cavity. Installation and removal of the rotor deserve more serious consideration than customary. The lesser space requirements for handling the Umbrella generator (see Chapter V) in lieu of a suspended type are particularly important. Even with suspended type, measures are taken to reduce the height of the cavity. In Bersimis Plant, the top of the generator housing to the top of the crane rail was held to about 16 Ft. as contrasted to surface plant installations where it varies between 30 to 40 Ft. for units of commensurate size (1,50,000 H.P) This is achieved by lifting the rotor from its spider rather than by the shaft. This method is employed at stornorrfors and Harspranget and in many other stations.

Reduction of power house height above the crane rail is possible if proper check is made in the selection of overhead cranes. In most of the stations, two cranes with lifting beam is employed instead of a single larger crane.

The cost of two cranes themselves may be greater than the cost of a single larger crane (10). However, two cranes with equalizing beam permit the top of the rotor shaft extended up between the cranes and the reduction of distance above crane rail to house the trolley reduces the power house height. In stations with small and medium sized units a single crane is provided whereas two cranes are mostly applied for handling heavy parts of large machines. Recent power houses in sweden and other countries are equiped with overhead travelling cranes to which turbine shaft governing the height of the machine hall could be attached by special yokes for lifting at an intermediate point below the flange at the end of the shaft. Compared to the conventional method of attachment a saving of 3 to 5 Metres could be attained in the height of the machine hall (49). In some swiss stations a gantry or portal crane is used eliminating crane beams and columns. The present day thought is to eliminate the high capacity station cranes, used during erection and very seldom thereafter, by the provision of built-in jacking facilities whereby the generator stator can be lifted to permit access and removal of rotor (37).

Ordinarily the power house cranes in underground stations are carried on beams supported on free standing columns (Jor station, Morrowpoint and others) independent of the rock walls. These column are designed only for supporting the crane beams. In some stations, columns are placed in recesses in the vertical walls as at Koyna station. When good rock is available the rail beam can be suspended from the roof arch as at Salamonde, Picote stations and several stations in Portugal. In such cases the roof arch is abutted into recesses in the rock-walls thus throwing the load into the walls and reducing the stresses in the vertical faces of the rock. Lateral stability problems exist when the columns are attached to the walls. Flexibility offered by columns is that they need not be located directly opposite one another in the machine hall and this arrangement affords a better utilisation of floor space.

Handling of the stop log gate in the tailrace also deserve due consideration. At the Formo and other developments, the main crane bridge extends downstream over the draft-tube stop logs so that the gates can be operated by the main crane. Another arrangement is to provide a special gallery, within the powerhouse cavity as in Ceanna croc station (Figure 6h) so as to use the powerhouse crane. Separate cavity downstream of the station with lifting arrangement is provided at Santa Giustina Stonorforrs and Korsselbranna stations (Figures 6u, 6g & 6v). Handling of the draft tube gates can be done from downstream surge chambers also as at Aldeadavila, Chute-des-Passes (Figures 6k & 6j) and in many other stations. Economical studies have to be conducted for the choice of the best alternative.

The roof arch is drawn to permit operation of the overhead crane and it is to be a designed to carry the weight of rock between the arch and the ellipteal opening (83). The shape of the power house roof may be semicircular (Chute-des-

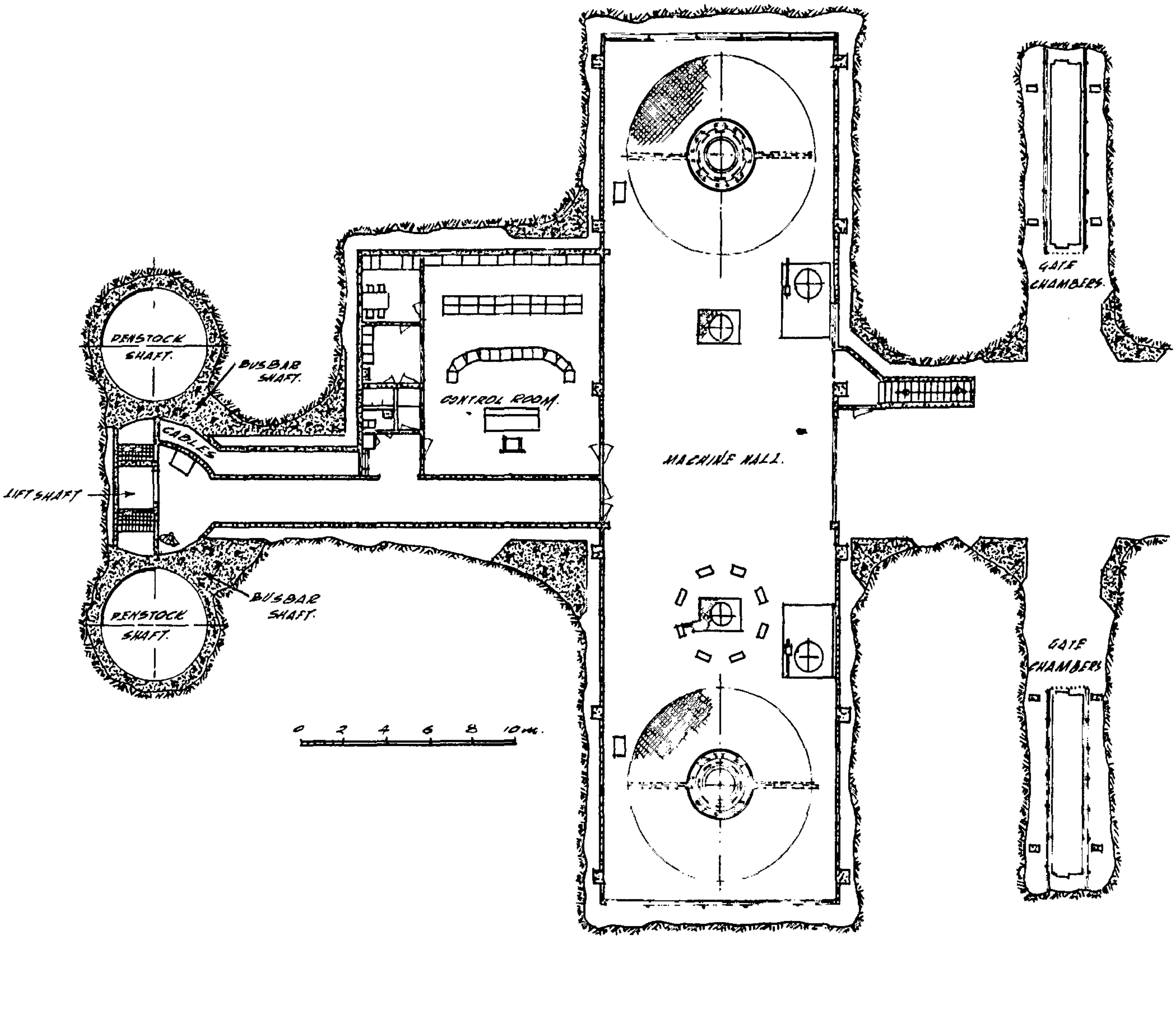


FIG: 6 (d) PLAN OF KORSSELBRANNA UNDERGROUND POWER STATION.

passes and many other stations) or elliptcal opening (Santa Giustina) or special shape as in Poatina (Fig.4b) station. The shape is primarily determined from stress considerations (see Chapter IV). Due consideration should be given for the shape when ventilation ducts and cables are to be taken through the space above the crane. In Santa Massenza station (Fig. 6r), an asymmetrical roof is provided for reducing excavation.

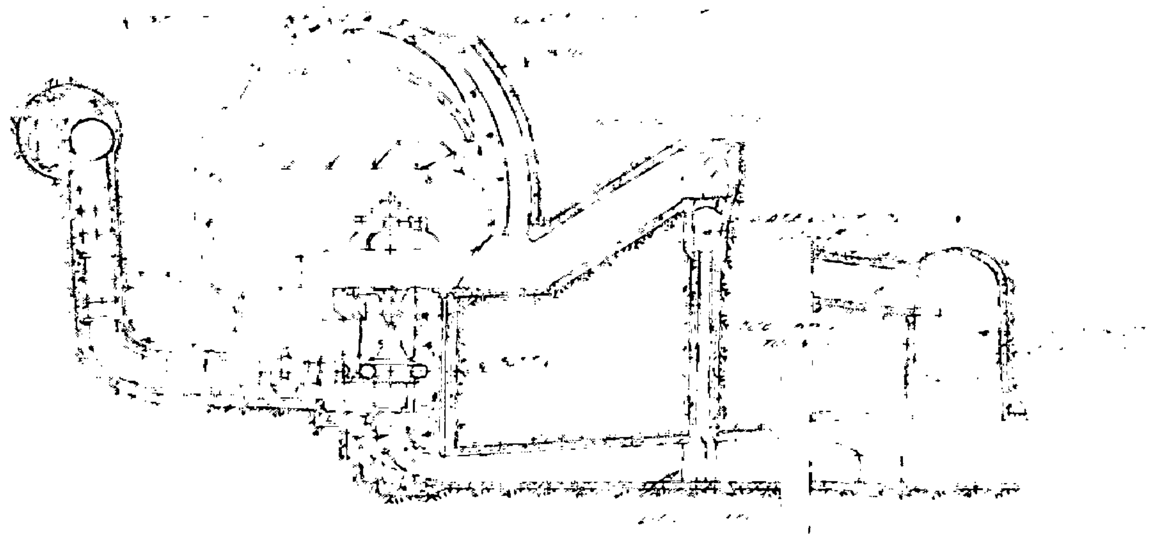
In sound rock, the roof arch is designed for only a part, in some cases as low as 25% of the estimated rock load. In the case of relatively weak rock structures, the roof arch is designed to carry the full load. In most of the stations the roof arch is monolithic concrete placed against the rock and grouted. Below the roof arch, a second arch is sometimes provided with a waterproof membrane on top as at Innertkirchen. In some stations, instead of a concrete arch, the roof is strengthened by ungrouted or grouted rock bolts and the rock surface is covered with wire netting and gunited. In such cases below main roof false roof is often provided which will prevent any seepage water from the roof entering into the machine hall. At Krangede station, inverted reinforced T-beams are used in the roof arch, so that an interspace between the rock and ceiling resulted, which picks up any infiltration of ground water. Norwegian engineers favour designs with concrete arched roof and the same located at some distance from the roof rock.

The roof loads are carried on thick walls of rock which are continuous except for penstock tunnels on the upstream side and the tailrace, cable and access tunnels on the downstream side (in most of the U.S.layouts). Some light inward deflection is to be expected as a result of redistribution of stress but most of this adjustment will take place soon after excavation, and any subsequent dimensional change should be too small to affect crane-rail spacing or machine setting. For these and other reasons, the walls of the power house are arched in the horizontal direction and comprise segments of cylinders with vertical axes (Lumiei station). To deal with higher rock pressures an elliptical (Santa Guistina) or egg-shaped cross section is adopted. A parabolic shape is adopted at Peccia (Fig.6r) while at Huinco, it is semicircular to withstand the rock pressure. In most of the stations, the walls are vertical. Ordinarily these walls need no support. Properly cleaned and sealed, granitic rock walls constitute a very attractive feature of many underground plants (Hojum in Sweden). Walls are sometimes lined with concrete. In many plants a second wall is constructed with an interspace between the rock and the wall through which air is circulated for removing accumulations of moisture. Such a solution is adopted at Brommat and Krangede and others. Norwegian engineers prefer to leave a space of from 1/2 to 1 metre between the rock and walls. The above technique is quite common in Norway and Sweden and Switzerland.

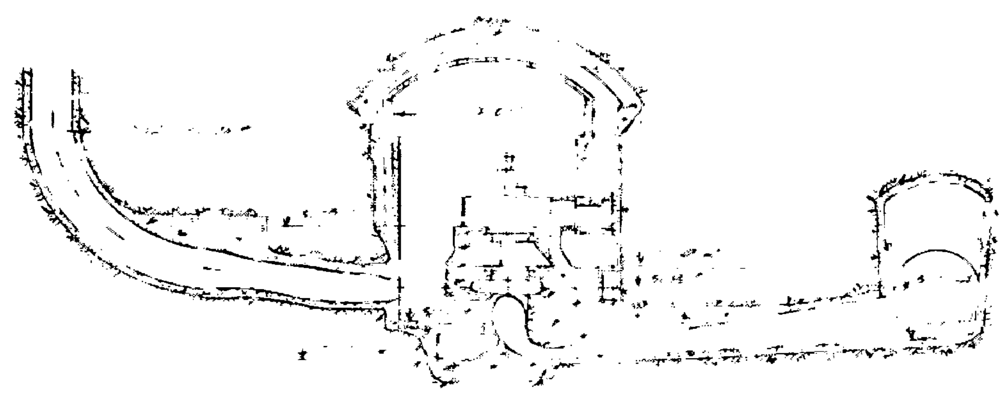
6.8.5 ACCESS TO THE POWER STATION:

An important element associated with underground power developments is means of access and exit. Traffic of the personnel for operation and maintenance and transportation of machine parts and other equipment should be ensured through tunnels or shafts. The section of the tunnel or shaft is mainly governed by the largest machine parts to be transported as a unit and the transport vehicle. some authorities prefer two access tunnels or one access tunnel and one shaft, located practically at the two ends of the machine hall, from the psychological view point. However, these are not always provided, for one, ~~as~~ cable ducts and other features can be used as emergency exit. The main access tunnel or shaft should be extended to erection bay in the station so that handling of the machine parts brought to the station can be accomplished by the Powerhouse crane (See Figure 61).

Access tunnels may be steeplyinclined (1 in 6.86 at Jor station). Initially such tunnels can be used as construction gallery as at Gando Project (Fig.6t). In most of the stations the main access is through tunnels with a slope ranging from 1 in 7 to 1 in 15 (The tunnel will be shorter when steep slope is adopted) and emergency exit through cable tunnels or cable shafts (in Aldeadavila and Crauchan the depth of such shaft is 1040 Ft. and 1200 Ft. respectively and these are partitioned for accommodating cables, ventilation ducts, stairway and lift). Access tunnels are usually preferred when large units are to be installed in the power station. In Miranda station (82 MW units, 3 Nos) main access is through a vertical shaft, 9 metres diameter and 63.8 m high compartmented to accommodate a stairway, a lift, busbars, cables and space for lowering



SANTA GIUSTINA (83 93 m)



SERF ARC (152 m)

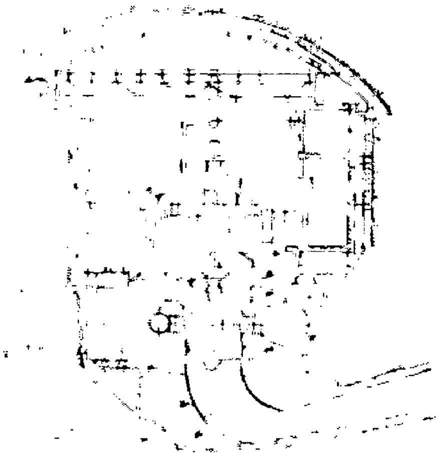


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FIG 6(U)

machine parts (79). For reducing construction costs in Korselbranna development (Fig.6v) a single excavation accomodates two penstock shafts and an open shaft accommodating a lift, staircase, busbar shafts and cables. In Salene development, a common tunnel is constructed for access and tailwater disposal. In this case two conditions exist (i) free flow conditions should exist in the tailrace tunnel and (ii) the difference between the elevations of the tailwater level and the machine hall floor should not be excessive, otherwise the economical combination of the two passages in a single tunnel would not be feasible. An interesting feature is seen at Haas Project (92 MW-Impulse units) in which the permanent access is through a vertical shaft 389 ft. deep and 18 ft. dia. accommodating lift, cables etc., while the emergency exit is through a walkway in a Gothic arch tailrace tunnel. The tailrace tunnel is used for permanent heavy maintenance access. The multiple use of the tailrace tunnel requires that the plant be shut down when loads greater than 2½ tons (shaft elevator capacity) must be handled. The capital cost saving in using the tailrace tunnel as the access tunnel is considered to greatly outweigh the possible but improbable impairment of operation (10).

From the above discussion, it is concluded that two independent means of access for personnel is considered necessary in underground stations. Vertical or steep shafts are provided to render the power house accessible in 'head' developments whereas horizontal or midly sloping tunnels appear preferable for 'tail' developments.

'D' sections are usually adopted for access tunnels. These may be lined or unlined depending upon rock conditions. M When good rock strata is available, only the invert portion of the tunnel is paved with concrete for heavy traffic. The sides and roof are strengthened by rock bolts.

Mostly shafts are circular in section. Rectangular or square sections have also been adopted. The section of the shaft or tunnel is decided after alloting space for the various requirements.

6.8.6 VENTILATION:

A permanent ventilation system is required for under ground power stations. such a system should (i) maintain a dust free atmosphere (ii) provide fresh air (iii) maintain a temperature to ensure confort of personnel and safe functioning of equipment, (iv) prevent condensation and keep exposed surfaces dry and (v) prevent objectionable or dangerous concentrations of air contaminants in accessible spaces.

The general design criteria adopted 82 are (i) rate of supply of outside air of 50 cu.ft/min/person, (ii) Temperature limits of 80°F and 65°F in control rooms and 90°F and 45°F in other spaces; (iii) provide good ventilation in battery rooms (15-20 air exchanges per hour at times of maximum gas emission), and toilets (5 to 10 changes per hour).

The data required for calculating the loads on the ventilation system are meteorological data, occupancy, internal heat loads and cavity sizes. In calculating heat transmission losses and gainst rock temperatures must be taken into account, which increases with increasing depth at a rate between extremes of 1°F per 70 ft. and 1°F per 100 Ft). (In the snowy Mountains the rate taken is 1°F per 100ft). Rock temperatures around the station are usually high enough to avoid any need to heat the station and for this reason it is not necessary to make provision on the generators for louvres which allow heated air into the station, as is common for surface stations located in cold places. The temperature of air supply should be at low temperature to absorb the heat losses from equipment without air temperatures rising above acceptable limits. To reduce the temperature of the incoming air, cooling equipment might be necessary. The maximum air quantity determines the size of the main supply fans, filters, air passages and other equipments.

A basic feature to be kept in mind in all times during the design of the ventilation system is the cost of space. Consideration must be given to the multiple use of space. In practice, ventilation passages between the station and the surface should not require extra excavation. Separate space will be required for ventilation plant rooms, but space used for air passages will usually be used for other purposes also.

Air is normally drawn from the surface through multipurpose shaft as in the Awe Project (6) and Kiewa Project (22) or through a separate ventilation tunnel as in Morrow plant (3) or through the main access tunnel as in Jor station

(11) and chute-des-Passes (42) or through the free flow tailrace tunnel as in Salanfe development. Stale air may be discharged through shafts or access tunnel or tailrace tunnel or cable tunnel. Some authorities prefer to discharge air through the access tunnels to carry exhaust fumes from the vehicles away from the station. In Jor station, the existence of a diesel engine exhausting to the tailrace determined the direction of ventilation as down the access tunnel and out through the tailrace. If intake of air is through the access tunnel a ventilation duct in the top section of the tunnel is to be provided. The tunnel is such cases are required to be enlarged to accommodate this duct also.

6.8.7 LIGHTING AND SOUND PROOFING:

With no natural light available, artificial illumination should meet (i) sufficient light intesity for inspection of machinery (ii)Illumination should be uniform; (iii) the light rays emitted by sources should contain the ultraviolet range of the spectrum in a proportion sufficient to replace, atleast partly, the germ killing effect of sun shine (49) and (iv) the ceiling, the walls, the floor and the machines should be painted a pleasing colour. For the optimum illumination, M. Mainardis suggests (i) in the machine hall 80 lux and in the control room 100 lux. with scattered light reflected by the ceiling and the side walls uniform illumination can be attained. Biological effects of illumination is ensured by the use of mercury vapour lamps. Large windows are installed at the interior walls and lamps mounted in the space between the rock and the slide walls, create the impression of sun-shile and such a solution is adopted in many Italian and Swedish power stations.

In underground stations, sound absorbing capacity of the wall is inadequate and the transformer noise is apt to be excessive (when transformers are also within the main cavity). Limit noise that could be tolerated is about 80 Phons. To reduce the noice to such a level various methods are adopted. In some cases, the side walls are finished with sound proofing materials & special provision is made in the design of transformers (see chapter V) so that the noise level can be minimised.

6.8.8 <u>OTHER REQUIREMENTS</u>:

A water tank should be provided within the station for supplying water for cooling purposes. The tank should preferably be located at an elevation permitting gravitational supply. At high head stations pumping may prove more economical because the power required for pumping into the elevator tank will be less than the energy loss caused by drawing off the cooling water from the pressure shaft. At low and medium head stations, it is more economical to draw off the cooling water from the pressure shaft.

Since the specific resistance of rock is much greater than than of soils close to the surface, earthing at underground stations constitute an intricate problem especially when the transformers are also in the cavern. The earthing should be designed in such a way so that the voltage gradient does not exceed the permissible limit along the underground passages and caverns (49).

Water infiltrating through the rock wall or dripping from the roof should be collected by gutters and discharged by means of a drainage gallery into the tailrace.

Auxiliary power supply to the station is given by taking an independent circuit from other stations. At Montpezat underground development auxiliary supply is obtained by installing a small Pelton wheel and connected electrical equipment.

When the power house is located deep underground workshop equipment should preferably be located within the main cavity. Such an arrangement will prove economical when the station access is through long tunnels.

6.8.9 TAILRACE TUNNEL:

The tailrace tunnel is one of the most important elements of an underground power scheme in its bearing on the overall economy. This is particularly true in the case of low-to-medium head plants where the cost of the tailrace tunnel may represent a substantial part of the overall investment. Tailrace tunnels with a gradient of 0.01 to 0.025 are usually constructed as unlined grade tunnels designed for a velocity of 6 to 8 ft/sec. The usual cross section is either horse-shoe shaped or egg-shaped or circular 'D' shaped or triangular section or rectangle with gothic arch may be adopted in poor rock.

Tailrace tunnel handles a fluctuating flow depending on load variations and hydraulically it can be designed in three ways as (i) to act as a free flow tunnel at all times (ii) free flow during steady flow and full flow by the surge waves

(iii) pressure flow during steady and unsteady flow. According to Lawton (37), tailrace tunnel upto 1500 or 2000ft. in length, the economical choice appears to be the first one. As the length increases a horizontal surge chamber in the form of an enlargement reduces the average size of the tunnel section and is economical for tunnel lengths of 7000 to 9000ft. For longer tunnels, surge shafts are more economical.

The type of turbine to be installed is also of importance. A free surface must be maintained under all operating conditions in the tailwater pit for Pelton wheel installations, and these pits are mostly connected to free flow tunnels. According to Mosonyi, there is no objection against linking the pit or a free flow initial gallery with a duct which discharges under pressure into the receipient. Both free flow and closed conduit tunnels are suitable in connection with reaction turbines.

Tailrace tunnels may be lined or unlined. Complete lining is not required unless the rock is of very bad quality. The question whether or not to provide a lining can be determined from economic studies. section should also be based on considerations of economics. When reaction turbines are installed in a station, water seal above the draft tube during low stages of the river is achieved by constructing a suitable weir at the exit of the tailwater tunnel. This will also prevent deposition of sediments in the tailrace tunnel.

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CHAPTER - VII

IDIKKI HYDRO-ELECTRIC PROJECT

7.1 GENERAL

Idikki hydro-electric Project is the largest power project taken up for execution by the state of Kerala. The project aims at utilising the yield available from 250 sq.miles of Periyar Catchment, the largest river basin in the state of Kerala, for power generation and irrigation. With a continuous power draft of 1440 cusecs over a head of 2185 Ft. the power output of the station would be about 213 MW at 100% Load factor. In the interest of long term planning, which envisages a super grid for the whole of India beginning initially with southern Grid (which is in operation now), the station is designed for ultimate operation at 30% load factor for purposes of functioning as a peaking station.

7.2 GENERAL TOPOGRAPHY & GEOLOGY:

The hills and mountains in the region form part of the western Ghat system. The general direction of the ranges is roughly north west-south east (figure 7a). The western slopes of the ghats are steeper and sheet rock exposed at many places is evidently due to heavier rainfall and better drainage. The eastern slopes are comparatively gentler. The strike of the foliations almost coincides with the pattern of drainage. Narrow intermittent ridges running short distances of 2 to 3 miles is a common feature of the topography.

A great part of the Periyar catchment is covered by rocks of the Archean Complex representing pre-cambrian forma-

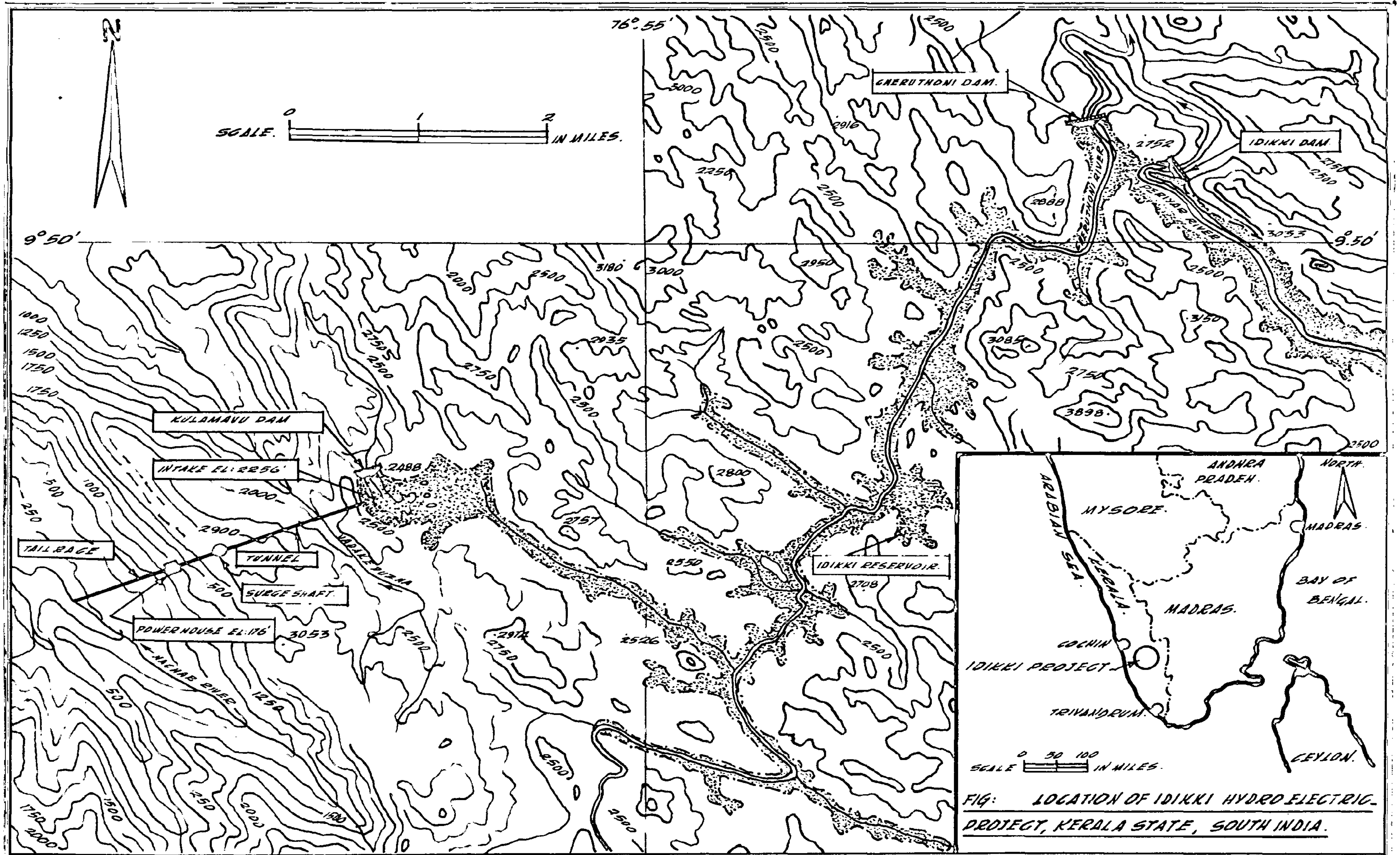

FIG: 7 (a). GENERAL TOPOGRAPHY AND LAYOUT OF IDIKKI PROJECT.

tions, of which gneisses, schists and charnockites are predominant. Gneissic rocks consist of a very heterogenous mixture of different types of granites intruded into schistose rocks after the latter were folded, crumbled and metamorphosed. The other important formations generally met with is laterite which is a porous pitted clay-like rock with mottled colours depending upon composition.

The region being a part of Peninsular India is one of high stability, the mountain building movements having ceased to be active long ago. Very rarely feeble earthquake shocks are felt, the reason probably being the fault along the Western Coast has not completely attained equilibrium.

1.3 PROJECT PROPOSALS IN BRIEF:

The project as originally contemplated by Central Water and Power Commission, Government of India envisaged construction of three dams viz. an arch dam 540 ft. height at Idikki across Periyar river, a straight gravity masonry dam 420 ft. height across Cheruthoni river and an earthen dam 245 ft. height across Kilivallithodu at Kulamavu, to form a combined reservoir having an effective capacity of 35600 Million Cft. at F.R.L.2385 ft. The dead storage provided was 24280 M.Cft. below the minimum draw down level fixed at 2300 ft. With these three dams, the yield from the combined catchment of 250.7 sq.miles estimated at 48045 M.Cft. would be controlled. The power waterway consisted of a tunnel 8646 ft. long with its intake located on the Kilivallithodu. The tunnel ends in a surge shaft wherefrom the water is led to a valve house through four pressure tunnels. From the valve house eight penstocks each 5575 ft. long then carry the water through a drop of 2000ft. into a surface power house where 8 turbines each of 100 MW capacity were to be installed with nozzle level at 175 ft. The tailwaters were proposed to be led through a tailrace channel into one stream Nachar, which is a sub-tributary of Muvattupuzha river.

Though the project was sanctioned for execution, some major changes in the project features were thought of by the Kerala state Electricity Board during the pre-construction stage. A very significant change from the C.W. & P.C.'s report was the decision to locate the power station underground. studies also indicated some economy in the use of larger units and it was proposed to install six units each of 120MW in the underground power house. Further studies indicated that the effective storage capacity of the reservoir could be increased to enable a higher degree of utilisation of the water power potentialities of the Periyar basin. It was consequently proposed to raise the full reservoir level upto elevation +2403 and to lower the minimum reservoir level down to elevation +2280, thus increasing the live storage to 54000 M.Cft. This further necessitated an increase in the capacity of the generating units to 130 MW. The design maximum flow for the above units would be 4950 cusecs and the probable load factor ranging between 0.29 and 0.30. Thus the preliminary proposals of the Kerala state Electricity Board involved (Fig.7a) creation of a reservoir by constructing three dams viz.

(i) an arch dam 550 ft. height across Periyar at Idikki; (ii) a straight gravity concrete dam 445 ft. height across Cheruthoni river and (iii) a masonry gravity dam, 250ft. height across Kilivallithodu at Kulamavu. The power tunnel with its intake on the Kilivallithodu is 6500 ft. long and 22ft.diameter. It is lined with concrete and horse-shoe shaped. The tunnel ends in a surge shaft from where the pressure shaft system take off as a single conduit upto a valve house located about 300 ft.away from the surge shaft. Just upstream of the valve house, the single conduit bifurcates into two pressure shafts each 12.5ft. diameter and these shafts aligned at an angle of 45° to horizontal. Two underground caverns were proposed, one to accommodate the turbine valves and the other to accommodate the turbines, generators, transformers, repair bay, control room and other connected equipments. Both the chambers were located deep underground accessible by a tunnel about 2500ft. long The machine hall proposed was 520 ft. long, 80fft. wide and 115 ft. high and the valve house about 350 ft. long, 50 ft. wide and 25ft. high. The tailwaters were proposed to be led through a 31ft. diameter horse shoe unlined tunnel of 4300 ft. long into Nachar.

7.4 ECONOMICS OF UNDERGROUND LAYOUT:

The decision to go underground was based mainly on the economical considerations of such a scheme, the main factor being substantial reduction in the quantity of steel for the penstocks as a result of the participation of the rock to withstand the hydrostatic pressure. The underground layout outlined above was proved to be cheaper by about Rs.3.5 crores due to (i) reduced length of the pressure conduit (ii) reduced length of the high pressure conduit system beyond the surge shaft, tends to result in lower overpressure due to water hammer and results in simplified governing conditions. Reduction in length of conduit brought down the total quantity of steel required for the high pressure conduit from 16200 tons to 4500 Tons. A major factor contributed to the reduction in quantity of steel required is by replacing three surface penstocks by one single pressure shaft. (iii) Costly and difficult anchors are avoided, (iv) construction work at underground station can continue without interruption even under the severest weather conditions. (v) Property to be acquired in the case of underground station was far less. Apart from the above, operational savings could be achieved due to smaller head losses on account of the following factors:

i) Reduced length of Power tunnel;

ii) Reduced length of pressure shaft;

iii) Replacement of three surface penstocks by a single pressure shaft.

As against the above economical advantages, the additional expenses involved in the underground layout were:-

i) Increased cost of the power house;

ii) Cost of cable tunnels, ventilation tunnel;

iii) Cost of construction of part of the tailrace tunnel and

iv) Cost of access tunnel.

A statement showing the comparative costs of surface and underground system for the project isgiven below:

No.	ITEM.	SURFACE SYSTEM Rs.Lakhs.	UNDERGROUND SYSTEM Rs.Lakhs
1.	Head race tunnel	156	115
2.	Surge shaft	35	22
3.	Valve house	2	6
4.	Penstocks/Pressure shafts.	480	170
5.	Power station.	75	145
6.	Tailrace tunnel	-	82
7.	Approach tunnel	-	25
8.	Cable tunnels.	-	6
9.	Ventilation tunnel	-	3
10.	River training works	-	20
11.	Capitalised cost over 50 years of revenue loss due to friction in Power tunnel.	33	25
12.	-do- in Penstock/Pressure shafts.	260	72
	Total:-	1041	691

Savings for underground layout....Rs.350 lakhs.

Besides economic considerations, it was considered better to locate the water conductor system and the power station underground so that danger due to land slip from steep slopes is avoided. Pending detailed investigations, it was tentatively decided to locate the power station underground.

The ~~s~~ various proposals for underground station layout are discussed in Chapter VII and the details given therein are mostly based on the recommendations of the

Consultants Messrs.Surveyer, Nenniger and Chenevert Inc., Montreal, Canada.

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CHAPTER - VIII

UNDERGROUND LAYOUT PROPOSALS

8.1 GENERAL LAYOUT CONSIDERATIONS:

The general arrangement of the power waterway (Fig.8a) was tentatively selected as the shortest route ie., an alignment close to a straight line from the intake located in Kulamavu pool (which was an obligatory point) to the portal of the tail race tunnel near the Nachar stream. Keeping in view of the project requirements, several proposals were considered. In the initial stage itself few limitations were imposed on the range of alternatives. A 'head' development necessitated locating the power station close to the Vadake Puzha. A 'head' or 'intermediate' development was ruled out on the ground of economical or technical considerations (length of tunnel system and height of high voltage cables, liner erection problems, external pressure design requirements). Investigations were therefore concentrated on 'tail' development. Alternatives were considered in 'tail' development, also. One was to have the power station right below the surge tank so that the penstock shafts will be vertical. This alternative was also ruled out due to the long length and height of the high voltage cables and liner erection problems. Besides these considerations, savings in cost of pressure shaft was overcome by increases in the cost of adits, access road and tailrace tunnel. In consideration of these aspects, the most economical layout dictated the keeping of the underground penstocks as close to the rock surface as allowed by geological conditions and cover

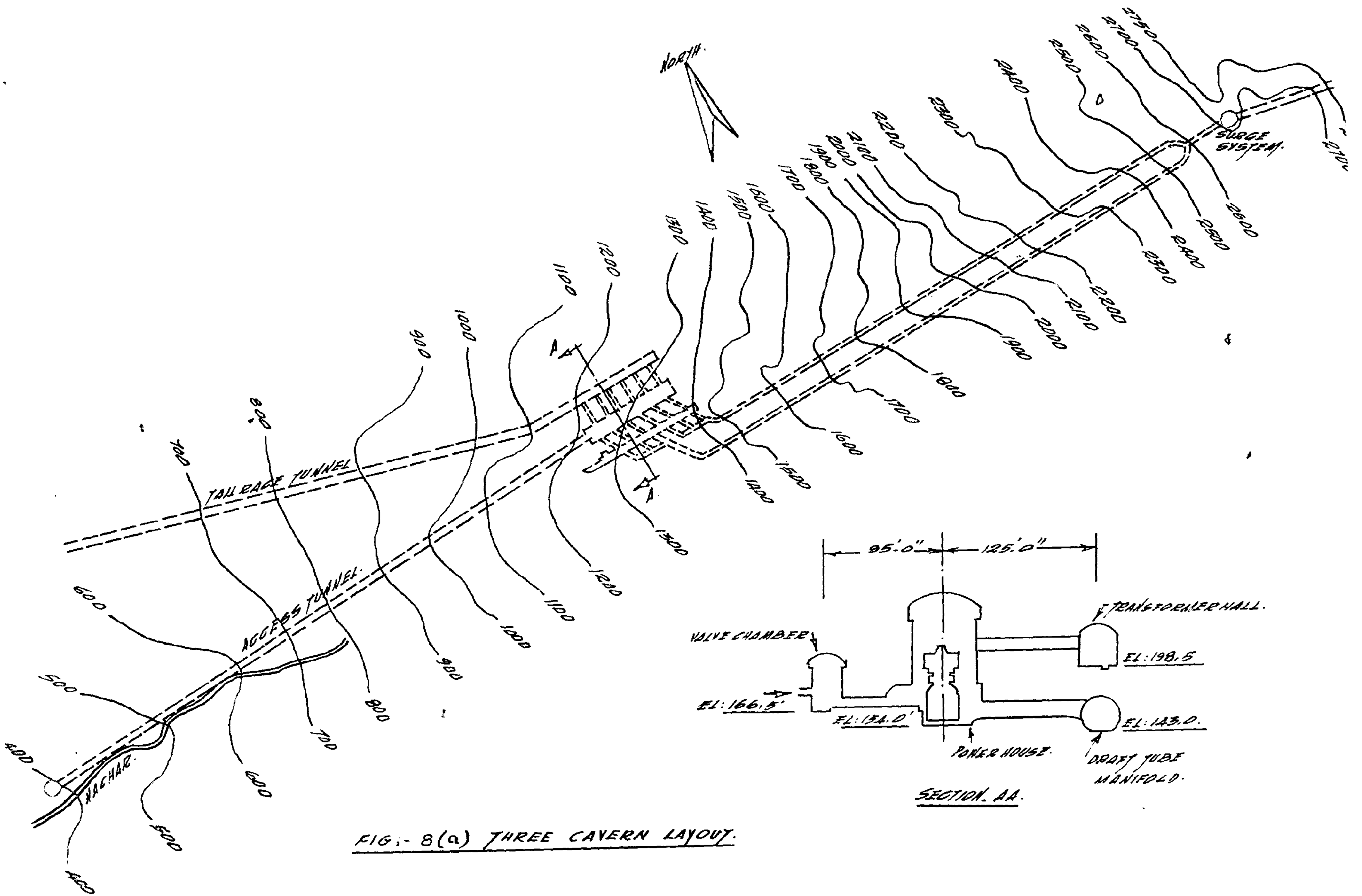


FIG:- 8(a) THREE CAVERN LAYOUT.

requirements. A 'tail' development was therefore chosen with sloping pressure shafts. (Fig.9a & 8b). Various studies conducted for finalising the layout for Idikki station are discussed below.

8.2 LAYOUT PROPOSALS:

Having fixed the intake point, a suitable site was selected for locating the surge shaft entirely underground with adit facilities. From the surge tank point, a tentative alignmentfor the pressure shaft was adopted assuming a minimum rock cover of 50% of design static head (a conservative value) at any point. To insure self mucking and to facilitate liner erection, the pressure shaft was aligned at an angle of 50° to the horizontal. The power house was also located tentatively right at the foot of the penstocks rather than to have an extended horizontal penstock section intervening. In so doing, additional costs of the downstream access or tunnels were more than compensated for by the corresponding savings on penstocks.

8.3 GEOLOGICAL STUDIES:

studies indicated that the entire area from the intake through pressure shafts, power station until the tailrace outlet, the rock type consisted of gneisses containing occasional intrusions of granite.

The kind of rock is granite in general, consisting mainly of two types: (a) Paragneiss; (b) granite gneiss. The paragneiss consists of alternating light and dark bands or layers and has a thickness from 6" to more than 10ft. The dark bands of layers are rich in mica and other ferromagnesian minerals and when thick or wet become susceptiable to alteration and often represent weak zones which require treatment in the tunnel. In contrast, the light bands which are rich in feldspar and quartz and lack mica, are strong not susceptible to alteration and behave almost like granite.

The granite gneiss is pink in colour, massive, slightly foliated and less jointed and stronger than the paragneiss. This rock formation appears in larte bands or sections, more than 100ft. in length in tunnels. It is structurally sound, represents minor weaknesses and requires no support in general.

The most striking features are the abrupt chain of mountains and its cliff at the surge tank location and the huge charnockite chain of mountains at the Idikki dam site. These features show a NW pattern.

Based on surface geology, tentative locations for the headrace tunnel, surge shaft, penstock and powerhouse were selected (fig.8a). But for a short distance near the intake and Vadake Puzha depression, the tunnel alignment provided acceptable and sufficient cover both horizontally and vertically. The exposed granite gneiss at the edge of the rock cliff near the proposed location of shaft was considered excellent. In the pressure shaft alignment, the direction of the foliation of the granite gneisses strikes at right angles and dips 45^{o}-60^{o} to NE or towards the hill. The 50 degree angle proposed is therefore considered ideal for tunnelling. Rock in the powerhouse area is granite gneiss frequently disturbed by the alternating mafic and feldspathic zones and degree of foliation or gneissosity. No faulting is anticipated in the area but x jointing and foliation is a major concern and its strike N30°W follows the rock ridge and dips 50° to 60°E towards it. Jointing is quite common in the area and its strike crosses the foliation planes and rock ridge and dips to the north from 45 degrees to 80 degrees angle. The major jointing system strikes N 25 degrees and dips 80 degrees to the south or North. The power station is thus oriented based on surface geology so that its longitudinal axis is at right angles to the foliation planes or the jointing planes. Any change required in the alignment is suggested to be done after reaching the proposed power house site through the access tunnel and after driving an adit in the power house area. The major tends of structural weakness in the rock as obtained from statistical analysis of the lineations in the access tunnel are:

1. N 39°W Dip 65°NE (Foliation) 100%
2. N 45°W Dip 52°NE (Joints) 49%
3. N 19°E Dip 67°NE (Joints) 27%
4. N 43°E Dip 36°SW (Joints) 24%
5. N 45°W Dip 66°W (Joints) 24%

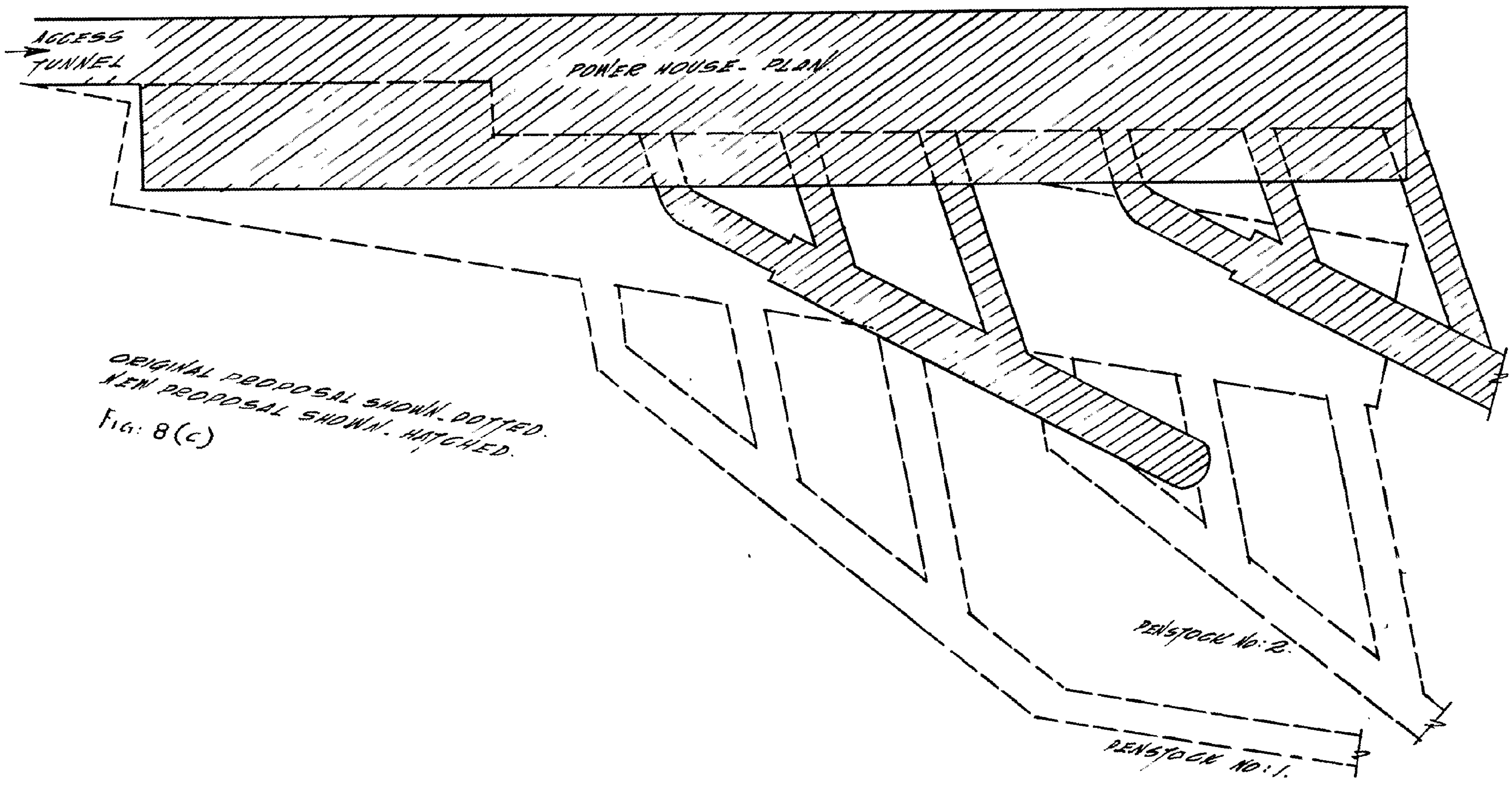
The above mentioned lineations which are the NW foliations or bedding planes, the NW jointing system, and the local geological mapping helped to determine the positions and orientation of the power house chamber. The geological information obtained from the construction of the access tunnel leading to the power house indiwated that the rock types and structure on surface are identical to the underground features.

The access tunnel leading to the power station has most of its length through Paragneiss, intruded from time to time by large sections of granitic gneiss. Two zones in the alignment was badly deteriorated by alteration with close jointing and fracturing. Based on these investigations, the axis was shifted by 10 degrees (Fig.8c) so as to make the long axis of the power house at right angles to the foliations or the bedding planes. The foliations or bedding plane directions are not only advantageous to the powerhouse itself, but are also at right angles to the two penstock.

The excavation of the access tunnel proved that the rock conditions were excellent and this allowed to take some radical changes in the power house design. Instead of more than one cavern as in Alternative-I and Alternative-II (Fig. 8i & 8j) a single cavern to contain the spherical valves, transformers and generating machinery was structurally found feasible.

8.4 ECONOMIC STUDIES:

8.4.1. POWER TUNNEL: The power tunnel from the intake to the surge chamber for the first half of the distance from the intake is under nominal rock cover requiring lining to carry a large portion of the internal pressure. The downstream portion of the tunnel could be left unlined as its alignment is through good rock. However, economic studies indicated that the enlargement of the tunnel to compensate for roughness was less attractive. Therefore a lined tunnel 23 ft. diameter



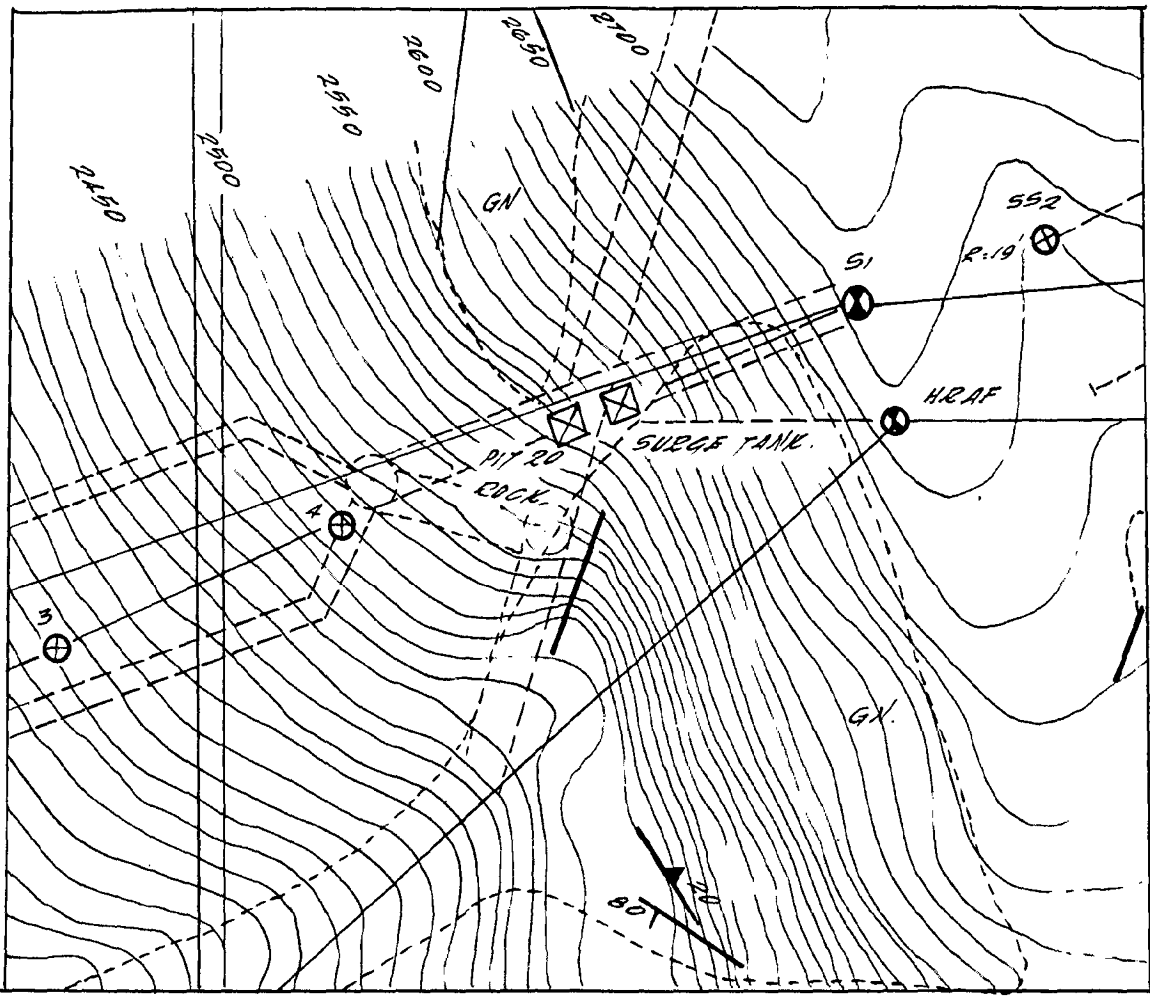
ORIGINAL PROPOSAL SHOWN DOTTED.
NEW PROPOSAL SHOWN HATCHED.

Fig: 8(c)

horse-shoe shape was adopted.

8.4.2 SURGE SHAFT: The surge shaft is located right at the head of the penstocks. Economic studies were conducted taking different types of surge systems; viz. (i) surge shaft with lower and upper expansion galleries; (ii) Differential surge shaft; (iii) Restricted orifice surge shaft with or without expansion galleries. Comparison is done on economical and operational advantages; limitation of the up and down surges, efficient damping, stability and effective protection of the headrace tunnel. The dimensions of the shaft has been fixed from the Thoma's minimum sectional area for incipient stability using a safety factor of 1.6 to the cross sectional area. The diameter required is found to be 27.5 ft. and the dimension of expansion gallery is fixed to 23 ft. The differential type is found to be too expensive. A restricted orifice shaft with upper and lower expansion chambers gave best results. With the proposed location of surge shaft, an adit at top could be provided which would ultimately be converted as the top expansion chamber. In the final layout, in order to get good rock throughout the depth of the shaft, a lateral shift of the shaft (Fig.9d) was found necessary.

8.4.3 PRESSURE SHAFT: Economical studies taking into account of civil works costs, capitalised revenue losses due to head losses and capitalisation have led to two 12'6" diameter sloping shafts which compared favourably with the largest high head power shafts in the World. A single shaft would have to be in the range of 18 Ft. diameter and welding with the required



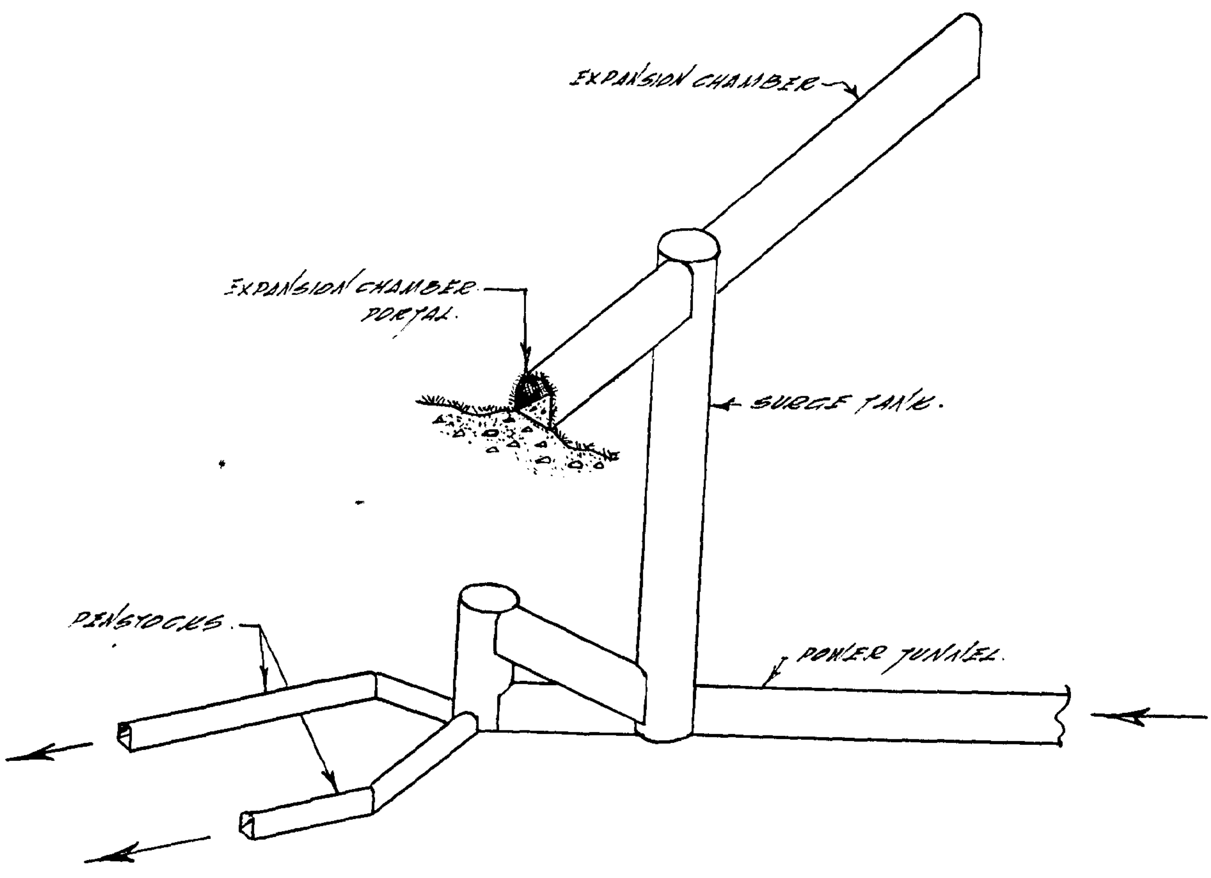


FIG: 8(d) ISOMETRIC VIEW SHOWING RELATION OF EXPANSION CHAMBER, SURGE TANK, POWER TUNNEL AND PENSTOCKS.

corresponding thicknesses necessitated very elaborate techniques and therefore abandoned. In addition, considering the development in two stages, a single penstock required large investment also. The two shafts were tentatively aligned so as to get a minimum rock cover of 50% of design head at any point. These shafts were also aligned parallel in plan upto the power station.

From the surge shaft upto the upper manifold, 20Ft. diameter would theoretically represent the most economical diameter. But considering the short length of the section and the fact that it will have to be excavated anyway to the head race tunnel section, 23' diameter has been maintained. This will save on concrete backfilling and avoid a transition from 23 Ft. to 20Ft. The liner in this section is to be designed for the external pressure on the basis of Amstutz theory. From the bifurcation to some 700 Ft. downstream, external pressure was still the governing factor. Below this upto the manifold, the liner was to be designed for internal pressure. The manifold and a short length of the penstocks was to be designed for internal pressure considering no rock participation.

In the arrangement of feeder pipes near the power house end, two alternatives were considered:

i) The usual arrangement with each penstock feeding three turbines through a three branch manifold.

ii) A more elaborate arrangement where the penstocks diverge near the power house and feed the machines from either side of the power station. In this case each turbine would

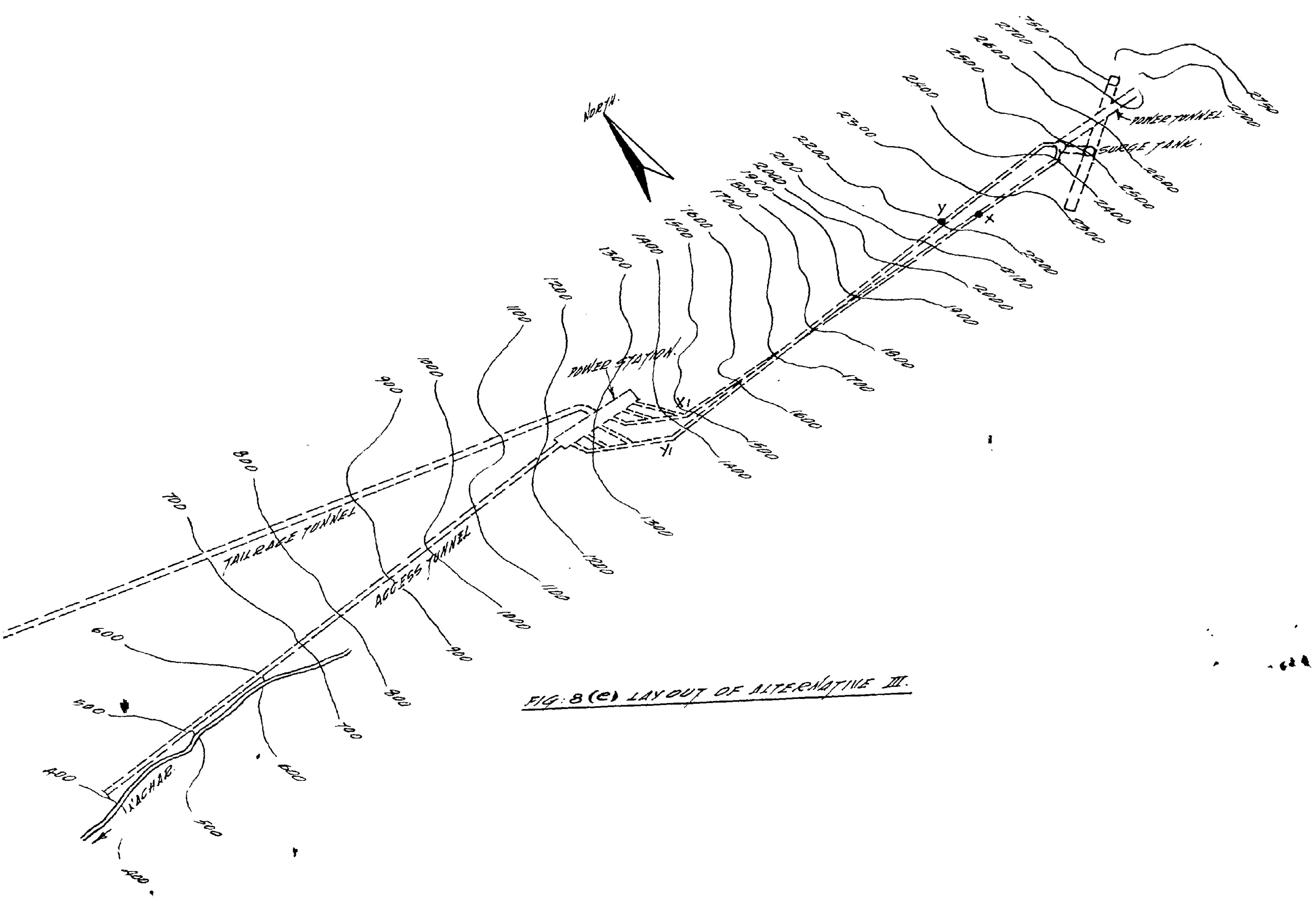


FIG: 8 (C) LAYOUT OF ALTERNATIVE III.

have three jets supplied by one penstock and three by the other one.

While alternative (ii) has the advantage of lending itself to more efficient operation at half load by feeding three jets only from one penstock, such a scheme has not been used in practice. Electricite de France is contemplating the use of this scheme for their Mont Cenis plant but it is still only in the design stage. This proposal did not offer any appreciable economy particularly when the project is proposed to be developed in two stages. Alternative (i) on the other hand offered better flexibility in operation and simplification in construction and therefore the same was selected for further study.

In the alignment of pressure shaft, it is of obvious interest to go deep enough underground to insure minimum rock cover required. By going deeper the liner would still have to be designed to the yield stress in considering internal pressures. By going deeper, the requirements of external ground water pressure will govern the design, especially in the upper sections of the penstocks. Initially (Alternative-I) the design was made with 50% design head as rock cover (Fig.8a & 8b). Before finalising the design, an access adit at about middle of the shaft was driven for detailed investigation and later to be used for penstock installation. It was found that the type of rock and joint patterns were similar to those described in the case of access tunnel to the power house. From these studies, it was found possible to make a change in the alignment of penstocks. The inclined part was shifted downstream and it was relocated as close as possible to the manifolds in order to reduce the length of the lower horizontal part. With this arrangement (Alternative III) the penstocks will cross each other as shown in Figure 22 8e & 8f) and the same at higher elevation will approach the ground surface so that thereduction of external pressure will reduce the thickness of steel. The saving due to reduction of external pressure and the elimination of the lower horizontal portion of penstock worked out as 10% of the total cost of the penstocks. Steel used in the design is of three types: (i) Mild steel 30,000 psi yield stress (ii) Intermediate steel 50,000 psi yield stress and (iii) High tensile steel 90,000 psi yield stress. In the design, the internal pressure at the turbine nozzles was taken according to maximum static head plus 10% increase due to water hammer effect. Design external pressure due to water percolation and infiltration in the rock was assumed corresponding to a water head equal to the vertical distance between ground surface and penstock axis. Whenever the rock participation can be taken into account, the required thickness was calculated allowing the steel to work upto the yield stress without rock participation which means a minimum safety factor of 2 to the yield stress, since the rock participation is found to be atleast 50%. Wherever the rock cover is less than 30% of the internal head ie. near the power house cavern, the required thickness was calculated using a 2.4 safety factor on the yield stress without rock participation. A transition section between thickness calculated with and without rock participation was established, with thickness varying linearly between values at the ends of it. The thickness varying required to prevent buckling of the steel liner was calculated according to the Amstutz curve using a nominal 1.1 safety factor on critical thickness. An increase of $\frac{1}{16}$" was given to the design thickness as corrosion allowance.

In the case of penstock manifold layout, studies indicated that 60° take off and straight connecting pipes represented the best arrangement. From economic studies it was found that 45° and 60° take off are equivalent while 90° take off lead to over Rs.10 lakhs additional actualised cost. Therefore 60° take off was selected on economical and practical considerations.

Based on the design principles discussed above, three alternatives were considered:

i) Two pressure shafts running parallel and these end in a valve chamber located parallel to the power house chamber (Fig.8a).

ii) The two pressure shafts cross near about the power station with the location of surge shaft at S_1 (Fig.8d).

iii) The pressure shafts cross near the power station with the location of the surge shaft downstream of S1 (Fig.8e).

Comparing the cost of Alternative II with Alternative-III there was decrease in cost for Alternative-II, eventhough there was increase in the length of headrace tunnel. Alternative II was found to be cheaper by about 10% than

Alternative-I and therefore Alternative II was adopted.

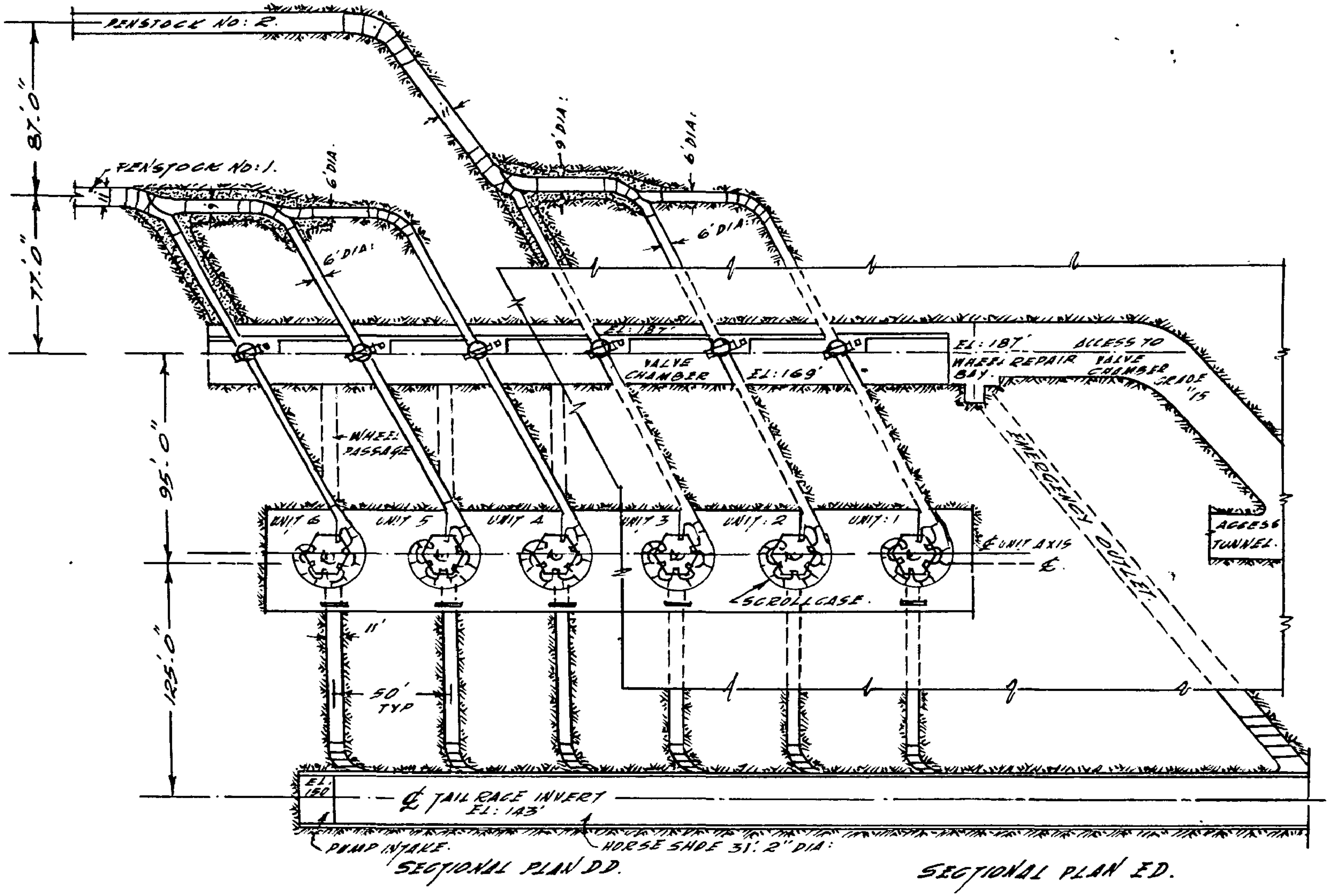
8.4.4 POWER STATION:

Generally, an agrangement with penstocks and tailrace aligned parallel to the power house axis would be most economical However, in most cases, geological considerations control the alignment of the underground caverns. Before peaching the power house site by adits in consideration of the general pattern of faults, joints and foliation in the power house area, two alternatives are considered. Alternative I (Fig.8g & 8i) provides three independent caverns one each for accommodating turbine valves, generating machinery and transformers. In alternative II (Fig.8h & 8j) there are only two caverns, one for turbine valves and the other for accommodating the generating machinery and transformers. In both the cases the turbine valves which are of spherical type 72" diameter are located in a separate cavern for reasons stated below:

i) The overall width of the powerhouse is mostly governed by the turbine manifold dimensions. To locate the shut off valves in the main cavern would increase its width considerably for the whole width.

ii) In case of bursting of one of the pipes, flooding of the power house can be avoided by providing an emergency outlet from the valve hall to the tailrace.

iii) Driving of the penstock shaft being a critical item in the construction programme, the valve hall can be used as a fast access to this part of the job without interference with the powerhouse excavation works.

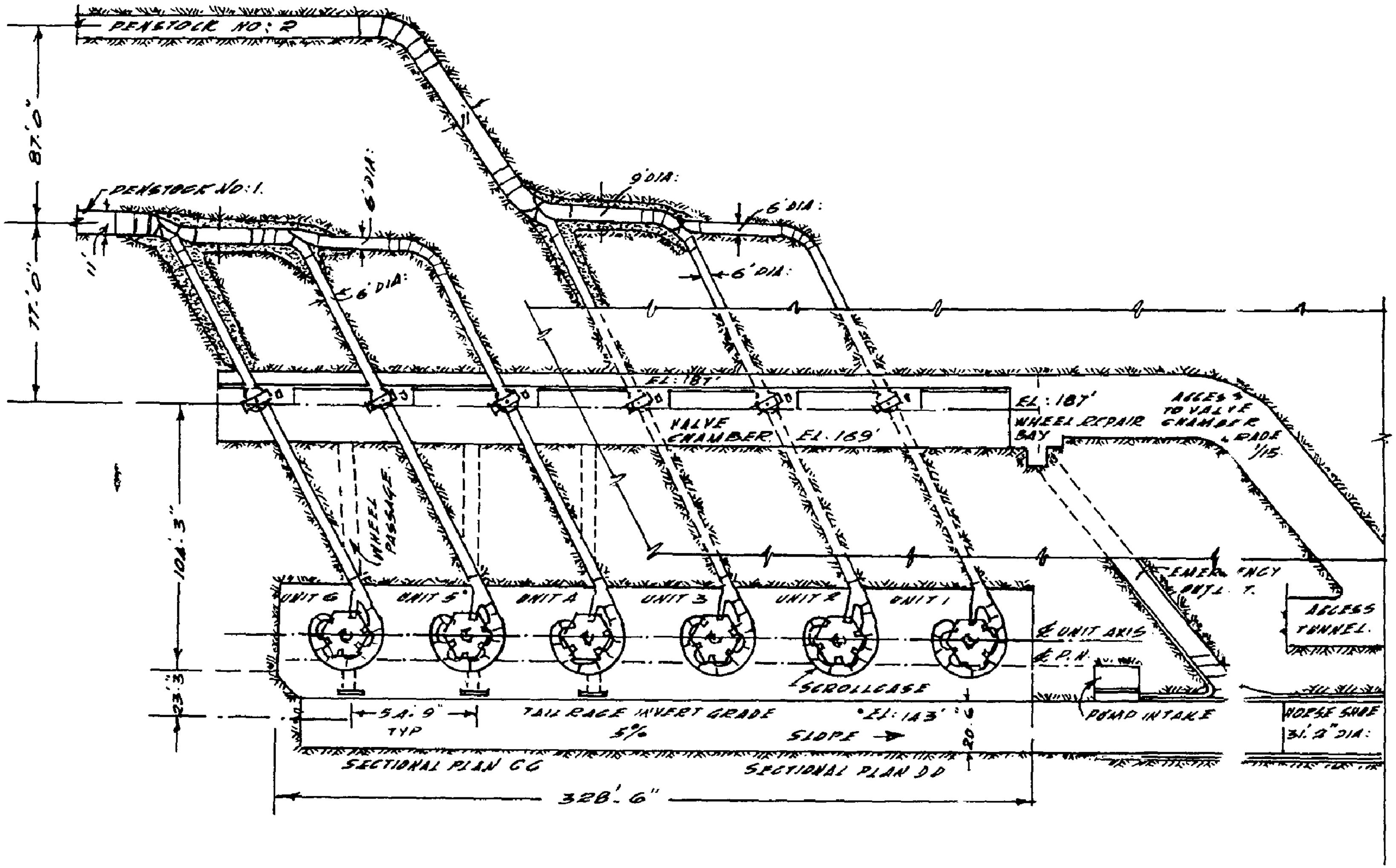


IDIKKI UNDERGROUND POWER HOUSE.

PLAN

ALTERNATIVE. I.

FIG:-8(g)



IDIKKI UNDERGROUND POWER HOUSE.

PLAN

ALTERNATIVE II.

FIG :- 8(h)

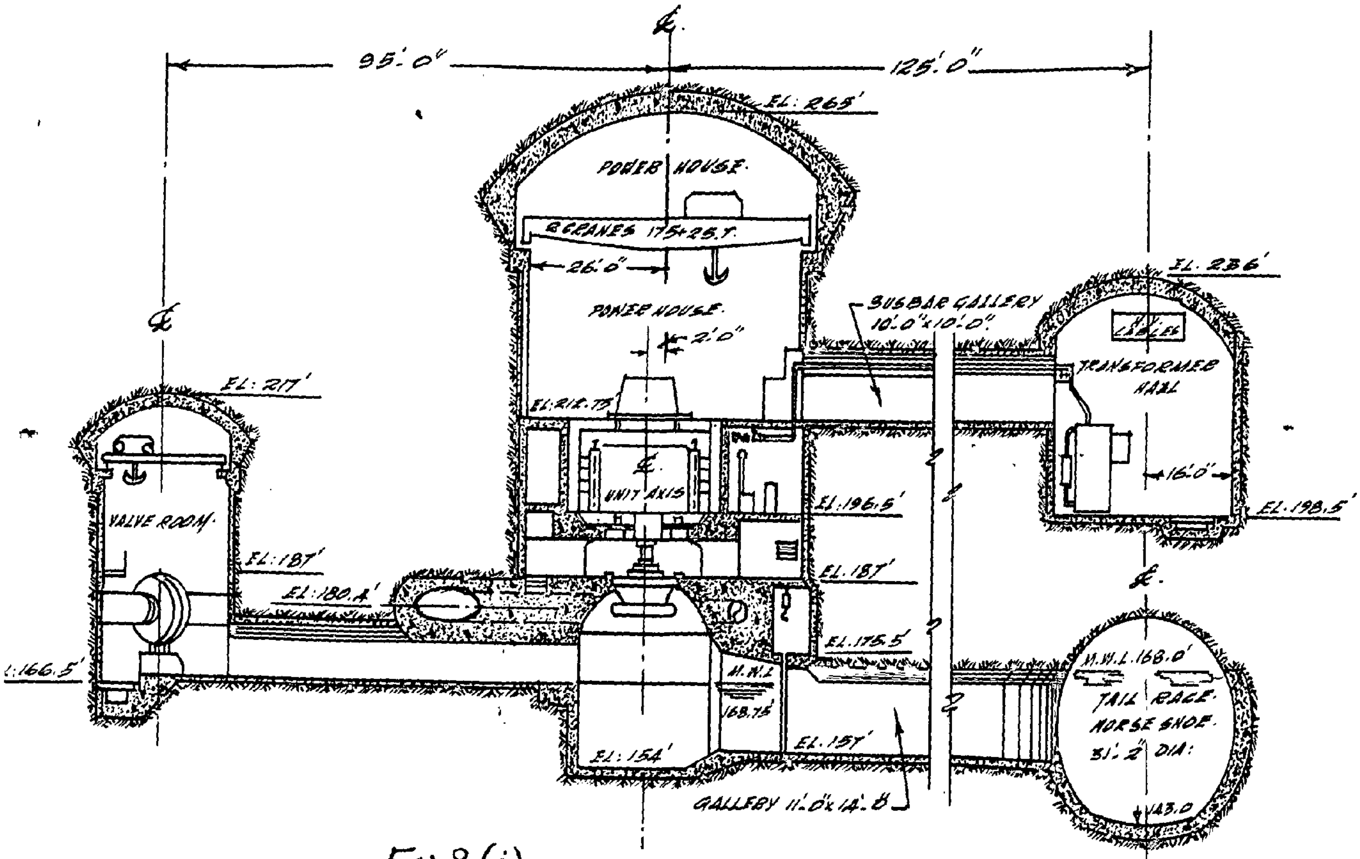


Fig: 8(i) ALTERNATIVE NO: I.

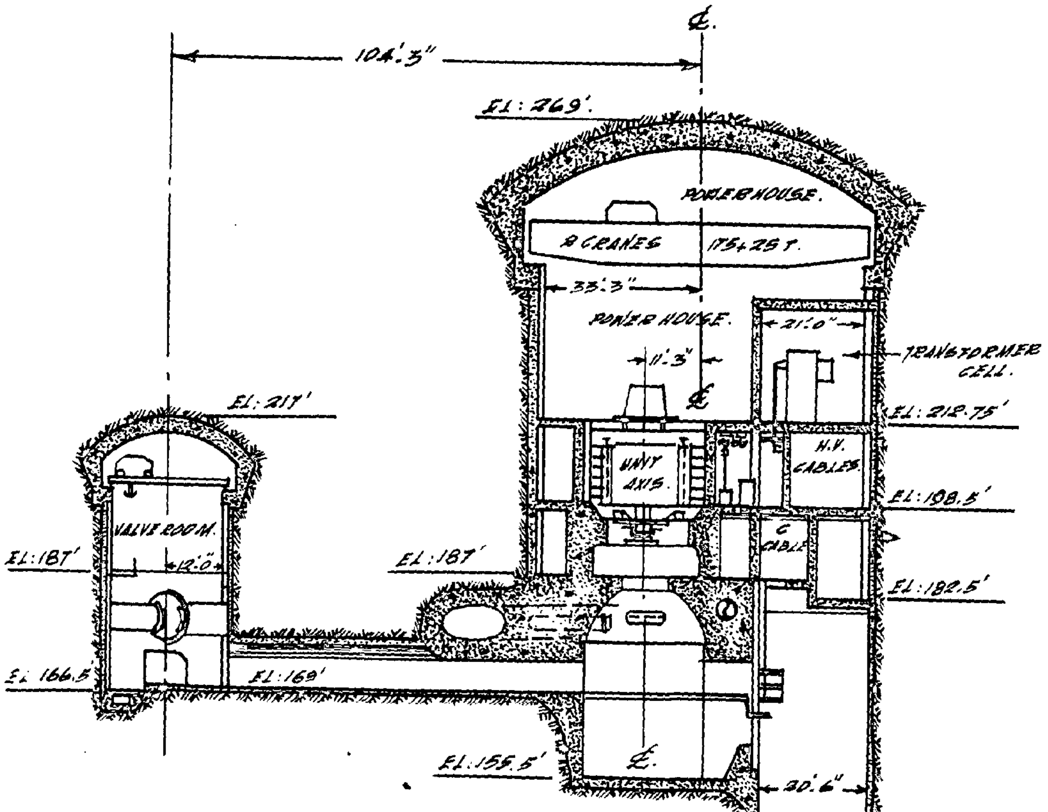


Fig: 8(j) ALTERNATIVE NO: II

8.4.5 TRANSFORMERS:

Since the switchyard could be located only at about 1500 Ft. from the station, location of the transformers close to the station is found to be more attractive both from point of view of capital cost and energy loss. Therefore, the transformers are located underground and it is proposed to use oil filled cables from the underground transformers to the outside switchyard. Because of shipping limitations on size and weight, single phase transformers 48 MVA rating are decided to be installed.

In order to reduce the length of low voltage bus bars with corresponding savings in equipment and energy loss it is preferable to locate the transformers as close as possible to the generators. In Alternative-I the transformers are in a separate cavern while in Alternative II the transformers are on the downstream side of the machine hall inside concrete cells which are designed for high gas pressures in case of transformer failure. The front part of the cell together with a section of the cover are removable to allow transformer displacement with the help of the station cranes. In Alternative II, however, the structural requirements to accommodate the transformers are appreciable due to the necessity of providing heavy floor members and the span of the cavern is increased. Besides this, with single phase transformers located inside the cavern, the spacing of the units is increased thereby increasing the overall length of the main cavern. Advantages claimed in the case of Alternative-I are (i) Handling of the transformers are simplified; (ii) Erection and construction problems are greatly simplified as the electrical equipment is the last to be installed in the power house. The segregation allows the work on the transformers and high voltage cables to proceed without conflict with the activities viz. (i) the disposal of hot transformer oil; (ii) the problem of fire hazard. These considerations led to the choice of Alternative-I.

8.4.6 SELECTION OF GENERATING MACHINERY:

The Idikki project as concerned will have an estimated firm output of 213,000 kw at 100% Load factor. The power station is proposed to be installed with enough plant capacity to operate at nearly 30% load factor in the ultimate stage in which case the installed capacity is to be between 750,000kw and 800,000kw.

Increasing the size of individual unit and its speed leads to reduction in the cost per kw of the installation. However, in pushing up the size, a stage is reached when the generator or the turbine sets the limit for the output of the unit. The bigger the generating unit, the more severe can be the disturbance to the power system, when there is an outage of the unit. In an interconnected system, the minimum size of the generating unit is usually limited to about 20% of the system capability. When Idikki Project is commissioned, it will together with other projects in operation will provide a system capability of about 760 MW at 70% Load factor. Thus a rating of about 150 MW can be adopted for the generator at Idikki station. It is proposed to instal six units each of

130 MW capacity. six pelton turbines, 185,000 H.P. rated capacity with six jets and running at 375 RPM are provided. A turbine with a speed of 428 RPM would have reduced the size of the unit and consequently the overall dimensions of the cavern. It is considered that the reduction in dimensions may induce interference of the jets and the falling back water in such a restricted space with corresponding losses of efficiency and for this reason units having a speed of 375 RPM are selected. The adoption of six jets for the turbine makes the vertical arrangement the obvious choice for the station as the maximum number of jets that can be accommodated in a horizontal arrangement is only Four. The vertical shaft multinozzle is selected over a horizontal shaft unit for a number of reasons, higher efficiency, fewer units, nozzles may be set closer to tailwater, higher speed and therefore smaller turbine and generator, effective energy dissipation into wheel pit walls, less hazard due to wheel or bucket failure and less water hammer hazard to penstock from a single faulty nozzle operation because each nozzle controls a small part of the flow. The vertical arrangement of the generating units with two guide bearings instead of the conventional three bearing type is proposed in order to reduce the height of the unit and consequently in a saving of about 8 Ft. of the power house height. Internally mounted needle servomotor for the jets in place of the usual externally situated servomotor is proposed on account of saving in space and better hydraulic conditions created for the jet. The dismantling of the wheels is

proposed to be done completely in the turbine pit without the use of the powerhouse crane. Economic studies indicated that two cranes in the station would be cheaper than providing a single crane of larger capacity.

The two layouts for the power station are prepared based on tentative dimensions of the equipment to be erected. These layouts are shown in Fig.8g, 8h, 8i, 8j. Economic studies indicated that alternative-I with three caverns is cheaper, as may be seen from the statement given below:

COMPARISON OF MAIN QUANTITIES.

ALTERNATIVES I & II

	ALTERNATIVE-I		ALTERNATIVE-II	
	Excavation 1000 Cu.Ft	Concrete 1000 Cu.ft.	Excavation 1000 Cu.Ft	Concrete 1000 Cu.Ft.
1) Valve Chamber	527	100	575	110
2) Machine Hall-Valve Chamber Gallery	5	1	5	1
3) Penstock Branches	28	17	28	17
4) Wheel Passages	28	6.5	28	6.5
5) Machine Hall	2269	561.5	3208	658.5
6) Bus Galleries	55	10	-	-
7) Transformer hall	428	55	-	-
8) Tailrace connection	78	13	-	-
9) Tailrace	324	27	-	-
Total:	3742	791	3844	793

In the station arrangement of Alternative-I, it was originally proposed to have the access tunnel and the erection bay in the power house at the level of the main floor or the exciter floor. Economic studies revealed that by lowering the access tunnel and erection bay to the level of the generator floor, there would be some reduction in the height of the powerhouse cavern and consequent economy in rock excavation to the extent of about 5000 cu.yds. In this arrangement the runner can be brought to the valve house for repairs through galleries provided.

Pending detailed geological data the dimensions of the underground cavities for Alternative-I are fixed only tentatively. The geological conditions and rock behaviour are well defined by the excavation of the main access tunnel and exploratory tunnels (fig. 8k). In an underground cavern, the behaviour of the roof arch and the side walls are most important and the rock behaviour influence the arch span and the longitudinal axis orientation of the cavern with respect to joint patterns. At the site, three patterns of joints are seen; one is parallel to the foliation or bedding planes and constitutes the major structural weakness of the rock. In the two other patterns the joints appear to be rare and with wide spacing, sometimes more than 100 ft. apart. Based on a detailed analysis, the orientation of the cavern is changed by 10° so that the longitudinal axis is at right angles to foliations and bedding planes. It was also established that the span of the cavern can be increased upto as much as 90Ft. as the rock conditions were excellent. This made to think a single cavern layout instead of the three cavern layout as per Alternative-I.

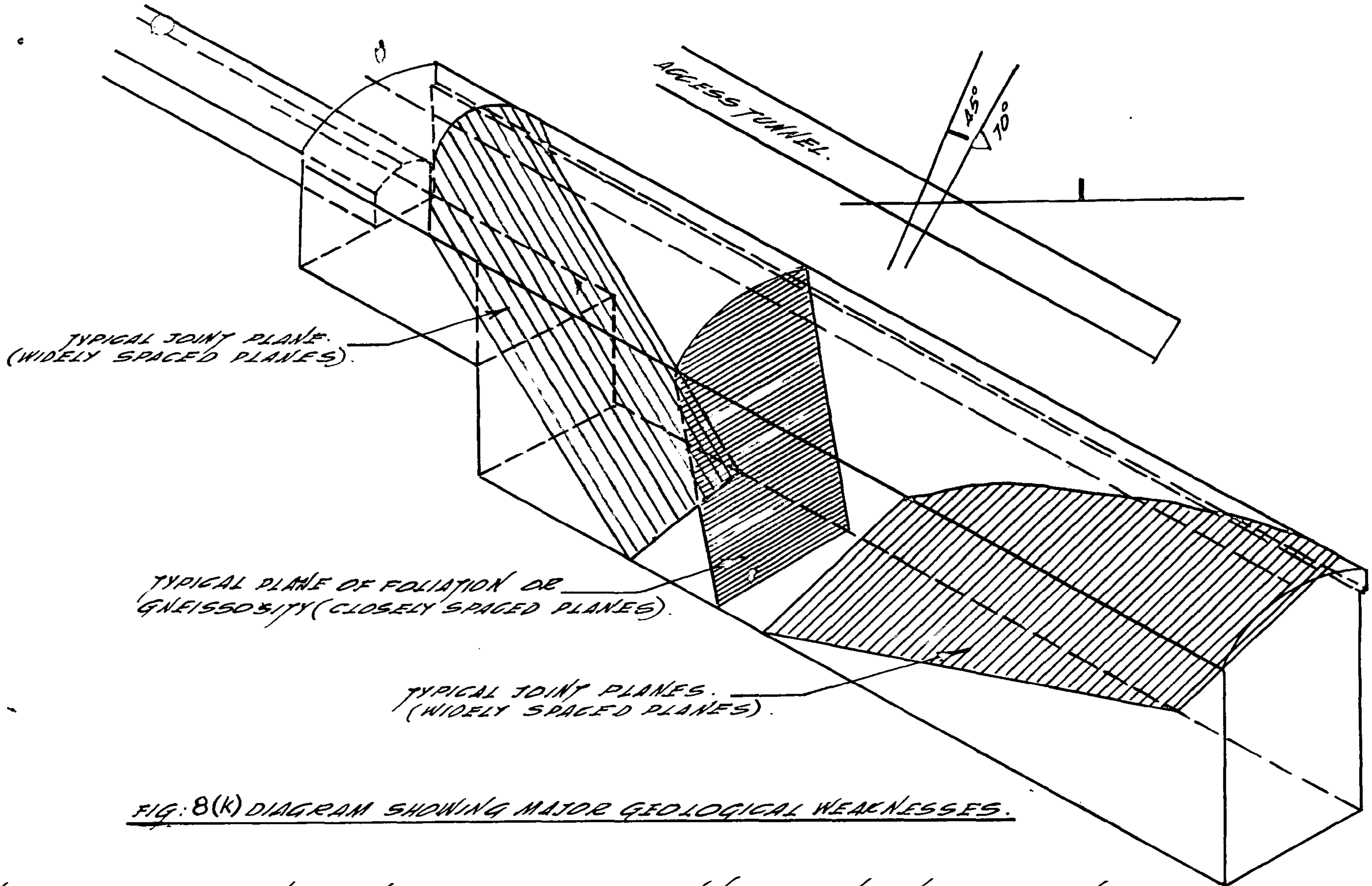


FIG: 8(K) DIAGRAM SHOWING MAJOR GEOLOGICAL WEAKNESSES.

NOTE:-
THE ROCK CONSISTS OF COMPETENT GRANITIC GNEISSES, SHOWING ALTERNATING DARK BANDS OR LAYERS RICH IN MICA AND LIGHT BANDS RICH IN FELDSPAR AND QUARTZ. THE IMPORTANT STRUCTURAL WEAKNESSES ARE THE CLOSELY SPACED FOLIATION PLANES. (GNEISSOSITY) AND OF LESSER IMPORTANCE ARE THE WIDELY SPACED JOINT PLANES.

The generating machinery details were malso available by then and therefore a detailed study was made. It was found that with proper allocation of space, the permanent equipments such as turbines, valves, generators and transformers can be accommoda-ted in a single cavern 62'6" wide. (Alternative III - Fig.81). In this arrangement the turbine valves (64" dia.) are located as close as possible to the turbines within the cavern thus eliminating separate valve chamber with its separate crane and the corresponding access galleries. To take care of the psychological objection of placing the valves in the main cavern, one water passage connecting each valve area directly to the turbine pit is provided. If one valve or one penstock should burst, the six water passages will take the full flow of such an unlikely occurrence without the water level reach-ing the electrical floor.

The transformers are located in independent cubicles in the main floor. These can be pulled out on rails and lifted and transported by the main crane. Each transformer has its own fire protection. The tailrace passes through the turbine pits. It is "W" type in plan to provide means for removing the turbine wheel.

A comparative statement giving the quantities (rock excavation and concrete) involved for Alternative III and Alternative-I is given below. Alternative IV in figure 8a provides a minor change in the tailrace arrangement and it is proved to be costlier than Alternative-III.

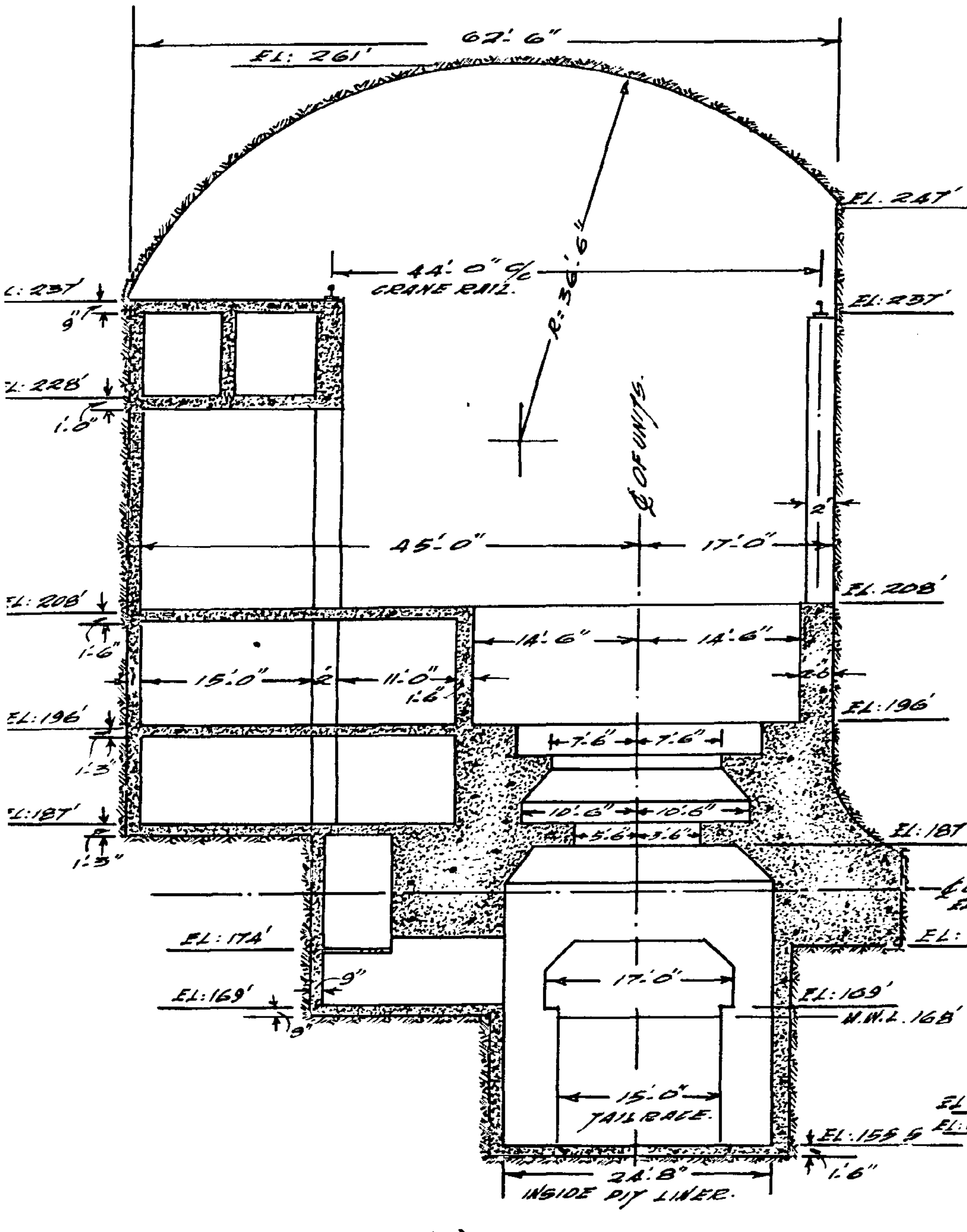


Fig: 8(l) ALTERNATIVE NO: III.

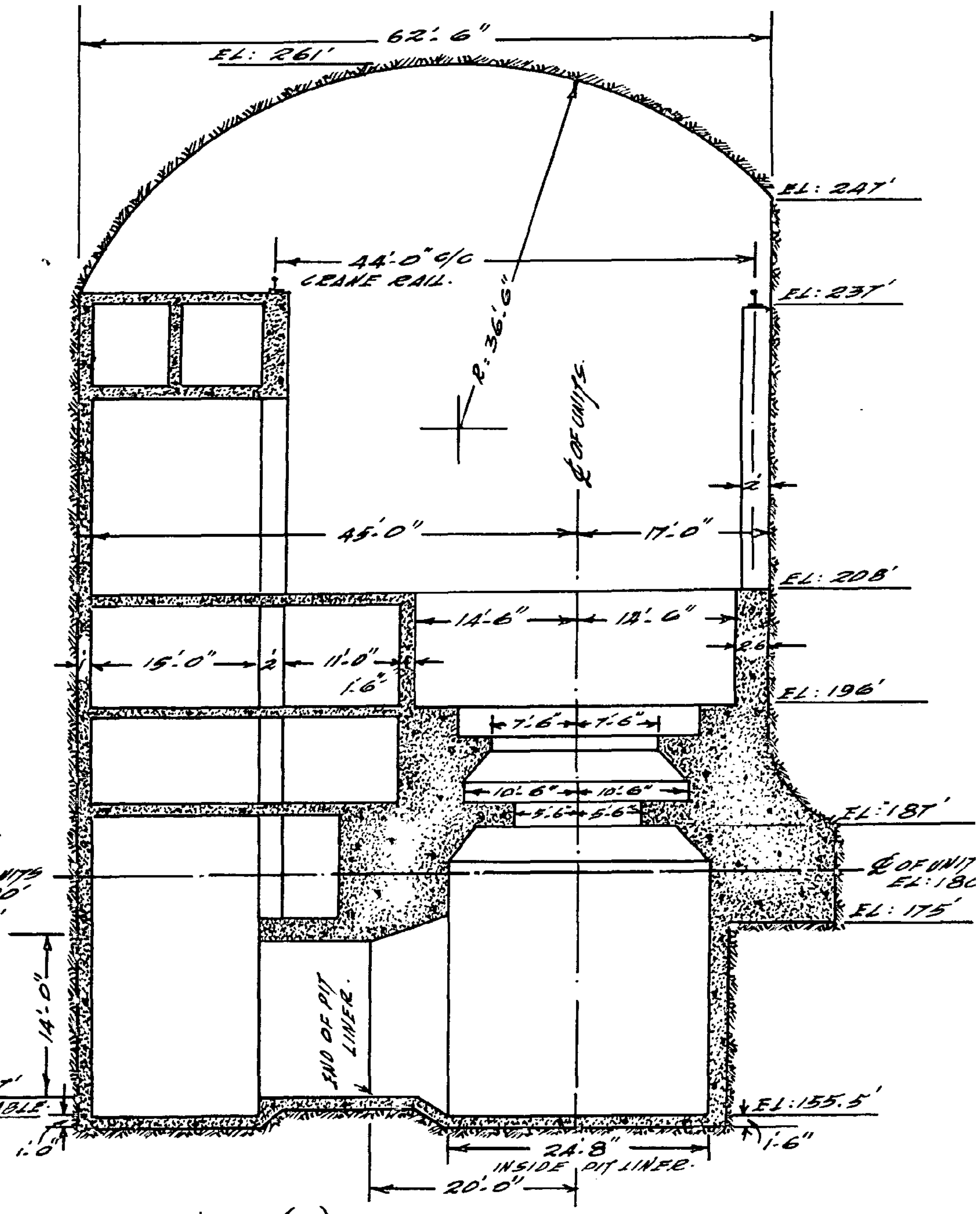


Fig 8(m) ALTERNATIVE NO: IV.

COMPARATIVE STATEMENT

ITEM	ALTERNATIVE--I Quantity in 1000Cft.ft. ROCK	CONCRETE	ALTERNATIVE-III Quantity in 1000 Cu.ft. ROCK	CONCRETE
1) Access to Penstocks, Valve chamber and Transformer hall.	206.8	-	-	-
2) Penstocks from Valve Chamber.	29.7	16.52	-	-
3) Tailrace tunnel, Manifold branches and collector	390.0	95.00	-	-
4) Manifold and Penstocks from EL.183.0	350.0	125.00	113.6	56.1
5) Cavern/Caverns	3950.0	910.00	2337.0	382.5
Total:	4926.5	1146.52	2450.6	438.6

Difference in rock excavation: say 25 lakhs Cft.

Difference in concrete: say 7.1 lakhs Cft.

The reduction in volume of excavation and concrete results in savings of 55% in cost.

Alternative-III results in a saving of about 55% in cost which is due to the fact that all the equipment is combined in one cavern. Paradoxically the useful volumes at the electrical and mechanical floors have been increased. The additional advantages of Alternative-III are;

i) Reduction of time required for the excavation, concreting and construction;

ii) Possible reduction of quantity of construction equipment.

In Alternative-III, the power station roof concrete is eliminated. From rock mechanic investigations and geological investigations it was found that the rock in the arch portion can support by itself and therefore only rock bolting and guniting is provided for. The above considerations led to the choice of Alternative III. Other features of the power station layout are discussed below:

8.4.7 SWITCHYARD:

The location of switchyard presented a major problem as suitable site (700' x 100') was not available within a reasonable distance from the power station. The pressure head on the 220 kv oil filled cables to be taken to the switchyard was desired to be limited to 500Ft. In order to satisfy this condition, a suitable site could be located only on the slope of the steep hill in which the power station is located. The site on the slope was proposed to be built up in part cutting and part filling by the muck removed from the power station & cable tunnel.

Several alternatives were considered for taking cables from the power station to the switchyard. Initially two cable tunnels (10' x 10') having a slope of 1:3 were suggested. These tunnels were also thought to be used for ventilation. A vertical shaft 15ft. diameter from the power station connected by a horizontal tunnel from the switchyard was also considered (Fig.8b). This alternative proved to be economical than the sloping tunnels. The shaft extends upto the surface with reduction in diameter (10') for ventilation purposes. The shaft will be used for emergency exit also.

7.4.8 TAILRACE TUNNEL:

The tailrace tunnel has to accommodate a maximum flow of 4950 cusecs. A combination of tunnel and channel had to be considered to lead the tailwaters to Nachar stream. Improvements to Nachar was also considered necessary to accommodate the large discharge. Several alternatives were considered to get an economical alignment consisting of a tunnel portion and a channel portion and a section of Nachar for which improvements have to be made. The selected alternative involve construction of: (i) 4015ft. of tunnel, (ii) 840Ft. of channel and (iii) Improvements of Nachar stream for a length of about 1½ miles.

As regards section of the tunnel, several alternatives were considered. The tunnel is to be designed for free flow conditions and therefore a circular section was out of question. Economic studies indicated that triangular section with curved roof would be the cheapest. However, on practical considerations a rectangular section 26' x 16' surmounted by an arch having a rise of 8' is adopted. The flow area in the tunnel is proposed to be lined with concrete while only rock bolting and guniting are proposed for the roof.

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CHAPTER - IX

CONCLUSIONS

Sufficient thought has been bestowed in finalising the layout of Idikki underground stations as given in Alternative - III. According to the author's opinion, however, a few refinements in the layout are possible and these will further reduce the cost of the layout by about 20%.

9.1 THE POWER WATERWAY:

The power waterway (Power Tunnel, Surge Tank and tailrace tunnel) selected for Alternative-III is the shortest possible alignment and geology permits construction of the various structures in the alignment. Therefore no modifications in the alignment is considered necessary. A minor modification in the design of the surge tank is suggested according to which the vertical shaft of the surge tank is connected by a small tunnel with restriction at the entrance. The small section of the tunnel will be at right angles to the power tunnel. With this arrangement, the construction of the surge tank is simplified.

9.2 GENERATING MACHINERY:

According to Alternative-III, six units are proposed to be installed in the power station. For an output of 130MW at 0.9 power factor at the generator terminals, a turbine of 185,000 HP having six jets and a runner speed of 375 RPM has been selected. According to the author's opinion a higher speed can be adopted so that the cost of the machinery and the cost of the power station excavation can be reduced.

The weight and approximately the price of a generatwr varies in inverse proportion to the square root of the speed. Present development in Pelton wheel design would permit a specific speed as high as 5.2 per jet. The specific speed per jet for a 6 nozzle single runner Pelton turbine of 185,000 HP and 200,000 HP under a net head of 2100 FT. for synchronous speeds of 300, 333, 375 and 428 are given below:

TURBINE	Synchronous Speed	Approximate specific speed/jet
	300	3.72
	333	4.14
185,000 H.P.	375	4.65
	428	5.20
200,000 H.P.	300	3.77
	333	4.2
	375	4.74
	428	5.4

It may thus be seen that for a turbine with a rating of 185,000 H.P. a synchronous speed as high as 428 RPM can be adopted. The impact per minute on the runner for this speed works out to only 2568 which is a reasonable value for a gross head of about 2200 Ft. Advancement of runner construction and materials used for runner enable pelton wheels to be manufactured to withstand impact of 3600/minute. To run-a-way speed ratio of a Pelton wheel being about 1.8 to 1.9, the maximum peripheral speed of the rotor of 144 MVA (130 MW at 0.9 PF) generator of normal construction may, however, reach the limit of nearly 3000/Minute for materials now generally used for rotor rims and poles. A generator 144 MVA capacity having a speed of 428 RPM for coupling to a Pelton turbine can be constructed and therefore there is decided economic advantage in going in for a unit of this size and speed. Increase in speed reduces the size of the unit and therefore the space requirement in the power station is reduced. If units of 428 RPM are chosen, the overall length can be reduced by about 18 Ft. while the width of the powerstation cavity can be reduced by 3 Ft. Such a layout will result in a saving of about 10% in cost on account of reduction in the volume of excavation and cost of machinery.

A normal voltage rating of 11 KV for the generators is adopted for Alternative III. In view of the very large size for the Idikki generator, an increase in the voltage rating to 14 to 15 KV may lead to more balanced and reliable design for the windings and reduction in cost of the heavy current connection between the generator and the step up transformers.

9.3 POWER STATION:

A basic design criterion was to put all facilities, valves, units, control room, transformers etc., in one excavation. It was proved that this would give lowest cost plant and the safest excavation. The height, crane rail to the top of the stater, for installation of rotor has been kept minimum that required. Factors that made this dimension minimum are (i) use of two cranes with equalizing beam; (ii) lifting thrust ring on shaft to be as far down on shaft as possible; (iii) flange between generator shaft and turbine shaft to be as close to rotor as possible. An excavation width 62' 6" was provided after earmarking space for all facilities. Crane access to turbine valves is not considered a requirement in determining power house width, since it seldom if ever, is necessary after installation. For the units of 375 RPM the space provided for the various features within the cavity is just the minimum.

The underground power house cavity is approximately 1200 Ft. vertically below the surface. From stress measurements it is generally found that the horizontal stress in the power house area is three or four times the vertical stress. It may be due to the proximity of deeply cut valleys. In the power house area, excellent hard rock is available as revealed by the adit driven from the access tunnel. Excavation of the cavity will result rock masses to decompress and move into newly created openings. Decompression in hard rock material results in spalling of the material into slabs of about an inch in thickness extending back into the walls for several feet. In its most extreme forms decompression occurs as bumbing ground or as rock bursts. According to Rabcewicz (60) rigid rock masses containing few structural discontinuities are more likely to exhibit serious decompression than well jointed rock masses, where the very existence of the joints to some extent may indicate that decompression has already occurred and additionally these joint zones provide space into which decompressing material may move. According to the author's opinion the effect of these stresses in the power house site will be considerable especially since cavity has vertical walls and therefore, outward curved walls are suggested.

It is better that the shape of the cavity is determined after conducting model studies.

In Alternative-III the side walls are covered with concrete. It is considered better that concrete walls are constructed 3" away from the rock face so that air drying of the rock walls is possible and the characteristics of rock can be improved. Further, any leakage from the walls can be properly dealt.

9.4 ACCESS TO THE POWER STATION:

Alternative III provides a 24 ft. x 20 ft. tunnel for permanent access to the underground power station. According to the author's opinion this tunnel could have been completely eliminated from the layout. This is possible by making use of the 26 Ft. wide tailrace tunnel for permanent heavy maintenance access and all other access requirements through a shaft which will be 22 ft. diameter accommodating a lift, a stairway, cables and ventilation ducts. The shaft as in Alternative III is 15' dia. and this can be increased to 22 Ft. dia. to accommodate all the facilities. The multiple use of the tailrace tunnel requires that the plant be shut down when loads greater than the shaft elevator capacity must be handled. When the access tunnel is completely eliminated, emergency exit provision is to be made. This can suitably be provided by constructing a walkway located above the maximum water level in the tailrace tunnel. For accommodating the walkway also, a Gothic arch roof is suggested to be constructed for the tailrace tunnel. The capital cost saving in using the tailrace tunnel as the access tunnel, is considered to greatly outweigh the possible but improbable impairment of operation.

9.5 TAILWATER DISPOSAL:

The tailwaters of the power station are led through a tunnel 4015 ft. long and channel 840 Ft. to the Nachar stream. Nachar is a small stream having a maximum flood discharge of about 6000 cusecs. The maximum discharge from the power station is about 5000 cusecs. Thus the Nachar stream below the channel exit has to be suitably widened and trained to accomodate a maximum flow of about 11000 cusecs. The elevation at the exit of the channel (when constructed) will be about + 144.00 whereas the nominal bed level of Nachar stream at the confluence point is + 176.00. This necessitates deepening of the stream by about 32 Ft. at the confluence point.

Since the stream has only a very mild slope, deepening and training works will have to be done for about 8000 Ft. from the confluence of the tailrace channel for conveying a maximum flow of 11000 cusecs. For the improvements of the stream, a large area has to be acquired for which very heavy compensation will have to be paid. Altogether the above proposal appears to be very costly. Besides, the problems that will be encountered in the acquisition of cultivated land are numerous. Perhaps commissioning of the power station will also be affected due to the probable delay in the completion of improvement works in Nachar stream.

Based on the studies conducted by the author, the following proposal appears to be cheaper and therefore it deserves serious consideration.

It is suggested that the capacity of the river channel need be only 6000 cusecs. With this capacity the flodd flow together with the maximum discharge from the station should not be allowed to pass through the channel. This is possible by shutting down the power station for a couple of hours during the peak flood period. Such a contingency of a complete shut down of power station may arise perhaps only once in 100 years and it may be permitted. The savings attained by constructing a channel of lesser capacity will more than offset the cost on account of power loss.

9.6 CONCLUSION:

The cost of alternative III can be reduced by about 20% if the modifications suggested by the author are incorporated in the Idikki underground powerhouse layout.

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(iii) Geological reports on Idikki H.E.Project by Dr.sam.

(iv) Geological reports on Idikki H.E.Project by Mr.Charlambakis.

(v) Notes on layout of Idikki Underground Power station submitted by the author to the Chief Engineer, Civil, K.S.E.Board.

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REPORT ON THE DISSERTATION SUBMITTED BY SRI M. GOPALANUNNI
FOR THE AWARD OF MASTER OF ENGINEERING DEGREE IN (W.R.D.)

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The dissertation on "Layout of underground Powerstations with reference to Idikki Station " given by Shri M. Gopalanunni is an outstanding performance. He has made a very wide study of the subject and has exhaustively covered all the aspects which determine the factors for layout of underground powerstations. His dissertation will be of great use to any body connected with underground works. M. Gopalanunni has a clear conception of what he has written and has a dynamic thinking.

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