RIVER DIVERSION PLANNING FOR DAM CONSTRUCTION WITH SPECIAL REFERENCE TO TEHRI DAM

A DISSERTATION

Submitted in partial fulfilment of the requirements for the award of the degree

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in

WATER RESOURCES DEVELOPMENT

By

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CANDIDATES' DECLARATION

I hereby declare that the work, which is being presented in this dissertation titled **"RIVER DIVERSION PLANNING FOR DAM CONSTRUCTION WITH SPECIAL REFERENCE TO TEHRI DAM"** in partial fulfillment of the requirement for the award of the Degree of **MASTER OF TECHNOLOGY** in **WATER RESOURCES DEVELOPMENT (CIVIL)** and submitted in the department of Water Resources Development Training Center, Indian Institute of Technology, Roorkee is an authentic record of my own work carried out during a period of July 2002 to Feb. 2003 under the supervision of **Dr. B. N. Asthana,** Emeritus Fellow, WRDTC, and **Dr. R. P. Singh**, Professor, WRDTC, IIT, Roorkee.

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SYNOPSIS

The design for a dam, which is to be constructed across a stream channel must consider diversion of the stream flow around or through the dam site during the construction period. In the case of important schemes the river will usually have a permanent flow and it will thus be necessary to temporarily dry out the zone where the works are to be carried out. The extent of the diversion problem will vary with the size and flood potential of the river. A diversion problem exists to some extent at all sites except those located off stream. The diversion scheme ranks in importance with other project features and is often a major cost item and therefore, requires intelligent planning.

Selection of the most appropriate scheme for handling the flow of the stream during construction is important to the economy of the dam. It has to be synchronized with the construction schedule of the dam and should be most economical for the given site conditions. The principal requirements of any diversion plan are to provide adequate facilities for handling the normal river flow and to provide additional capacity for passing any anticipated floods without causing damage to the structure during construction. The proper diversion plan will minimize serious potential flood damage to the work in progress at a minimum of expense. The factors that need consideration in determining the best diversion scheme are characteristics of stream flow, the type of the dam, overall economy, the topography & geology in the vicinity and the construction schedule.

A literature study regarding the importance, general requirements and factors to be considered for the river diversion has been carried out from relevant textbooks. The different types of river diversion arrangements adopted for various Concrete dams and Earth/Earth & Rock fill dams in India and abroad are studied with referring to papers published in ICOLD, CBIP, ASCE, Water Power and Dam construction and other relevant journals. Important aspects for both types of dam are drawn from this study. The typical river diversion arrangement for Tehri Earth & Rockfill Dam, which is under construction, is critically examined.

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INTRODUCTION

1.1. Background of Study

Within the complexities of large dam construction, one of the most difficult works, and most important in its effect upon the whole, is that of river diversion. There are, two main reasons for this, the study of the diversion system includes a number of factors, which are difficult to estimate and there is usually a limited time period for completion of these works. In order to carry out the excavation and foundation works, of the dam with sufficient guarantee of quality, one or more water tight areas must be created. This can be accomplished by diverting the river waters from their natural course.

The best method of stream diversion to use at any dam site is dependent on the stream flow characteristics, the water velocities that will occur during diversion, the type of foundation material at the dam site, the type of dam that is to be built, the topography at the dam site, the methods and equipment with which the dam material will be placed, the time required for construction of the dam and the cost that would result if overtopping of the diversion facilities occurred. The number of stages or phases of the diversion operation during construction will be governed by the length of time required for construction, type of valley, diversion flow capacity, maximum expected stream flow and layout of structures.

Since it is part of the critical path of a dams construction schedule it controls the time required for construction. The diversion scheme ranks in importance with other project features and is often a major cost item and therefore, requires intelligent planning. Selection of the diversion method is therefore one of the first tasks that should be undertaken in planning dam construction.

The diversion works must form part of the overall project design in that

- They must be based on the same hydrological, topographic and geological features of the site as the main dam.
- They may be partially governed by a requirement for early commissioning of the permanent works under partial heads.
- They may effect the main dam position, type and/or appurtenant works.

- They involve major construction problems that may have considerable impact on overall construction time and cost.
- They may have to take in to account environment factors.
- They may have to be eventually incorporated in the permanent works.

In view of importance of river diversion planning for dam construction, this topic is chosen for this study.

1.2. Objective and Scope of Study

The objectives and scope of this study are:

i. To discuss the importance of diverting the river flow during dam construction.

ii. To summarize the general requirements and factors be considered for the selection of best diversion scheme.

iii. To make the important inferences from the experience of diversion arrangements adopted for various existing Concrete and Earth & Rock fill dams in India and abroad.

iv. To undertake a detailed study of diversion works provided for Tehri Dam, which is under construction.

1.3. Methodology of Study

The methods of river diversion, general requirements and factors to be considered for the selection of a diversion scheme are studied from standard books on dams and from papers presented in ICOLD, CBIP, ASCE and Water Power & Dam construction.

The experiences of diversion arrangements selected for existing Concrete Dams and Earth / Earth & Rock fill Dams in India and abroad are reviewed from the above literatures and then the important inferences are made.

The diversion scheme for Tehri Earth & Rock fill dam, which is under construction has been studied in detail.

1.4. Organization of Study

The study is presented in six chapters as out lined below.

CHAPTER 1: Introduction giving the brief background of study and the organization of report.

CHAPTER 2: Illustrates the general requirements and factors to be considered for the selection of a best diversion scheme.

CHAPTER 3: Deals with the review of diversion arrangement adopted for some concrete dams and the inferences drawn.

CHAPTER 4: Deals with the review of diversion arrangement adopted for some Earth/Earth & Rockfill dams and the inferences drawn.

CHAPTER 5: Case study of diversion arrangement adopted for Tehri Earth & Rockfill dam.

CHAPTER 6: Conclusions and suggestions for further study.

GENERAL REQUIREMENTS AND FACTORS TO BE CONSIDERED FOR THE SELECTION OF A BEST DIVERSION SCHEME

2.1 Need for River Diversion

In order to carry out the foundation excavation and treatment as well as dam construction, it is almost always necessary to confine and dewater the river reach in which the dam is located. During the construction period of the dam the river flow has to be diverted away for the construction area. River diversion is the most important and critical activity in the construction programme of the dam. Judicious planning is necessary taking in to consideration the sequence of construction, the type of dam environmental factor and the downstream water requirements during construction.

Any miscalculation by the designer in predictions concerning the river diversion during the construction period may have serious repercussions on the cost and may result in delay of the works. There are two reasons for this; the study of the diversion system includes a number of factors, which are difficult to estimate and there is usually a limited time period for completion of these works. The river diversion at the dam site is always a singular case and it is imperative to be well briefed on the problem.

2.2 General Requirements

Diversion of the river during the construction period is necessary, whenever a dam is built across a river. The extent of the diversion problem varies with the size and flood potential of the stream. At some dam sites the diversion may be costly and time consuming and may affect scheduling of construction activities, as in the case of Earth and Rock fill dams, while at other sites no great difficulties are encountered in this regard. However, a diversion problem exists at all sites except those located off stream.

Selection of the most appropriate scheme for handling the flow of the river during construction is important to obtain economy in the cost of the dam. The scheme selected ordinarily will represent a compromise between the cost of the diversion facilities and the amount of risk involved. The objective is to select the optimum scheme considering practicality cost, pollution control and the risks involved.

The principal requirements of any diversion plan are to provide adequate facilities for handling the normal river flow and to provide additional capacity for passing any anticipated flood without damage to the structure during construction.

The first requirement must be satisfied by means of diversion arrangement chosen. The second requirement may be satisfied by diversion capacity sufficient to take care of floods or by suitable construction schedule so that appropriate facilities of structure itself will be completed in time to accommodate the expected high water.

The diversion works should be such that they may be incorporated into the overall construction program with a minimum delay.

2.2 (b) Data to be collected

The following data are required to enable planning and design of works for temporary river diversion after preliminary selection of site of diversion works has been made (IS: 9461-1980)

(i) **Topographical Survey** - Includes the preparation of :

- An index plan showing the main work, the entire scheme of river diversion and other important works affected by proposed scheme.
- A contour plan of the area around the proposed site of the main work extending well beyond the proposed site of the river diversion work with contour intervals of 0.5 to 1.0m. Salient features like firm banks, deep channels, large shoals and islands; deep pools, important landmarks etc. should be shown on the plan.
- Cross section of the river shall be observed at intervals of upto 200 m and should be extended in a length at least 600 m on either side of the work area and diversion scheme. The cross section should be prepared upto 2.5 m above design flood level and highest observed flood level if available at site, should be marked.
- L-section of the river for a distance 1 km upstream to 600 m downstream beyond the area covered by the entire diversion works.
- Erosion characteristics of the river should be observed and marked on the plan and cross-sections.

(ii) Hydrological data

Following hydrological data shall be required for planning & designing of river diversion works.

- Daily rainfall recorded at different rainfall stations in and around the catchment area. Pattern of rainfall in the area in previous years with duration of dry and wet spells should also be studied to help in forming an idea about the dry period available for construction.
- Flood hydrographs for isolated rainstorms for working out unit hydrographs.
- Peak flow data for monsoon and non-monsoon periods.
- Information regarding high flood level.
- Gauge-discharge curve at least one from upstream and one from downstream of the main work.

(iii) Sediment transport studies

The data regarding quality and quantity of bed and suspended sediment and boulders carried by the river, especially during flood season, should be collected.

(iv) Timber Survey

A detailed timber survey shall be carried out to collect information about the size and quantities of timber sleepers or wooden logs floating down the river in various months of the year at the site or work. For such floating timber, provision shall be done in diversion scheme to collect either at upstream of diversion works or to pass down through the diversion works.

(v) Surface & subsurface investigation

These investigations help in the layout of diversion scheme. Following investigations should be carried out.

- Nature and characteristics of over burden and loose deposits.
- Geological investigations through drill holes & drifts.
- In case of permanent diversion tunnels, physical & mechanical strength properties of rock by plate bearing tests, flat jack tests etc.
- Permeability of the ground soil.
- Water table.

2.3 Factors to be Considered

In considering diversion schemes for the construction of a dam, the main aims should be to provide this temporary structure at minimum of cost and risk. The problem of by-passing the stream flow during construction of the dam is highly important and a difficult undertaking. Each dam site has specific requirements. A few considerations involved in the design of diversion system are:

2.3.1 Hydrological Consideration

i. *Hydrological aspects on river diversion during construction of dam*

The diversion and closure of large dams present very challenging and difficult problems, especially on large streams. It is important that these problems be given careful consideration during early planning stage since in many cases these are the governing features in the selection of the type of dam, its composition and type and size of outlets.

During the planning stage all available hydrological data should be studied particularly hydrographs, which indicate the most favorable period for effecting final closure and diversion through the outlet works.

ii. *Characteristics of stream flow*

Stream flow records provide the most reliable information regarding stream flow characteristics, and should be consulted whenever available. Depending upon the size of the drainage area and its geographical location, floods on a stream may be the result of snow melt, seasonal rains or cloud bursts. Each of these types of run off have their peak flows and their periods of low flow at different times of year. Most streams have large variations between high and low flows. Low flows can be used to great advantage in the construction of cofferdams particularly in making closures and turning the stream in to diversion channels.

For determining the characteristics of stream flow graphs showing the volume of run off that will pass by the dam site and anticipated flood flows with varying frequencies should be plotted. This graph should also be prepared showing calendar months when flows could occur for floods rated as 5,10,100 years and minimum, maximum and average daily flow.

If stream flow records are not available or have been kept only for a short period of time, the design of the diversion facilities must be based on the area of watershed and the amount of anticipated precipitation, correlated with the stream flow records that are available. For large flows and high velocities the diversion design calculations should be supplemented by hydraulic model studies. Specific points that may be decided by the model studies are [Parker, 1971].

- The water velocities that will occur along the cofferdam sides or during cofferdam closure.
- The size of rocks that will be required to prevent cofferdam erosion.

iii. Diversion flood

The magnitude of the diversion flood determines the capacity of the diversion arrangements and the height of the cofferdams to create adequate head for forcing the discharge through the diversion tunnels or channels. The combination of the cofferdam height and the tunnel size also has to be chosen from considerations of maximum economy and the limiting capacity of the organization to build the cofferdam in one working season of low river discharges. The temporary storage provided by the cofferdam, if appreciable may be considered for moderating the design flood.

Obviously it will not be economical and viable to design the diversion works for the same magnitude flood as the spillway. It is therefore, a common practice to consider floods of higher probability of occurrence for design of diversion works. For small dams, which will be, constructed in a single season, only the floods, which may occur for that season, need be considered. For medium dams involving at the most two construction seasons it should be sufficiently conservative to provide for a flood with a probability of occurrence of 20 percent. For larger dams involving more than 2 year construction season a design diversion flood with a probability of occurrence of any where between 20 and 4 percent may be established depending on the loss risk and the completion time for the dam.

The selection of design flood for diversion mainly depends on the amount of risk, which one is prepared to take. A situation wherein overtopping may have disastrous results, calls for more conservative design than one in which nominal damage would result. Thus for a high concrete dam which could pass substantial amount of flood discharge without appreciable damage or failures, considerably less diversion capacity would be required than for an embankment dam of equivalent height. Usual value of frequency of design flood adopted for concrete dams is 1 in 25 year and for embankment dams it ranges form 1in 100 to 1 in 1000 depending on the period of construction, which determines the probability of overtopping or the risk involved.

In selecting the return period, however, it is useful to remember that the probability of an event with a return period of T years occurring at least once in the course of these T years is approximately 0.64. The risk R of a flood of a return period of T years being exceeded at least once in L years over which the diversion structure operates is given by [Sharma, H.D., 1998].

$$R=1-(1-1/T)^L = 1-e^{-L/T}$$

Or approximately

$$R=\frac{L}{T+0.5L}$$

For example, if the diversion works are designed to operate over a 3-year construction period and the cofferdam is built to withstand the 10 year flood, the percent risk of overtopping or failure is

$$R=\frac{3}{10+0.5\times 3}=0.26 \text{ or } 26\%$$

The following considerations should be taken into account in selecting the diversion flood [USBR, 1976].

- The number of flood seasons which will be encountered for the completion of the work.
- The additional cost or damage to the work completed or still under construction if it is flooded.
- Increase in cost on account of delay in completion of work while the flood damage is being repaired.
- The safety of workmen and safety of the downstream inhabitants in case the failure of diversion works results in flooding.

2.3.2 Topographic and Geotechnical Conditions

Site topography and geology are also factors in the choice of diversion woks. They will not be the same in a narrow, steep sided valley as on a wide flat river. A narrow valley may be well suited for the classical two bank to bank cofferdams solution, where as a wide valley make it possible to isolate one part of the river bed by means of a cofferdam while the water flows over the unobstructed area. If the dam site is on a winding stream at the end of a narrow ridge, the ridge may permit construction of short diversion tunnels at low cost.

Abecasis, M., et.al. (ICOLD, 1973, Q41, R5) wrote that the geology of the site must also be considered in design and for that geological investigations are essential. The designer should have a detailed and accurate geological study of the site area in order to become well acquainted with the geological profile. Geotechnical conditions can influence.

- Diversion tunnels either by limiting their size or whether lining is required or not.
- The construction of the closure embankment.
- The zones used for diverting the river, when they are subject to erosion, making it necessary to protect them.
- The cofferdam foundations, determining their type or requiring special arrangements or grouting.

Exposed rock in the streambed as compared to heavy earth overburden makes the driving of sheet piles difficult but tunneling easier, thus influencing the method of diversion. And also, the on-site availability of suitable materials like timber, rock, clay etc may influence the selection of the optimum scheme.

2.3.3 Type of Dam, Position and Appurtenant Works

The choice of dam may depend on how the river is to be controlled during construction. For example, an earth dam is not feasible, if the diversion works can't prevent it being overtopped before completion.

It may sometimes be necessary to move the dam to a point where the valley is wider in order to facilitate construction of the diversion works, despite the extra volume of materials in the main dam. This is particularly important on large rivers, not only for low head developments but for high heads too, where the narrower parts of the valley do not always afford the best sites if the diversion works there would be difficult and costly to build.

The choice of the diversion scheme may also be related to other parts of the permanent works besides the dam although diversion works are by definition temporary, they may in some cases be designed to be wholly or partially incorporate in the permanent works as spillways, bottom outlets, power station head races, irrigation or other outlets.

2.3.4 Overall Construction Time and Cost

Apart from the cost of the diversion works themselves, the method adopted has an important impact on the overall construction programme and the cost of the permanent works.

If the river control scheme is such that construction of the permanent works proceeds in fits and starts, costs will rise because of the larger amount of constructional plant needed and the less efficient use of manpower. Delays in the construction of the diversion works may put back the project to a point where a whole year may be lost on seasonal rivers.

Therefore it is important to give due weightage to practical considerations such as the lead time necessary for mobilization on site, and deliveries of plant and materials, particularly for diverting the river in to the temporary passages.

2.3.5 Economical Consideration

The method of stream diversion must be one that will not endanger life or cause excessive property damage when the diversion facilities are subject to maximum flood flows. If the method chosen restricts the flow of water past the dam site, it must prevent overtopping. For economic reasons it must be the choicest method which may divert water past the dam site without delaying construction, with minimum probability of overtopping.

To determine whether diversion facilities should be designed for controlled overtopping or with sufficient capacity to prevent overtopping, the overtopping damage cost should be compared with the cost of providing increased diversion capacity. Based on this study the determination of whether the diversion facilities should be designed with sufficient capacity to handle flood flow with frequency of 1 in 5 year, 1 in 10 year, 1 in 20 year and so on, is simply a matter of economics. It is usually more economical to risk occasional overtopping than to provide sufficient diversion capacity to handle all flood flows.

2.3.6 Environmental Consideration

i. River traffic

Arrangements must be made to prevent disruption of river traffic during construction. The most common way of solving the problem is to build the permanent works in the river channel in stages, by providing the requisite cofferdams.

ii. Fish passage

The diversion works are often required to allow fish to pass. One option is to build either a special conduit with a fish pass but it is costly. A cheaper alternative is to provide a short fish ladder and a chamber at the downstream end providing access to main diversion tunnel or channel.

iii. Floating debris

Arrangements must sometimes be made to allow floating logs, ice and other debris to pass through the diversion works without jamming inside them & reducing their capacity.

iv. Reservoir area

Population resettlement, working mines and quarries and the like in the reservoir area may delay river diversion and impose an upper limit on the rate of rise of the headwaters.

v. Water quality downstream

The construction works can seriously affect river quality in both physical and biological terms. One must take this problem into account and provide for necessary steps in order to suppress or reduce the pollution to a degree acceptable to downstream users.

vi. Alarm system

Arrangements should be made at the beginning of construction to have an alarm system so that men and plant can be evacuated in good time in the event of a flood exceeding the capacity of the diversion works.

2.4 Diversion Methods

Generally there are two types of diversion methods: single stage and multi stage [ICOLD Bulletin 48a, 1986]. These are shown in figı. 2.4.1 and 2.4.2 respectively.

2.4.1 Single Stage Diversion

This type of diversion is used in narrow valleys. The sequence of diversion is as follows

i. Build a partial cofferdam, allowing construction of diversion tunnels, culverts, flumes and control works.
ii. Divert flow through these passages.
iii. Build a full size cofferdam.
iv. Build the permanent works.
v. Close diversion passages and start impounding.

The bye pass structures used are tunnels, culverts and flumes. Tunnels are obvious choice for narrow valleys in rocks, whereas culverts and flumes are used where the river channel is wider and flatter. Diverting the river into the temporary passages and ultimately closing them off to begin impounding are usually the critical activities in this approach, and careful study is necessary. One must be sure that the diversion passages are ready as soon as the temporary closure dyke is built, otherwise the river will be damned instead of being diverted.

2.4.2 Multi Stage Diversion

On wide rivers diversion can proceed in stages, which is the most common arrangement. In the multi-stage approach, the sequence is normally as below:

i. Build the cofferdam, which usually extends into the river channel and provides a dry area for construction of the permanent outlet works and part of the main dam.
ii. The outlet works and dam section are built in the dry area behind the cofferdam.
iii. The cofferdam may then be enlarged and extended farther outwards in to the river channel to increase the dry working area before the river is eventually diverted through the permanent outlet works.
iv. Part or all of the cofferdam is demolished so that the river can flow through the permanent outlet works.
v. The second-phase cofferdam is built.
vi. The remainder of the permanent works is built behind the second-phase cofferdam.
vii. River closure consists merely of stopping the flow through the outlet works.

The ends of the cofferdam in contact with the flowing water must be protected by heavy rock fill, gabions and plain or circular sheet piling. Flow velocities are usually kept to reasonable values; however, to avoid erosion problems, and the head differential between the upstream and downstream ends of the opening must never be more than 5m. It is important to provide enough working space behind the cofferdam to accommodate the constructional plant and access works.

2.5 Diversion Passages

i. Tunnels and culverts

a. Tunnels.

River diversion through tunnels is the commonest practice in the construction of high storage dams. Tunnels are usually confined to steep sloped rock abutments. They have the advantage of not interfering with foundation excavation and dam construction work. A tunnel is not in direct contact with dam embankment, and therefore it provides a much safer and more durable layout than can be achieved with an open channel structure. A minimum of foundation settlement, differential movement, and structural displacement will be experienced with a tunnel, which has been bored through competent abutment material. The tunnel should be sufficiently far from the dam that its excavation will not effect the soundness of the dam foundation. Tunnels must have an adequate cover of rock. The cover to the tunnel and its location will be determined by the nature of the rock, depth of weathering and overburden [Golze, A.R., 1977].

The size of the diversion tunnel will depend on the magnitude of the diversion flood the height of the upstream cofferdam (the higher the head, the smaller the tunnel size for a given discharge), and the size of the reservoir formed by the cofferdam if this is appreciable. An economic study of cofferdam height versus tunnel size may be required to establish the most economical relationship [USBR, 1976].

Except on smaller rivers, twin tunnels i.e. one in each bank are most usually employed for safety and convenience. Twin tunnels are easier to convert them to permanent use, since the work can proceed in one while the other is still being used for diversion in the low water season. A diversion tunnel should be of a length that is adequate to bypass the construction area. Construction of the tunnels and upstream portals is often on the critical path. Diversion tunnel capacity is usually not more than around 2000 m^3/s in good, hard rock without lining and 1000 m^3/s in moderate to poor rock with concrete lining. The advisability of lining the diversion tunnel will be influenced by the cost of a lined tunnel compared with that of a larger unlined tunnel of equal carrying capacity, the nature of the rock in the tunnel, as to whether it can stand unsupported and unprotected, during the passage of the diversion flows and the permeability of the material through which the tunnel is carried. Tunnel velocities can be of the order of 10 to 20m/s.

Tunnels may be designed for pressure or free surface flow. In both cases tunnels will be provided with entrance works a bell mouth entrance which conforms to or slightly encroaches upon free jet profile will give the best entrance shape upstream, to create favourable conditions to the arrival of the flow, and with exit works with or without stilling basins. In the latter case, they must not run more than 70% full for the design flood or 80% if the flood is of very short duration. With circular section, the unit loss of head corresponding to this depth is practically equal to that of flow under pressure (Singh, B. & Varshney, R.S. 1995). For a pressure tunnel, a circular cross sectional shape is the most efficient both hydraulically and structurally. For a free flow tunnel, a horseshoe shape or a flat bottom tunnel will provide better hydraulic flow but it is not as efficient as the circular shape for carrying external load (Golze., A. R., 1977).

Fig. 2.5.1 shows different shapes which are generally used for tunnel cross sections.

The levels required upstream for the free surface or pressure flow of the design flow rate will therefore not differ much, one or the other type of flow may occur depending on the altimetric position of the tunnels. Flows under pressure will usually imply lower altimetric position of the diversion tunnel.

b. Culverts.

Tunnels are used when the rock is good whereas concrete culverts in the dam foundation may be preferable, if the rock is poor and the valley is wide enough. The costs are however high. Work in the foundation excavations may be hampered and there may also be problems at the interface with the core of embankment.

Culverts are built in the dry behind cofferdams, and when they are ready, the river is diverted through them and they can be closed off in the same way as tunnels. Concrete culverts may also be used as permanent spillways, bottom outlets etc.

ii Channels

Diversion channels are commonly used in wide valleys where the high flow makes tunnels or culverts uneconomic (Fig. 2.5.2). These are mostly used for concrete dams. One of the largest diversion channels ever built was at Ukai Dam, India (Capacity: 49,500 m^3/s) and Itaipu, Brazil (Capacity: 35,000 m^3/s).

A natural channel or streambed may sometimes be used, but excavation is necessary in most cases. Special attention must be given to the possible need for lining all or part of the channel and to the risk of bank slides, if it is deep. Water tightness of channel walls and aprons is often achieved by using concrete but other materials like sheet piles, rock fill and masonry may also be used [Singh, B. & Varshney R.S. 1995].

iii. Conduits through dam

Diversion conduits at stream level are sometimes provided through a dam. These conduits may be constructed solely for the purpose of diversion or they may be conduits, which later will form part of the outlet works or power penstock systems as shown in Figı - 2.5.3. As with tunnels, some means of shutting off the flow at the end of the diversion period and a method of passing downstream water requirements during the filling of the reservoir must be incorporated into the design of the conduit.

Conduits are provided as outlet works in earth dams of low to medium heights to pass regulated releases of water in accordance with requirements. When located on earthen foundations the conduits must be properly and adequately designed to withstand likely differential settlements (Fig. 2.5.3). There is also the danger of seepage along the contact surface of the conduit with the foundation and the earth fill around it. This requires proper embedment into the foundation, compaction of the earth fill by special devices and provision of cutoff collars to lengthen the path of percolation.

iv. Flumes

In a wide canyon, an economical method of diversion may be the use of flume to carry the stream flow around the construction area. A flume may also be used to carry the stream flow over a low block and through the construction area. The flume should be designed to accommodate the design diversion flood, or a part of it, if the flume is used in conjunction with another method of diversion. The most economical scheme can be found by comparing costs of various cofferdam heights versus the corresponding flume capacity. Large flumes may be of steel or timber frame with a timber lining, and smaller flumes may be of timber or metal construction, pipe, etc.

The flume is usually constructed around one side or the other of the dam site or over a low block. The flume can then be moved to other areas as the work progresses and stage construction can be utilized.

v. Diversion through or over permanent works

In some cases, river floods may be so large that it is neither economic nor practical to provide diversion passages even for average floods, the only alternative being to have them pass over or through the dam. Smaller floods are handled by the permanent outlet works or temporary low-level outlets with the larger floods overtopping certain dam blocks purposely lagging behind the others.

Small diversion passages are of course necessary while building these low-level outlets. Large temporary openings can be left in concrete dams, especially arch dams, with special arrangements for closing them off permanently at the end of construction.

The predominant trend in diversion arrangements for concrete dams, which has also been extended to earth and rock fill dams consists of reducing the diversion works to a minimum by passing some of the flow over or through the dam. Allowing water to spill over earth and rock fill dams when they are raised above the downstream cofferdam is more difficult problem generally avoided by careful planning and rapid construction of the fill to provide enough capacity for flood impoundment. In some cases, however, fill dams have been purposely overtopped or overtopping has resulted because of delays in construction or unexpected large river floods.

2.6 River Closure for Diversion

Closures on the natural riverbed may encounter problems in rivers in which the flow remains significant even during the dry season. This closure is made at the beginning of the works when the dam is built in one stage and the river diverted into temporary tunnels or, sometimes, into a channel. In wider valleys where the dam is built in two stages, the closure is made at the end of the first stage and water is diverted into temporary or permanent openings left in the first-stage structures.

Although the direct cost of closure is generally a small percentage of the total cost of the dam, it has to be studied carefully as a failure or delay may cause a serious and costly delay to the whole scheme and because the design of the temporary diversion relates to the maximum water head chosen for the closure. It is consequently of interest to define safe and realistic conditions for closure and to estimate for it.

River closure for diversion can be achieved by one of the following methods:

- End dumping of rock fill of boulders (or point tipping or vertical closure) method when advancing the embankment above water from one bank or from both until the gap is closed.
- Frontal dumping (or horizontal tipping) method when materials are placed uniformly across the whole width of the gap in the river channel, generally requiring a special structure such as a bridge or cable crane for transport of materials.

For both end dumping and frontal dumping the maximum final water head for a single embankment seems to have nearly always been under about 5 m in the case of large flows. Closure of the river by a cofferdam is initiated usually in the low-water period. There exist several methods of affecting the closure of a river. The pioneering method involves filling the river-bed with a gravel-sand mass to a layer of 1.5-2.0 m (riprap) form either one or both shores to be covered with a 'blanket' of large stones, tetrahedrons specially fabricated of concrete, hedgehogs and strings of these interconnected with wire. The blanket is topped with material borrowed for the closing operation and delivered to the site by a barge, overhead tramways and dump barges.

The closure can be effected by the hydraulic method of filling the river-bed with sand, gravel and small size stones. The mass (blanket) creates a backwater, the water in part, passing in a tunnel and some percolates through and flows over the fill (Fig. 2.6.1). To preclude shifting of the riprap as well as formation of a pit in the river bed below, which may 'bury' the fill, head is limited to 1-1.5m for materials other than rock and to 2.5-5 m for rock foundation. Head created by the upstream cofferdam should be sufficient to ensure though diversion of all water through tunnels. The downstream cofferdam is now filled in tranquil waters. Then the area between the two cofferdam is de-watered and construction of the main works can be taken up. Immediately after the closure, the cofferdams will be raised to the designed elevations.

2.7 Cofferdams

2.7.1 General

A cofferdam is a temporary dam or barrier used to divert the stream or to enclose an area during construction. The diversion operation is accomplished through a group of auxiliary works, the most important of which are the cofferdams. Thus, the selection of a diversion system and the type of cofferdam to be employed therein plus design and execution there of, comprise one of the main factors in the successful construction of a dam.

Although basically the purpose of a cofferdam or a dam is the same, the conditions of construction and operation and the short life of cofferdams may explain a number of differences between the two types of structures.

Some conditions are often more severe for a cofferdam [ICOLD Bulletin 48a, 1986] :

- Construction in a shorter time even for high cofferdams and often at a fixed season when weather conditions may be unfavourable for certain solutions;
- Difficulty or impossibility of dewatering the foundations;
- Sometimes lack of space and difficulty in access;
- Construction or operation in rapid flow;
- Higher risk of or design for overtopping; if the life of a cofferdam is 1 to 2 years flood instead of about 10000 years for a permanent dam;

- Impossibility of controlling filling of the reservoir.

The design aspects of cofferdams are interlinked with the construction programme and optimum river diversion capacity made available. A careful study of the aspects of river diversion, construction planning and programme, would determine the optimum height of the cofferdam. The design of cofferdam is also governed by considerations of limitations of area required for dam construction, stability against overturning, overtopping due to high floods in the river, stability against sliding and dewatering of the foundations of the same.

Scope

More often, particularly for high dams built in narrow valleys. Diversion is accomplished through tunnels bored in the abutments. Cofferdams are constructed at the upstream and downstream ends of the works area. The height of the downstream cofferdam has to be higher than the high flood level at the site by a safe margin. The height of the upstream cofferdam provides the necessary head for flow through the tunnels, and also creates some storage for flood moderation-thus for a given diversion flood hydrograph, a higher cofferdam reduces the required discharging capacity of the tunnels.

The combination of cofferdam height and tunnel size has to be chosen from considerations of maximum economy, and the limiting capacity of the construction organization to build the cofferdams in one working season of low river discharges. An economic and engineering study of cofferdam height versus tunnel size may be done to establish the most economic combination for the maximum diversion discharge. It is also possible to plot a chart to see the tunnel-to-cofferdam cost alternatives and to select the most economical in the first approximation (Fig. 2.7.1). Thus the sequence of diversion will be first, to construct a tunnel of sufficient capacity to carry low season discharges with little heading up, next to build temporary, low cofferdams to divert the low season flow, and then during the same low water season to construct the main cofferdams inside the temporary coffer dams. The temporary cofferdams are built by dumping rock in the river bed, and then plugging the leakage by dumping gravel, sand and finally clay on the outer slopes.

This work must be completed in one construction season; the cofferdam should have an impervious zone properly bonded with impervious ledge, and would thus involve considerable excavation as well. To make use of the substantial fill placed in the upstream cofferdam, it is often incorporated in the main dam. If the lowest draw down level is lower than the top of the cofferdam, its impervious zone should be removed to the drawdown level after completion of the dam, to permit free drainage. If the entire dam section can be raised to required diversion level in one working season, the dam itself would serve the purpose of the cofferdams.

Construction of cofferdams under water can be achieved by placement of rockfill either by end dumping or by dumping from trestles. Prior to dumping only the abutments are stripped, usually, although pervious deposits are also stripped at times.

2.7.2 Factors Influencing Selection of Cofferdams

A wide variety of designs exist for cofferdams and especially for low head cofferdams as they need to be adapted to suit local materials and available equipment. Cofferdams are often designed more according to local experience and construction facilities, specially the availability of materials on the site, than to any theoretical approach to stability; the same problem may, for instance, be solved, depending on the country, by cribworks, sheetpiles cells or rockfill cofferdam with an upstream impervious layer of clay placed under water.

Following are the factors to be considered for selection of cofferdams.

i. Type of material
ii. Availability of material
iii. Local conditions
iv. Time of construction
v. Cost of construction
vi. Head of water to be retained

Following requirements should be taken into account in selecting the type and size of cofferdams and the shape of dam foundations [Singh, B. & Varshney, R.S.1995].

- Distance from a cofferdam to the foundation of a structure should be less than and, sometimes, in excess of 10m.
- Shape of dam foundation, in plan should allow for suitable placing of roads on the crest of cofferdams, access to tracks to the dam foundation and a good approach to haul roads.
- Shape of dam foundation should be suitable for locating cranes, other construction equipment, water lowering and overflow drainage installations.

2.7.3 Types of Cofferdams

(i) Earthfill cofferdams

It is not possible to adopt for cofferdams the solutions generally used for permanent dams, due to lack of time and often the impossibility of dewatering. It is advantageous to use materials involving little preparation and/ or relax specifications and to design structures which may accept more settlement and possible seepage thereby avoiding long and costly preparation work on most of the foundations. This is not however an absolute rule as some cofferdams are built in the dry (during the low flow season and with protection from small preliminary cofferdams) according to classical dam designs.

The simplest type of cofferdam is one constructed of earth. This type may be used only where low velocities will occur during or after construction. Sufficient space must also be distance, this type of cofferdam will be the cheapest. Velocities greater than those producing scour are permissible during construction provided that fill can be placed faster than it is carried away. Velocities after construction must be less than those producing scour. Scouring velocities are shown below: (Stubbs, F.W., 1959)

Material	Scouring velocity, m/s
Fine clay	0.0762
Silt	0.1524
Sand	0.3048
Coarse gravel	1.2192

Velocity that will be encountered in final closure must be computed, in they become critical final closure of the cofferdam must be made with large rocks rock necklaces gabions tetrahedrons or other precast concrete shapes.

In most cases, a fill with the minimum top width required for construction equipment (usually 6m) and with side slopes at the natural angle of response of the available material (approximately 1V: 1.5 H for dry sections and flatter for fills placed under water) will prove adequate. For very high earthfill or where severe damage would result from failure, coffer dams must be designed and constructed according to the usual practice for earthfill dams. Where coarse materials and used for the main body of the fill, it ma be necessary to use an impervious core or an upstream blanket of clay on the water face to reduce percolation through the fill. Earthfill –cofferdams must not be permitted to overtop. Earth fills are usually built by end dumping up to river level.

(ii) Rockfill cofferdams

Next in order of simplicity is the rockfill type of cofferdam. This type merits consideration where supply of rock is economically available as, for instance, from an excavation of spillway, power house, where disposal to some point or other is required in any case. The rockfill cofferdam can be constructed in more turbulent water, which would wash away the ordinary earthfill. However, the rockfill requires some form of a seal on the upstream. This can be done by throwing earth on the upstream with timber matting at the water line to protect the earth against wave wash.

Rockfills are usually built by end dumping, but they may be placed in layers where it is necessary to reduce velocities. Except for high structures, the minimum section possible to construct by dumping is adequate for the cofferdam. (Stubbs, F.W., 1959).

The chief precaution necessary with this type is to have a good seal, because with a poor seal it is difficult to locate the point of leakage and make adequate repairs after the cofferdam has been dewatered. This condition becomes even more serious on porous foundation where the rockfill itself may be quite impervious but water is passing into the cofferdam through dame of the rock fissures at the base. Owing to the irregular face of the rockfill and the tendency for large boulders to roll down and spread over a considerable area of the river bed, it is quite difficult to locate such bases and seal them effectively.

Rockfill dams with upstream membranes are almost never used because installing the membrane subsequent to placing the fill requires too long a time and because of the difficulty in constructing the upstream toe foundation for the membrane. (ICOLD Bulletin 48a, 1986),

A clay core or fill with precise specifications and moisture control (especially if borrow pits are on the wet side) may be difficult to place in short time and the high rate of placing large quantities of sophisticated filter material which may be necessary, requires preparation plants which are to always available at the beginning of the work.

iii. Concrete or masonry cofferdams

Gravity type structures are often used for small cofferdams, these having the advantage of being able to accept overtopping. But cost and construction time do not favour them for medium to high structures.

Arch cofferdams have been used successfully with a design either very similar to permanent arches when used on single stage scheme, or as a large part of the whole of a circular protection for one phase of a multiple stage scheme. Kariba experience was very interesting with the cofferdam height reaching 40m. to allow rapid construction, the design may be simpler than for permanent structures either as regards geometry (vertical circular arches are often used) or in the treatment of joints. Part of the foundation may be constructed under water (for instance by colcrete or underwater concrete between two sheetpile curtains and improved by grouting) but it is essential that the foundation area receiving the main arch forces should be well founded. (Singh, B. & Varshney, R.S. 1995)

vi Steel sheet pile cofferdams

Steel –sheet pile cofferdams are now used instead of timber-crib cofferdams when space is limited, where high water velocities occur along the cofferdam sides, and where cofferdam sides, and where cofferdam closure must be made against a high differential head causing high velocity during cofferdam closure. Besides having a cost advantage over timber cribs, they can give a more positive water cutoff since the piles can be driven through overburden material to rock. Steel sheet piles are used in circular cell cofferdams, double diaphragm-wall cofferdams and as a single-diaphragm wall reinforced with a rock fill. At Kentucky River Dam, Cabora Bassa Dam and Rocky Reach Dam single diaphragm wall type was adopted.

Cellular cofferdams are used when large, deep cofferdams are required since these units are individually stable and do not require bracing, the use of individual cells has an advantage in that each cell can be completely filled after erection to allow progressive construction. Such cells must be designed to prevent overturning, sliding, and rupturing of the piles in the interlocks.

Steel sheet-pile cellular cofferdams consist of interconnected cells constructed of interlocking steel sheet piles filled with earth, preferably a free draining material such as sand or gravel. The more common types are circular and diaphragm as shown in Fig. 2.7.3. Circular type consists of circular cells of interlocking sheet piles, the segments being attached o the circular units by means of piles incorporated in the circles. Diaphragm type consists of two rows of segments of circles whose chords are perpendicular with diaphragms connecting the two rows of segments of circles whose chords are perpendicular with diaphragms connecting the two rows of segments at the end of each segment. Circular type has this advantage of permitting complete filling of each major cell independently of the others, whereas the fill in diaphragm type must be brought to approximately the same elevation of each side of the straight diaphragm. The height of this type of cofferdam is limited only by the strength of the interlock.

For low heads, cofferdams maybe constructed of sheet piles, either of tongue – and-groove wood or of interlocking steel. These are driven in parallel rows with tie rods and wales to tie the two rows together as shown in Fig. 2.7.4.

Precautions that may be needed for high cellular cofferdams include.

- Grouting of rock below the cofferdam to ensure tightness against seepage and uplift.
- Adequate draining of berms of where required for stability.
- Ripping of berm slopes.

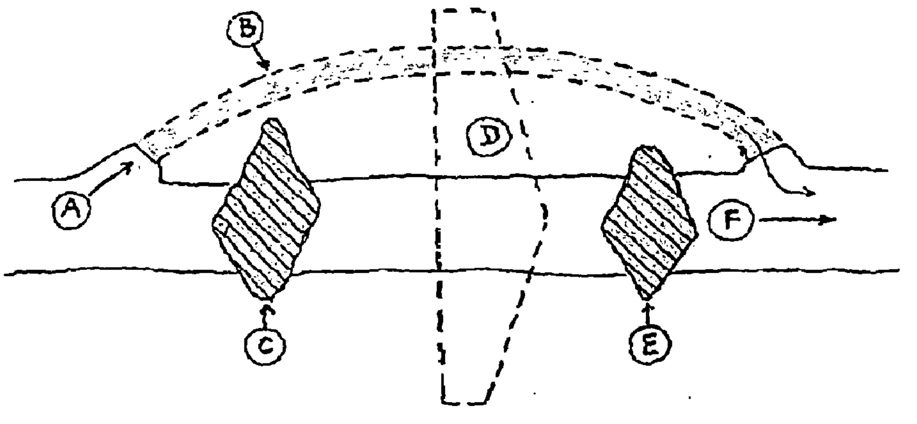
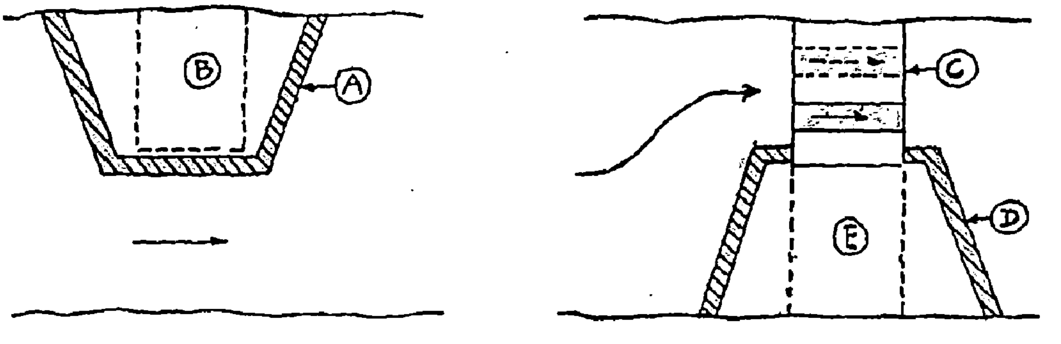


Fig. 2.4.1: Single stage river diversion

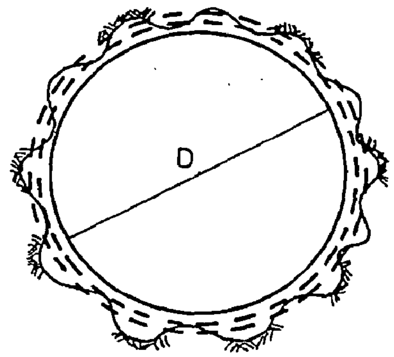
A. Diversion tunnel (canal, conduit, culvert) inlet
B. Tunnel, conduit, canal or culvert
C. Upstream cofferdam
D. Main dam to be constructed
E. Downstream cofferdam
F. River bed



First stage Second stage

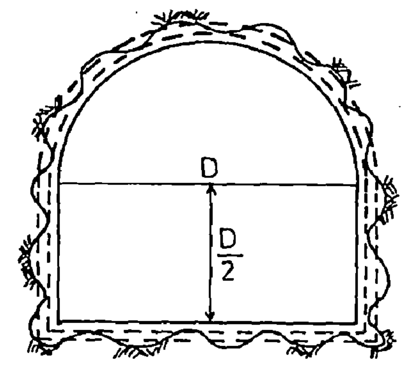
Fig. 2.4.2: Two stage river diversion

A. First stage cofferdam
B. Dam and diversion openings under construction
C. Dam and diversion opening completed
D. Second stage cofferdam
E. Permanent works under construction



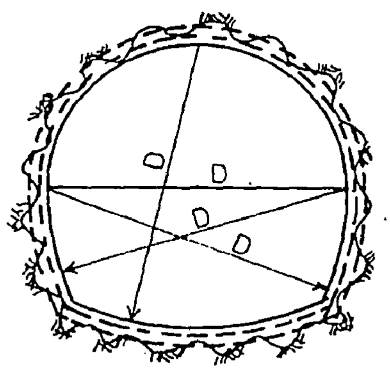
Circular

$A = 0.7854\ D^2$



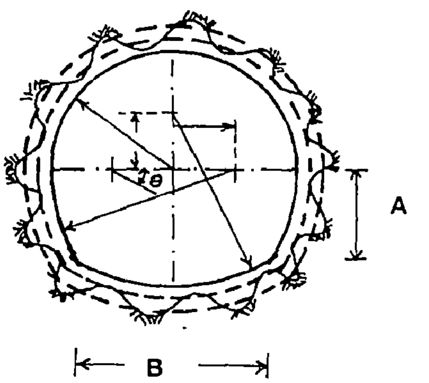
D- Shaped

$A = 0.905\ D^2$



Standard Horse Shoe

$A = 0.8293\ D^2$



Modified Horse Shoe

r	$= 0.9876\ R$
A	$= 0.7808\ r$
B	$= 1.5616\ r$
θ	$= 31^0\ 22'\ 01''$

Fig. 2.5.1: Different shapes of tunnel cross section generally adopted

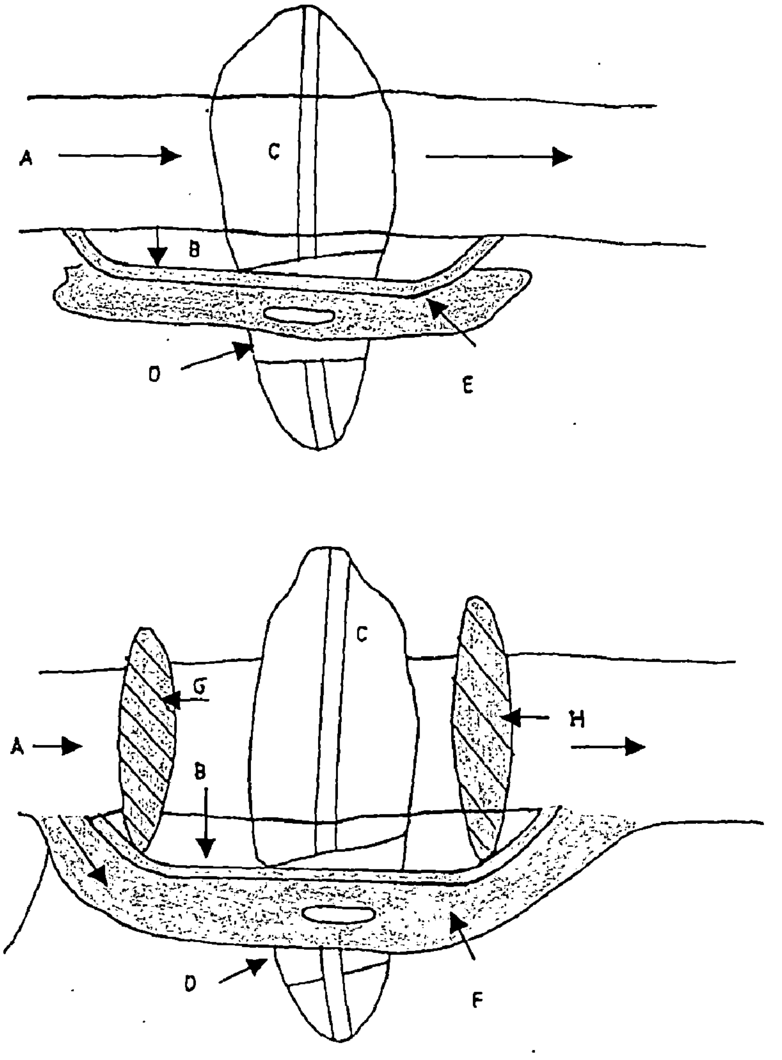


Fig. 2.5.2: River diversion by means of a channel

A- flows; B- lateral cofferdam; C- main dam; D-temporary opening (one or more)
E- channel (partially completed); F-channel completed; G-u/s cofferdam
H- d/s cofferdam

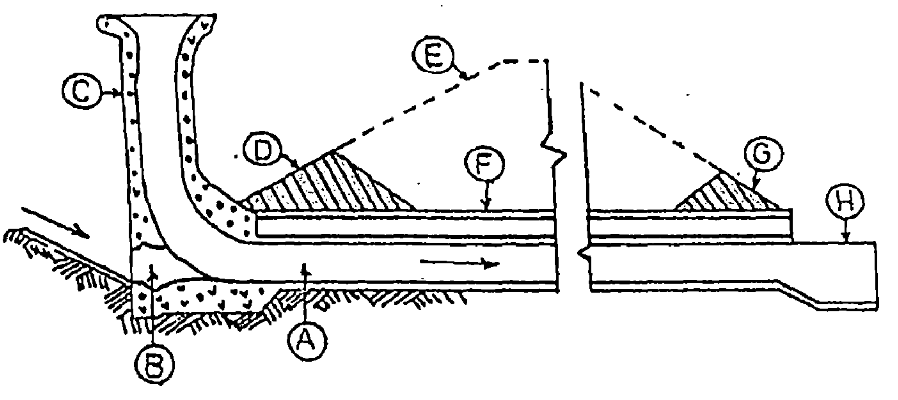


Fig. 2.5.3: Main components of river diversion by conduit

A- conduit; B- river diversion opening (plugged at closure); C-morning glory spillway or intake tower; D- upstream cofferdam incorporated in main embankment; E-main embankment; F- upper gallery housing outlet pipes; G- downstream cofferdam incorporated in main dam; H- stilling basin

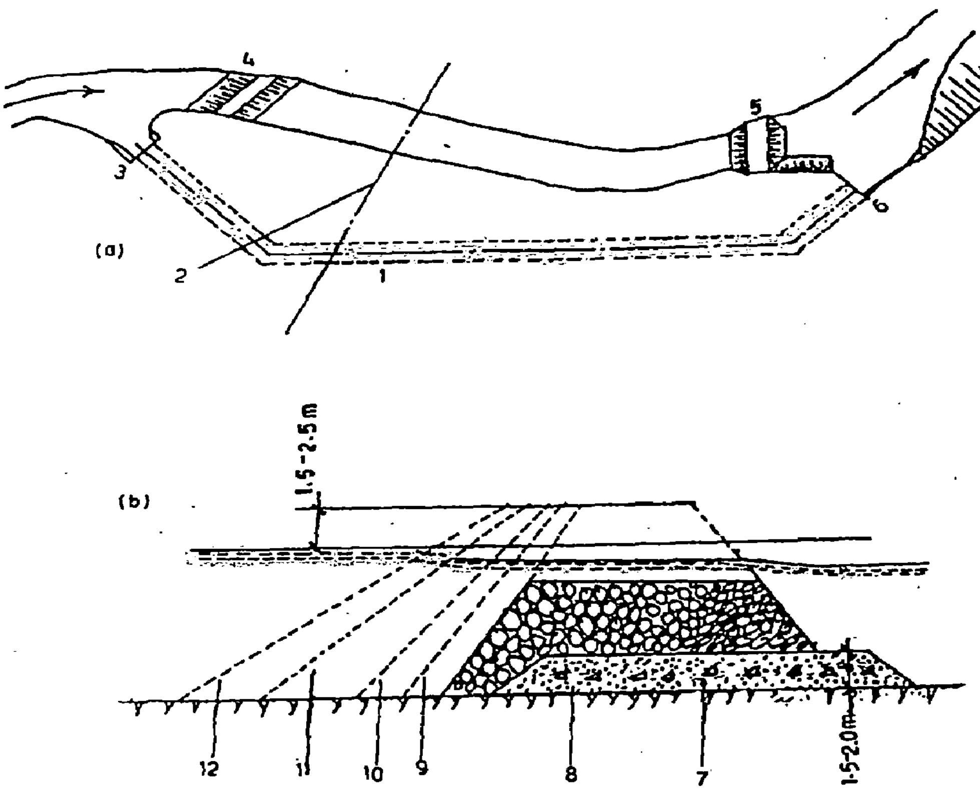


Fig. 2.6.1: River diversion in tunnel and closure by hydraulic filling of river bed

1- tunnel; 2-river section at dam site; 3,6- upstream and downstream tunnel portals; 4,5- upstream and downstream cofferdams; 7- gravel sand mass; 8- tetrahedrons; 9- gravel; 10- sand; 11- loam; 12- stone

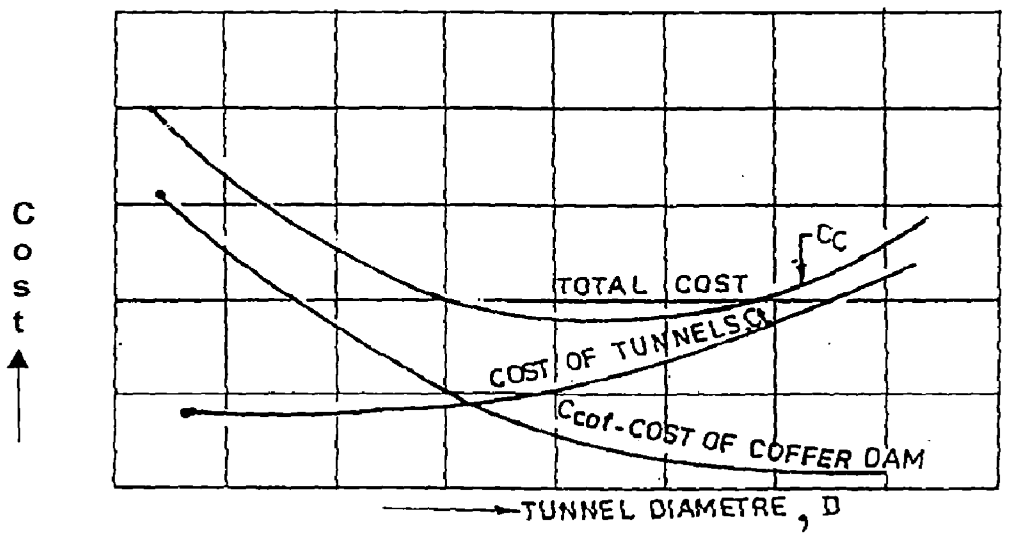


Fig. 2.7.1: Relationship of tunnel size to cofferdam height

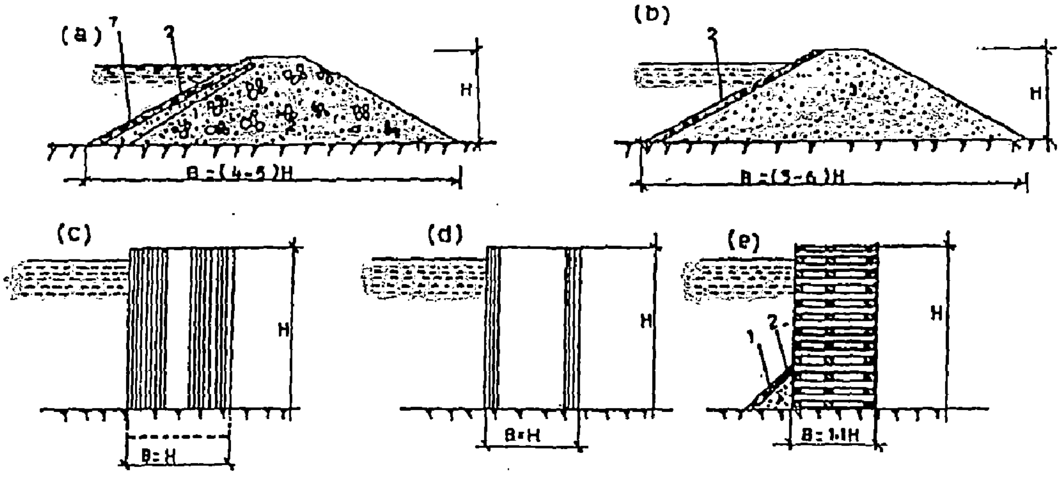
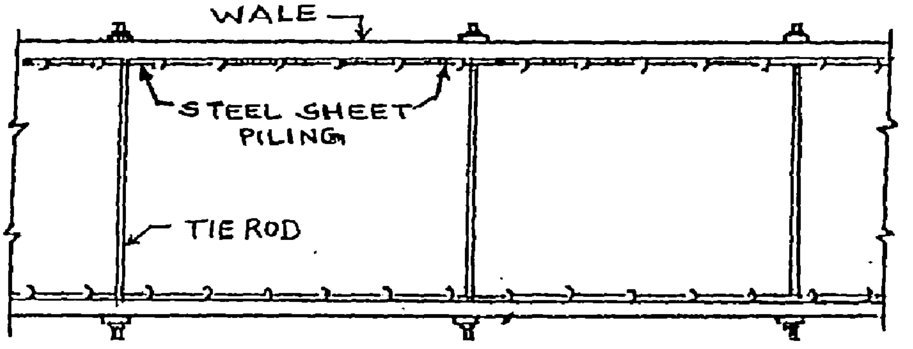
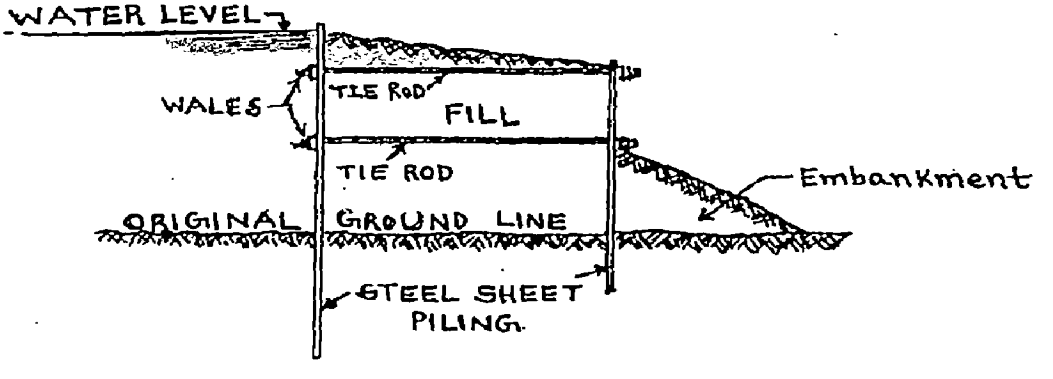


Fig. 2.7.2: Different types of cofferdams

a- riprap with loam screen; b-sand; c-cylindrical lattice of steel sheet pile; d- radial lattice of steel sheet pile



PLAN



TYPICAL SECTION

Fig. 2.7.4: Cofferdam of parallel rows of sheet piling

REVIEW OF DIVERSION ARRANGEMENTS ADOPTED FOR SOME CONCRETE DAMS

3.1 General

a. Narrow valleys

The most common method of river diversion for concrete dams in narrow valleys is through tunnels constructed through the abutments. The diversion is effected by construction of cofferdams both on the upstream and downstream of the construction area. The tunnels designed for a capacity sufficient to take the maximum flood flow have the advantage of not interfering with the foundation excavation and the dam construction work during construction period. Except on small rivers usually two tunnels one in each abutment are employed for safety and convenience [Sharma, H.D., 1998].

For concrete dams where conditions permit diversion tunnels are constructed to bye pass only low river flows for a sufficient period of time to permit work. Some of the sections of the dam are left low or openings are left to take high river discharge with construction of the balance work continuing during high river stages.

In case if the rock is of poor quality concrete culverts in the dam foundation may be preferable instead of tunnels. The culverts are built in the dry behind cofferdams and they can be closed off in the same way as tunnels. On a small stream a flume may be used for carrying the stream flow along one abutment, water being restrained from entering the dam foundation area by upstream and downstream cofferdams. Flumes may be constructed of wood steel or concrete as part of the permanent dam construction. If flows in excess of flume capacity are to occur, arrangements must be made to breach the cofferdam to allow flooding without damage. Care must be exercised to make a tight cut off around the flume at the cofferdams and arrangements made for stop-logs or other measures to close the flume. Diversion conduits at stream level are some times provided through the dam.

b. Wide rivers

The river floods may be so large that it is neither economical nor practical to provide diversion passages even for average floods, the only alternative being to pass them through or over the dam. On most wide rivers, the diversion has to proceed in stages and this is the most common arrangement where characteristics of the site and stream, type of dam and construction schedule permit.

Method of diversion

Construct a first stage cofferdam around a portion of the work. This may sometimes involve cutting in to the opposite bank to provide adequate discharge capacity of the river. A portion of the dam is then constructed within this area leaving permanent or temporary sluiceways to take the river flow during subsequent stages of construction. After the work within the first stage cofferdam has progressed sufficiently to permit completion after flooding, the cofferdam is removed to allow diversion through sluiceways and another section of dam is enclosed within a second cofferdam.

3.2. Diversion Arrangements for Concrete Dams

3.2.1 EL Cajon Dam

a. Project description

El Cajon is a multipurpose hydro project in the central part of Honduras, its main purpose being energy generation with secondary benefits of flood control and improvement of downstream irrigation. The dam site is located in an 800 m long, narrow canyon. The dam is an arch gravity type with maximum height of 226 m and crest length of 382 m. Thickness at crest is 7 m and at the base it is 48 m.

Catchment area of the dam site is 8320Km^2 with a reservoir water surface of 530000 sq. Km, it is perhaps the largest ever executed for a hydroproject. Mean annual rainfall is 1300mm. Annual average inflow volume is 3500Mm^3. General layout of works at El Cajon Dam is shown in Fig. 3.2.1.

b. River diversion scheme

The diversion of the Humuya river at EL Cajon dam site during two and a half years of construction was accomplished by a scheme of two cofferdams and one diversion tunnel. The arrangement and the dimensions of the diversion scheme are the result of a least cost evaluation consistent with the construction schedule. A 13 m dia diversion tunnel 570 m length was excavated in the right abutment. Both upstream and downstream cofferdams are of rockfill type with maximum height of 40 m and 15 m respectively. The upstream cofferdam has a steel sheet pile as a centre-sealing wall. Fig. 3.2.2 shows river diversion scheme adopted. Both the cofferdams were designed for overtopping up to 6 m. For this hydraulic model tests were conducted on a detailed model of the upstream cofferdam, scaled 1:40 at the laboratory of Hydraulics, Hydrology and Glaciology in Zurich, Switzerland. The capacity of the diversion scheme is 4500 m^3/s, a flood of 40-year return period. Three of the four cofferdam slopes are protected by gabions to resist hydraulic forces during overtopping. Erosion resistance was checked by hydraulic model testing.

3.2.2. Dworshak Dam

a. Project description

Dworshak Dam, which is constructed on the Northfork clear water River, Idaho, is the highest straight axis concrete dam in the United States. Benefits from the project include flood control, hydroelectric power and recreation.

The initial power plant installation will consist of two units at 90,000 kw each and one at 2,20,000 Kw. The dam is 218.5 m high with a crest length of 1001.9m. The gross and usable capacities of the reservoir are 4.259 Mm^3 and 2.427 Mm^3 respectively. The drainage area of the basin is 6320 Sq. Km. The mean flow at the site is 159.6 m^3/s.

b. River diversion scheme

i. Diversion criteria

The diversion problem was complicated by the necessity of providing safe passage for a log drive of 137000 m^3 of timber each year (most of the logs were from 4.9 to 6.1 m long, but occasionally longer ones up to 9.8 m) and for the annual run of steelhead trout that spawned above the dam site. General plan of diversion facilities is shown in Fig. 3.2.3.

Hydrologic and economic studies dictated that the diversion facilities should be larger enough to pass a flood having a probable frequency of occurrence of once in 25 years (1930 m^3/s). A tunnel excavated in the left abutment was chosen at best meeting these criteria. The tunnel was excavated in the dry through the left abutment of the dam and a small temporary dike was constructed by conventional end dumping of random sized rock, to divert the river into the completed tunnel. In October 1966 water first entered the tunnel and by December the main upstream cofferdam was completed and by January 1967 the down stream cofferdam was completed.

i. Tunnel design

As design studies proceeded it became evident that several corollary requirements must be met inorder to satisfy the three major criteria outlined below.

- The tunnel must be lined with concrete to avoid the possibility of logs jamming in a location that could not possibly cleared from the outside.
- There could not be a dividing pier at the tunnel intake A pier would have greatly simplified the design of the intake closure gate by reducing the span.
- During log passage there should be a free water surface with a minimum freeboard of 1.8 m below the tunnel roof.

A five-year flood (1245 m^3) was selected and at the intake portal it was found expedient to raise the roof from the nominal height of 12.2 m to 16.4 m and reduce the width to 10.7m. Cross section of diversion tunnel is shown in Fig. 3.2.4.

iii. Hydraulic model studies

A model of the entire tunnel including intake and outlet, fish trap and approaches was built to a scale of 1:50 at the Corps of Engineers Hydraulic Laboratory, Bonneville, Oregon.

The tunnel originally proposed was 479.8 m long with the central portion curved in plan, having a deflection angle of 58^0 30' and 182.9 m radius. The entrance and exit conditions proved to be totally unsatisfactory and flow conditions were very rough. Consequently, a new alignment was tested, the length increased to 524.6 m and the curvature eased to a 38^0 deflection angle with a radius of 609.6 m flow conditions improved immensely with in the tunnel as well as at the intake and outlet portals. Model logs of varying sizes all aligned themselves axially as they approached the tunnel inlet it was found practically impossible to form a jam.

An energy dissipation scheme that would provide a continuous downstream surface flow and still permit good approach conditions for the fish was adopted. The plan consisted of four 0.9 m high by 3.0 m wide transverse concrete baffles in the channel below the sill. These successfully eliminated the backroll and permitted the logs to pass unimpeded.

3.2.3. Glen Canyon Dam

a. Project description

Glen Canyon Dam was constructed on the Colorado River in north-central Arizona, it was the second highest dam in the Western Hemisphere. The reservoir impounded by the dam, named Lake Powell has a total storage capacity of 33304 Mm^3 and will extend 186 miles up the Colorado River and 71 miles upstream on the San Juan River.

The dam is a constant-radius concrete arch with fillets. It has a structural height of 216m and a crest length of approximately 473 m. The crest of the dam is 177 m above the riverbed, and accommodates a 11 m-wide roadway, which is a service road for the dam and provides access between the spillway. General layout of works is shown in Fig. 3.2.5.

b. River diversion scheme

Diversion of the Colorado River during construction of Glen Canyon Dam was accomplished through two 12.5 m diameter concrete-lined tunnels, one located in each abutment as shown in figure 3.2.5. The size of the tunnels was determined from routing studies made using the 25-year frequency flood which has a peak flow of 5551 m^3/s. Routing of this flood through the tunnels resulted in maximum reservoir water surface elevation of 3277 and a combined discharge of 4050 m^3/s. The height of upstream and downstream cofferdams is 51 m and 10 m respectively. The diversion tunnel through the right abutment is 833 m approximately 276 m of the downstream portion of the diversion tunnel was later incorporation into the right spillway tunnel. The diversion tunnel through the left abutment is 912 m approximately 329 m of the downstream portion of the diversion tunnel was later incorporated into the left spillway tunnel. The left tunnel entrance was set at the higher elevation so that a temporary outlet works could be installed in the left tunnel plug during a low flow season without the use of an entrance closure structure.

After the construction of dam and power units had advanced sufficiently, the upstream portions of these diversion tunnels were abandoned and permanent seals or plugs were installed to close them. The temporary outlet works consisting of three 2.12 m × 3.20 m outlets controlled by 2.12 m × 3.20 m slide gates in tandem, were located in the left tunnel plug to control releases during reservoir filling.

3.2.4. Itaipu Hydro Electric Project

a. Project description

The main structure of Itaipu Hydroelectric project at Rio Parana border between Brazil and Paraguay comprise of a 196 m high concrete dam with 18 unit power plant at the toe of the main dam (14 units) and the diversion control structure (4 units) with an installed capacity of 12600 MW.

The principal features of Itaipu project are : diversion control structure, main cofferdam, main dam, rockfill dam, left and right earth fill dams, spillway and powerhouse.

At the Itaipu site the Paraguay River flows almost due south and is about 400m-wide at the average flow condition. At its bed about 400 m wide at the average flow condition. At its bed the river channel is 250 m wide. For the first 80 m both banks rise at 40^0 then flatten to about 30^0 and reach almost flat plateau on the both sides at a height of about 130m.

Drainage area of the Rio Parana above Itaipu is 8,20,000 Km^2. The average and maximum daily flows at the site are 8500 m^3/s and 2920 m^3/s respectively. The historical maximum daily flow at the Guaira, 190 Km upstream of Itaipu was 28,400 $m^{3/s}$.

b. River diversion scheme

It was decided that the river diversion works should be designed to protect construction of permanent project features against a maximum flood of 30,000 m^3/s. Statistically this is estimated to be a 100-year flood. Layout of diversion works has been shown in Fig. 3.2.6.

Principal features of the Itaipu diversion works.

- Diversion channel of two kilometers length has a bed width of 100 m which widens to 150 m at the entrance with nearly vertical side slopes and designed to pass a flow of 30,000 m^3/s, without overtopping the main cofferdams. This channel was located at left abutment.
- Diversion control structure is a straight gravity concrete dam of height 160 m aligned on the same axis as the main dam. with twelve sluices each 6.7m wide and 22.0 m high, in the diversion channel.
- Auxiliary arch cofferdams in the diversion channel.
- Main cofferdams in the river with two rockfill dykes for each cofferdam (dikes A and B for upstream cofferdam and dikes C and D) for downstream cofferdam.

Construction of diversion works of this project consisted of the following steps :

- Excavation of the diversion channel with two natural rock plugs.
- Construction of the first stage of the diversion control structure.
- Under water excavation at the entrance and exit of diversion channel.
- Construction of the first stage of dykes A, B and D of the main cofferdams.
- Construction of the arch cofferdams in the diversion channel.

- Demolition of the natural rock walls or plugs at the entrance and exit of the diversion channel.
- Demolition of the arch cofferdams and diversion of river flow through the diversion channel.
- Closure of dykes B and D.
- Closure of dykes A and C.
- Dredging of sand deposits between dykes A and B and between C and D.
- Placement of clay by the squeeze displacements method in nearly still water for the cores of the cofferdams.
- Completion of the two main cofferdams to their full height.
- Dewatering the work area between the cofferdams.

Construction of above diversion works especially the main cofferdams were not easy due to the twin considerations of large depth of water and high flows. During relatively low flow season from May to November when the average discharge is 6500 m^3/sec, river depth will range from 30 to 35 m, and from October to April when the average discharge is about 10,000 m^3/sec the water at the sites of cofferdams would be an additional 5 m deeper. For this condition, conventional types of cofferdams, consisting of dumped rockfill with a dumped impervious zone on the outer or inflow side, would not be feasible.

An extensive series of hydraulic model test, was carried out, and the following finally adopted:

- Each cofferdam shall have two rockfill dikes, which could be built during the low flow seasons.
- Adequate space shall be provided between two rockfill dykes of each cofferdam for the impervious core.
- The clay in the two cores shall be placed in the water by the 'squeeze displacement' method.
- Upper portions of cofferdams shall have the section of a conventional central-core rockfill dam, constructed of compacted material.

The different stages of construction developed from design analysis and the hydraulic model tests, were as follows (Fig. 3.2.7 & 3.2.8).

Stage I	:	Partial construction of dikes A, B and D from the right bank, prior to opening of the diversion channel .

Stage II : Closure of dikes B and D, following the opening of diversion channel.

Stage III : Closure of dikes A and C. Dredging and cleaning of riverbed between the dikes of each cofferdam. Placement of clay in water to complete the cores to just above the expected water level.

Stage IV : Completion of the above water portion of the cofferdams to their final height, and start of dewatering

3.2.5 New Bullards Bar Dam

a. Project description

New Bullards Bar dam is located on the North Yuba river approximately 225 km northeast of San Francisco, California. It is a 193 m high double curvature concrete arch dam with a crest length of 716 m.

New Bullards Bar dam is located in a relatively narrow valley of the North Yuba river and the watershed covers an area of 1270 km^2 of the Sierra Nevada ranging in elevation from 580 to 2745 m above sea level. Average annual precipitation ranges from 404 to 1650 mm. About 55% of the run off is a contribution from snowmelt. The average annual basin run off is 1204.6 Mm^3 and the average daily stream flow ranges from 5.5 m^3/s to 85.5 m^3/s.

b. River diversion scheme

The design, construction planning and schedule of the New Bullards Bar Dam provided for safe handling of flood flows at three stages: (i) during excavation of foundations (ii) during storage behind the partially completed dam, and (ii) up on completion of the dam and the spillway. Since it is a high, multipurpose dam, where earliest storage of water is an economic requirement, control of high river flows with minimum adverse impact on the rate of construction and without damage to permanent facilities is essential.

Since the natural riverbed was only 30 m wide and rock formations were exposed at both abutments for 45 m above streambed, a tunnel was adopted to divert the river around the foundation of the dam. A 12.8 m x 13.4 m horseshoe shaped, 1475 m long tunnel was located under the left abutment to discharge down stream of an existing small dam.

To handle the equivalent of the December 1964 flood of record, a 61 m high rock fill embankment was constructed as the main upstream cofferdam. Fig. 3.2.9 shows the general layout of the diversion tunnel and cofferdams.

The diversion tunnel, which was excavated in sound amphibolites did not require any permanent supports and was unlined. During two subsequent wet seasons, the maximum discharge through the tunnel was approximately 565 m^3/s and the upstream cofferdam was never close to being overtopped.

Project schedules required storage of adequate quantities of water to assure full power operation before June 1970. Since analyses of records for dry years indicated that to achieve this goal, storage should commence in the fall of 1968, it was decided to permanently plug the diversion tunnel.

Flood handling during initial storage

The storage capacity of the reservoir following closure of the tunnel was not adequate to store the flood on record without overtopping the dam. Consequently simultaneously with tunnel closure the following arrangements were made to meet such an eventuality.

- The top of central block (no. 17) of New Bullards Bar Dam was maintained 6.9 m below the next higher block. This block was selected since any flow over it would not impinge either on a permanent appurtenance or on either abutment of the dam but would land in the 10.7 m deep tail water pond.
- A temporary outlet, controlled by the 4.57 m dia butterfly valve normally to be used for closure of the power penstock was installed.
- The 1.83 m diameter permanent low level outlet valve was to be maintained in the open position.

The temporary outlet was considered necessary to facilitate draw down of the reservoir in the event of continuous high inflow following initial filling. As a normal operational requirement the 4.57 m butterfly valve was designed to open under balanced pressure and to close under full head and full penstock flow condition. Laboratory model test were made to design the outlet nozzle. The result was a nozzle tapering to 2.75 m at the discharge end. The 45^0 elbow was adopted to direct the jet into the middle of the river channel and at the same time direct the thrust in to the sound rock foundation.

3.2.6 Mossyrock dam

a. Project description

The Mossyrock dam, powerhouse and other facilities comprise the second of two developments of the Cowlitz river power project of the department of public utilities of the city of Tacoma, Washington. The dam is a double curvature multicenter concrete arch with a maximum height of 185 m. It is 394 m long at its crest and has a top thickness of 8.2 m and 38.0 m at the base. The reservoir known as Davisson lake, extends 23.5 miles upstream and will provide a usable storage capacity of approximately 1600 Mm^3 with a draw down of 52 m.

Mean annual run off at Mossyrock is 144 m^3/s. Snowmelt contributes substantially to the spring run off at Mossyrock is 144m^3/s. Snowmelt contributes substantially to the spring run offs. Most of the peak flows occur between November and March its highest recorded flood is with a peak of about 2365 m^3/s. The probable maximum flood would have a peak inflow to the Mossyrock reservoir of 8496 m^3/s and outflow of 7816 m^3/s. The reservoir will provide substantial flood control benefits down stream.

b. River diversion scheme

The river flows were diverted around the dam site in two Gothic-shaped tunnels, 12 m high and 10 m wide excavated in the left abutment (Fig. 3.2.10). The tunnels were unlined; one is 455 m long, the other 561 m. The selection of the Gothic shape tunnel type was dictated by strength reasons and by the requirements for moving migrating fish past the site during construction the unlined sections providing the best hydraulic and structural shapes for this purpose. The square corners at the base of the section of low water velocity and a natural passageways fools. The shape was also found particularly suitable for enlarging to form on each side of the tunnel at 30 m intervals.

The tunnels were designed for a diversion flow of 1540 m^3/s that is, a flood of 13 years return interval and the tunnels ran under free flow at 0.9 of the depth for full discharge with an average water velocity of 7.6 m/s.

3.2.7 Revelstoke Dam

a. Project description

The 2400 MW Revelstoke hydroelectric project of the British Columbia Hydro and Power Authority occupies the last site for a major new development on the concrete revetments on both cofferdams, it was possible to avoid straining the site services available at the time to keep to the scheduled programme.

3.2.9 Srisailam Dam

a. Project description

The Srisailam Hydro Electric Project forms part of the scheme for integrated development of the water resources of river Krishna in the state of Andhra Pradesh in India. The Srisailam project is situated at a point 869 km down stream of the origin of river Krishna. River Krishna is the second biggest river in peninsular India draining an area of 2,48,448 km^2 during its course of 1240 km.

The project involves the construction of a masonry dam of straight gravity type with an overall length of 512 m and a maximum height of about 144 m from the deepest foundation level the reservoir farmed behind the dam will have a storage capacity of 8720 mm^3. The spillway portion is 266.4 m long having 12 bays of 18.3 m clear span each and will be controlled by 18.3 m × 16.7 m radial gates. It will have a discharging capacity of 37,380 m^3/s. The non-overflow of the dam is on either side of spillway portion.

A powerhouse is constructed on the right side of the gorge at a distance of 457.2 m downstream of the axis of the dam. The powerhouse has 7nos of generating units of 110 MW each.

b. River diversion scheme

In summer months, the river flows dwindle down to less than 28.32 m^3/s. The yearly maximum flood commonly observed in river Krishna at the dam site is of the order of 18,408 m^3/s during monsoon period (June-October). It would be very costly to provide for a diversion capacity of such a huge magnitude. The maximum river discharge data for fifteen consecutive years were studied to select the optimum discharge, which would give maximum working period. The studies indicated that the diversion arrangements designed for 849.6 m^3/s capacity would ensure about 195 working days during the period from Nov – June. Diversion arrangements were therefore designed for a capacity of 849.6 m^3/s.

The diversion arrangements designed for non-monsoon discharge of 849.6 m^3/s finally adopted (Fig. 3.2.14), comprised –

i. A 9.14 m diameter circular tunnel of 566.4 m^3/s capacity through the left abutment.

ii. A diversion channel of 15.24 m bed width to carry the balance of 283.2 m^3/s.

iii. Two semi-permanent colcrete upstream and downstream cofferdams, to divert the river flows for isolation of the construction area.

The diversion tunnel is lined with concrete (minimum thickness 400 mm) and is fitted with regulating gates in shaft located 32 m downstream of the entrance. The diversion channel of 15.24 m wide and 747 m long was excavated in the rock ledge on the right side. The bed of the channel was lined while the sides were left unlined. The diversion channel will be kept in operation to supplement the diversion capacity of the tunnel. Three sluices of size 1.22 m x 1.52 m were provided in the left sidewall of the diversion channel for admitting water (to serve as water cushion) in to the main dam foundation area between two cofferdams.

The main problems in the formation of the cofferdams were the bouldary bed of the deep channel course and the high minimum water level over the bed due to backwater from Nagarjuna Sagar Dam located further downstream. More than half the height of the upstream cofferdam and practically the full height of downstream cofferdam were to be constructed under water. In view of the bouldary fill in the riverbed and the considerable difference between the water levels on either side of the cofferdams, the treatment of the cofferdam foundations required very careful consideration in order to reduce the seepage into the foundation area.

In addition, the cofferdams were also to withstand the overtopping of floods for a few seasons. Considering all the above factors, cofferdams of semi permanent nature, which could withstand the overtopping were to be constructed.

The colcreting technique was adopted for construction of the cofferdams. Under water construction made it obligatory to provide vertical sides for the cofferdams (16.5 m wide for upstream and 21.3 m wide for downstream) upto prevailing water level. A masonry wall was then built over the colcrete cofferdam.

The foundation bed below the upstream cofferdam consisted of boulders and sand to more than 30 m depth, whereas the downstream cofferdam rested on rock muck deposited from earlier construction. The measures to impermeablise the foundation comprised provision of upstream grout curtain extending to bedrock, grouting the foundation contact, and treatment of the junction of the colcrete and the abutments at the flanks with cement grout. Protection against scour was also provided by placing gabions downstream of the cofferdams as shown in Fig. 3.2.15. Both upstream and downstream cofferdams finally had been built successfully and the seepage into foundation pit even with a differential head of 40 m was only of the order of 0.17 m^3/sec.

3.2.10 Supa Dam

a. Project description

Supa dam is the main storage reservoir for entire Kalinadi Hydroelectric project in Karnataka state of India. It is a 101 m high concrete gravity dam, highest in south India. This dam creates main storage reservoir for the entire project with a total capacity of 4300 Mm^3.

The total length of the dam of 322 m is divided in to 20 wedge shaped blocks to suit the geology of the terrain and to facilitate speedier construction. The water impounded in Supa reservoir is being utilized for power generation at Nagajhari Power house.

b. River diversion scheme

River diversion scheme was designed to carry a maximum fair weather discharge of 170 m^3/s expected in the month of October. This enabled carrying out construction activities in the riverbed blocks during the period form October to May every year. River diversion structure comprising of masonry walls of 7 m height is constructed of proportion 1:4 weight.

The scheme contemplated consists of construction of

- A cofferdam wall across the river at a distance of about 70 m upstream of the axis of dam with necessary gap to allow water to flow to the river diversion conduit.
- A wall along the river course at the upstream side of Block No. 11 to channelise the water to the river diversion conduit.
- A wall parallel to the river course in the foundation area of block No: 12 near the block joint 11/12.
- A wall parallel to the river course at the downstream side of block No: 11.
- RCC river diversion conduit of 3 m width and 4 m height in block No: 11 with a bed slope of $1^0\ 40'$ to the horizontal. The floor of the conduit was made tangential to the apron near the exit. Bell mouth entrance was provided at the upstream side of the conduit (Fig. 3.2.16).

Construction of cofferdam walls for the river diversion scheme involves 9000 cum of masonry. Fair weather flow was diverted initially on the right bank blocks 12 and 13. Exposure of the foundation grade in the riverbed blocks 7 to 11 on the left bank was taken up. After exposing the general foundation grade of block No. 11, a weathered zone was encountered at the heel side of block no.11. Treatment of this weathered zone took considerable time since the acceptable rock was available at a depth of about 5 m below the general foundation level.

Further, there was an apprehension that this weathered zone might extend in the foundation of the adjacent block No: 12 also. In such an eventuality there was need to take up back filling of the excavated trench monolithically in blocks 11 & 12 after excavating the weathered zone in block No: 12. Hence, if the river diversion conduit was persisted for construction in block No: 11, diversion of river through the conduit would have been delayed. Thus there was a necessity to shift the location of the conduit from block No. 11 to block No. 10.

The revised river diversion scheme as shown in Fig. 3.2.17 involved construction of an additional masonry wall parallel to the river course at the upstream side of block No: 10 (i.e along the extended block joint 9/10). This helped in channelisation of the river through the conduit in block No:10. The masonry of the cofferdam wall at the upstream of the river diversion conduit in block No. 10 was dismantled to divert the water through the channel.

The gap provided earlier in the cofferdam was plugged by masonry. In addition approximate bell mouth shape was provided in the channel walls in masonry at the entrance of the conduit. The river was ultimately diverted through the conduit formed in block No: 10. After plugging of the diversion conduit two nos. of river sluices of size 2.14 × 3.2 m with necessary gates are provided one each in each of the blocks 10 and 11 to enable regulation of the flow from the reservoir.

3.3 Summary of the Review, study and the inferences

3.3.1 Summary of the review

The review of diversion arrangement adopted for existing concrete dams is summarized in Table 3.3.1 and important inferences are drawn from the experience of diversion schemes which have been studied.

3.3.2 Study and the inferences

3.3.2.1 Inferences drawn from the review of diversion arrangements adopted for existing Concrete Dams.

From the review it can be inferred that normally in case of concrete dam, diversion and care of the river is not a big problem because concrete dams can undergo overtopping without much damage, particularly if openings or blocks are provided intentionally at low level (New Bullards Bar, Supa, Revelstoke, Mossyrock, Glen Canyon).

Type of valley

In case of steep sided rock valleys for Concrete dams, the most common method of river diversion adopted is through tunnels constructed through the abutments. The diversion is effected by construction of cofferdams both on the upstream and downstream of the construction area (Dworshak, El Cajon, New Bullards Bar, Cabora Bassa, Mossyrock).

Tunnels were used when the rock is good. If the rock is poor for efficient tunneling and valley is wide enough or where the high flow makes tunnels uneconomic, diversion channels are being used for carrying the stream flow (Itaipu,). At some dams the diversion tunnel was used in combination with open channel (Srisailam).

Diversion Flood

In case of concrete dam, which could pass substantial amount of flood without appreciable damage, considerably less diversion capacity would be required than for an Earth & Rockfill dam. Usual values of frequency of design flood adopted for concrete dams vary from 1 in 10 to 1 in 50 years (Mossyrock, Revelstoke, Dworshak, Glen Canyon, El Cajon). And the diversion arrangement was designed for maximum dry season flow in most of the remaining dams reviewed.

Method of Diversion

On the basis of review it has been noticed that in most of the cases during low flow season the river diversion was carried out through tunnels or channels for a sufficient period of time to permit excavation and construction of the dam to streambed. Low construction blocks or diversion conduits were provided to take high-river discharges (El Cajon, New Bullards Bar, Revelstoke, Dworshak, Glen Canyon, Supa).

In large number of cases the diversion arrangement design was developed with the help of models.

TABLE 3.3.1: SUMMARY OF DIVERSION ARRANGEMENTS ADOPTED FOR SOME CONCRETE DAMS

S. No.	Name of Dam	Type of Dam	Height (m)	Country	Diversion Flood (m^3/s)	Return Period (yrs.)	Type of Valley	Type of Rock	Diversion Arrangement		Remarks
									Waterway	Height of u/s Coffer Dam (m)	
1.	EL CAJON	Arch Gravity	226.0	Honduras	4500	40	Narrow	Limestone	TUNNEL(1) Dia: 13 m	40	Both u/s and d/s cofferdams were designed to be overtopped upto 6 m.
2.	DWORSHAK	Concrete Gravity	218.5	USA	1930	25	Narrow	Granodiiorite	TUNNEL (1) Horse Shoe 10.7 m × 16.4 m	70	After tunnel closure flow was allowed to pass through three low level outlets.
3.	GLEN CANYON	Arch Gravity	216.0	USA	4050	25	Narrow	Sand Stone	TUNNEL(2) Dia: 12.5 m	51	After closure of tunnels the flow was allowed to pass through low level outlets.
4	ITAIPU	Composite	196.0	Brazil	30000	100	Wide	Sand Stones Clay Beds	Diversion Channel Width: 100 m	160	A diversion control structure of 160 m height was located in the div. Channel containing 12 temporary sluices (6.7 m × 22 m) and closure gates.
5.	NEW BULLARDS BAR	Concrete Gravity Arch	193.0	USA	565	Designed for low season flow	Narrow	Amphibolites	TUNNEL(1) Horse Shoe 12.8 m × 14.6 m	61	After tunnel closure flood was allowed to pass over block No. 17 and through permanent low level outlet.
6.	MOSSYROCK	Arch Gravity	185.0	USA	1540	13	Narrow	Andesite & Basalt	TUNNEL (2) Gothic Shaped Size: 12 m × 10 m	-	After closure of tunnels flow was allowed to pass through low level outlets.
7.	REVEL STOKE	Concrete Gravity	175.0	Canada	2400	30	Narrow	Gneiss Quartzite Marble	TUNNEL(1) Dia: 13.1 m	38	After closure of diversion works flow was allowed to pass over low level blocks
8.	CABORA BASSA	Concrete Gravity Arch	158.0	S. Africa	4500	Designed for non-monsoon flow	Narrow	Granitoid Gneiss	TUNNEL (2) D-Shaped 16 m × 16 m	40	Both u/s and d/s cofferdams were designed to be overtopped.
9.	SRISAILAM	Masonry – Concrete Gravity	144.0	India	850	Designed for dry season flow	Narrow	Quartzite	(i) TUNNEL (1) 9.14 m Dia (ii) Div Channel Width: 15.3 m	18	Two semi-permanent concrete cofferdams were provided at both u/s and d/s.
10.	SUPA	Concrete Gravity	101.0	India	170	Designed for fair weather flow	Narrow	Calcareous Magnetite Quartzite	RCC Conduit 3 m × 4 m	21	On closure of conduit 2 Nos. of permanent river sluices of size 2.14 m × 3.2 m are provided.

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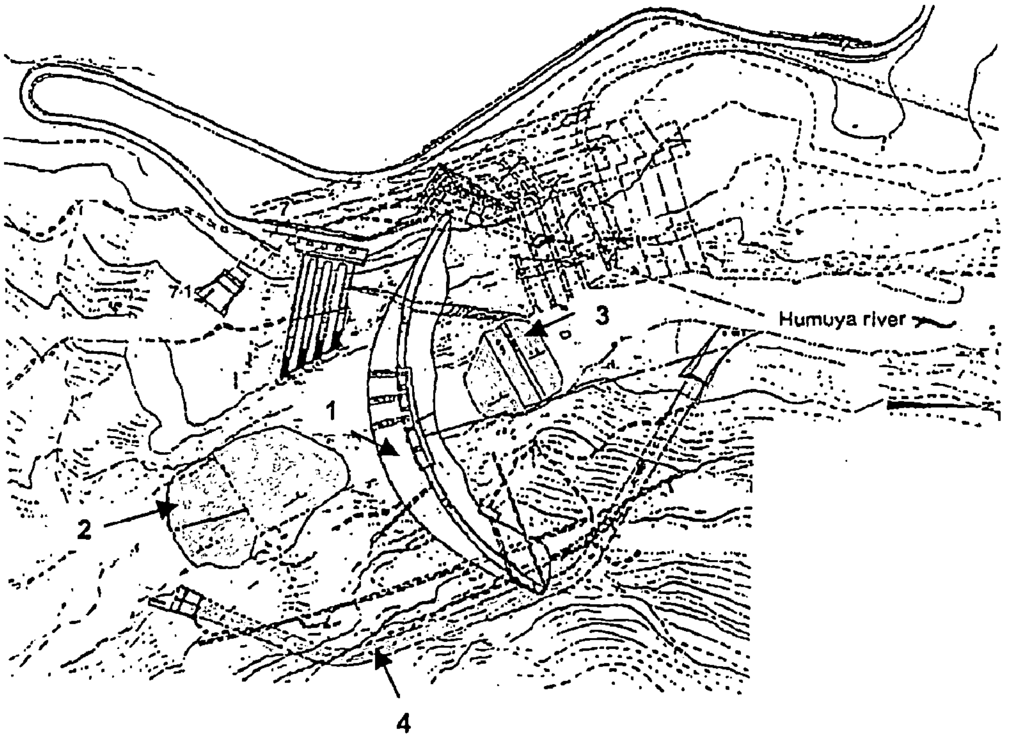

Fig 3.2.1: General Layout of EL Cajon dam

1. Arch dam; 2. Upstream cofferdam 3. Downstream cofferdam; 4. Diversion tunnel

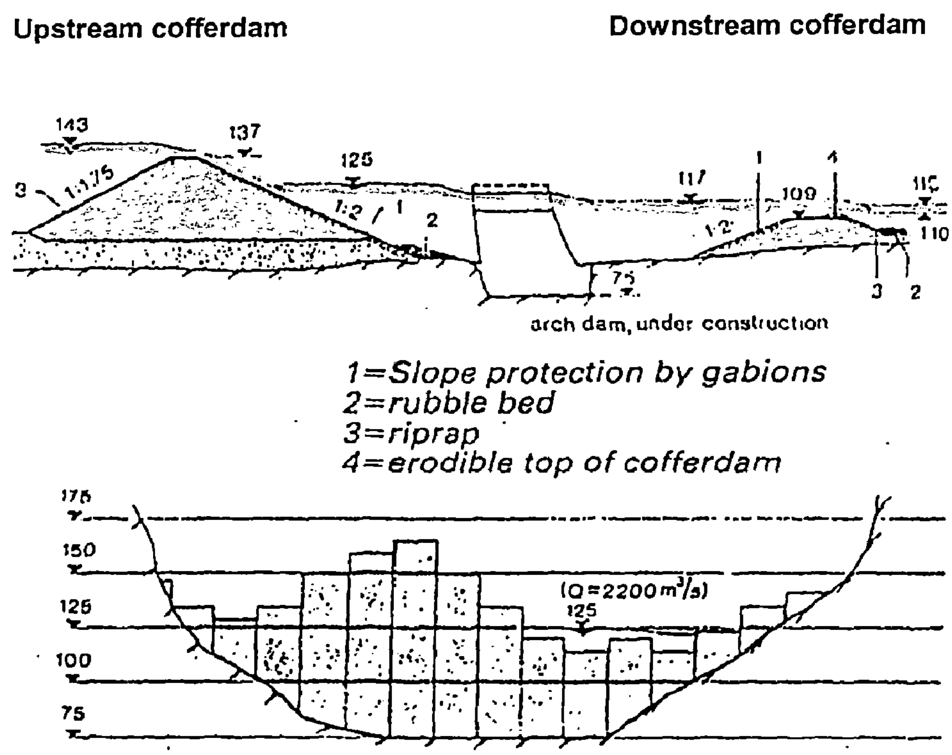

Fig. 3.2.2: EL Cajon dam – River diversion scheme

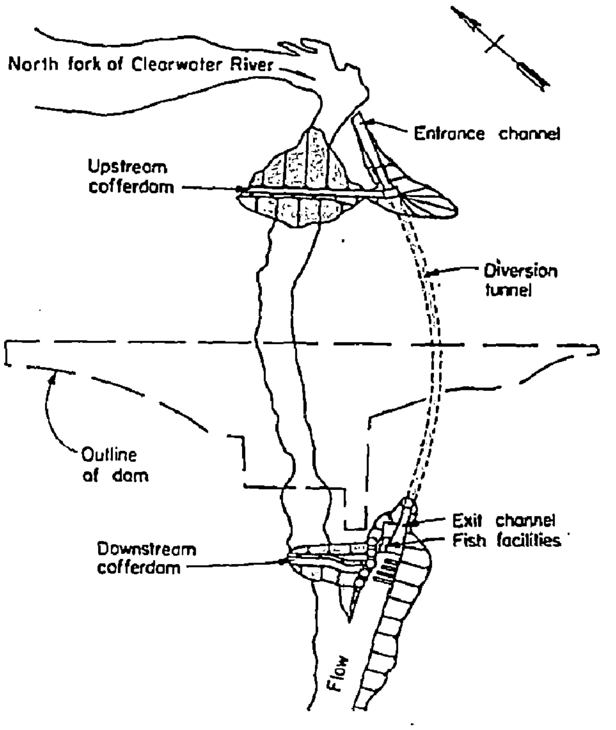

Fig.3.2.3: Dworshak dam-Genaral plan of diversion facilities

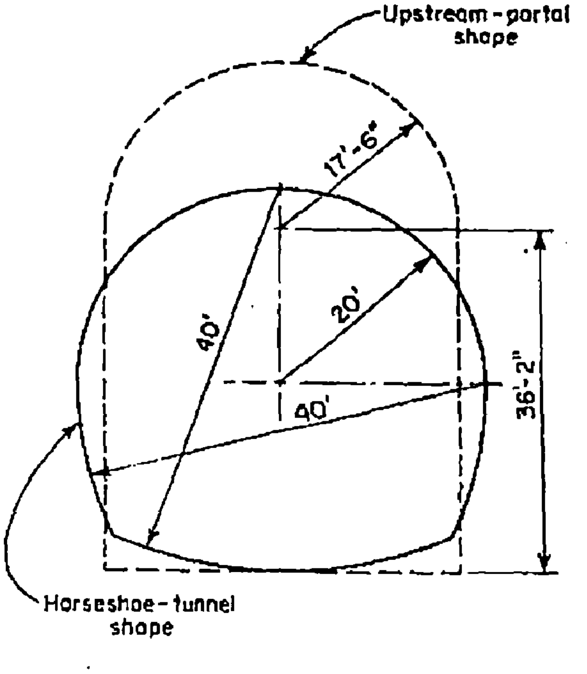

Fig.3.2.4: Dworshak dam-Cross section of tunnel

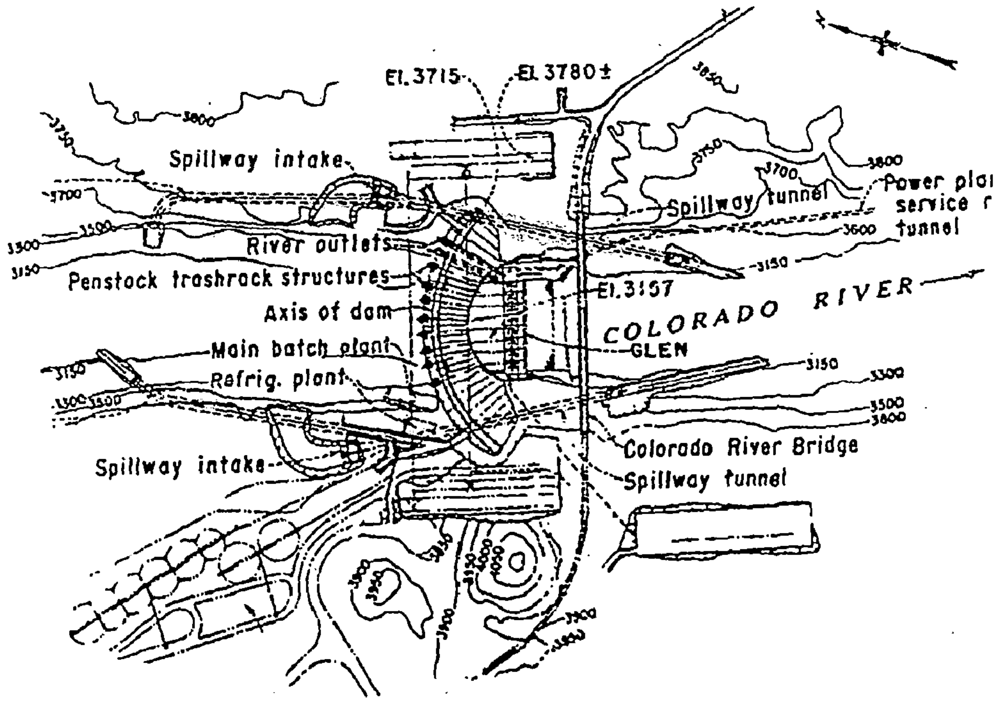

Fig.3.2.5: Glen Canyon dam and power plant

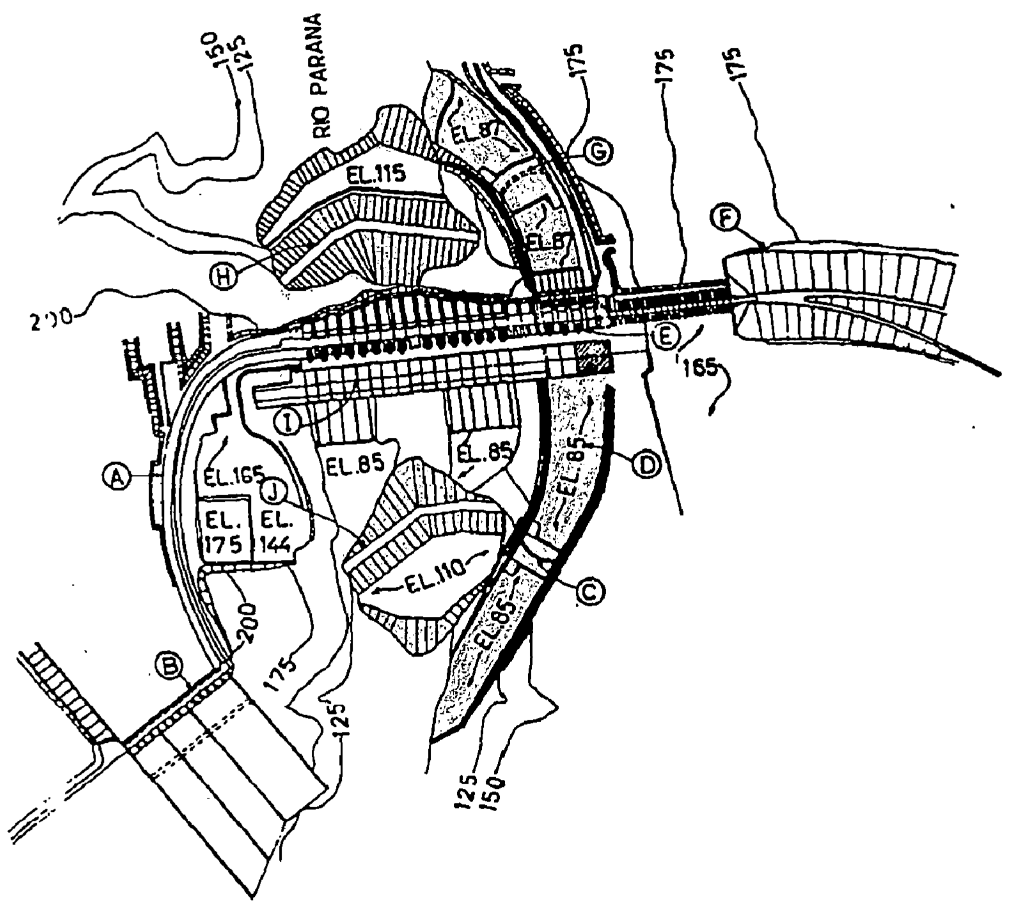

Fig.3.2.6: Layout of diversion works at Itaipu Project

A- right wing buttress dam; B- spillway; C-downstream arch cofferdam; D-diversion channel; E- diversion control structure; F- rock fill dam; G-upstream arch cofferdam; H- upstream main cofferdam; I- power house; J- downstream main cofferdam

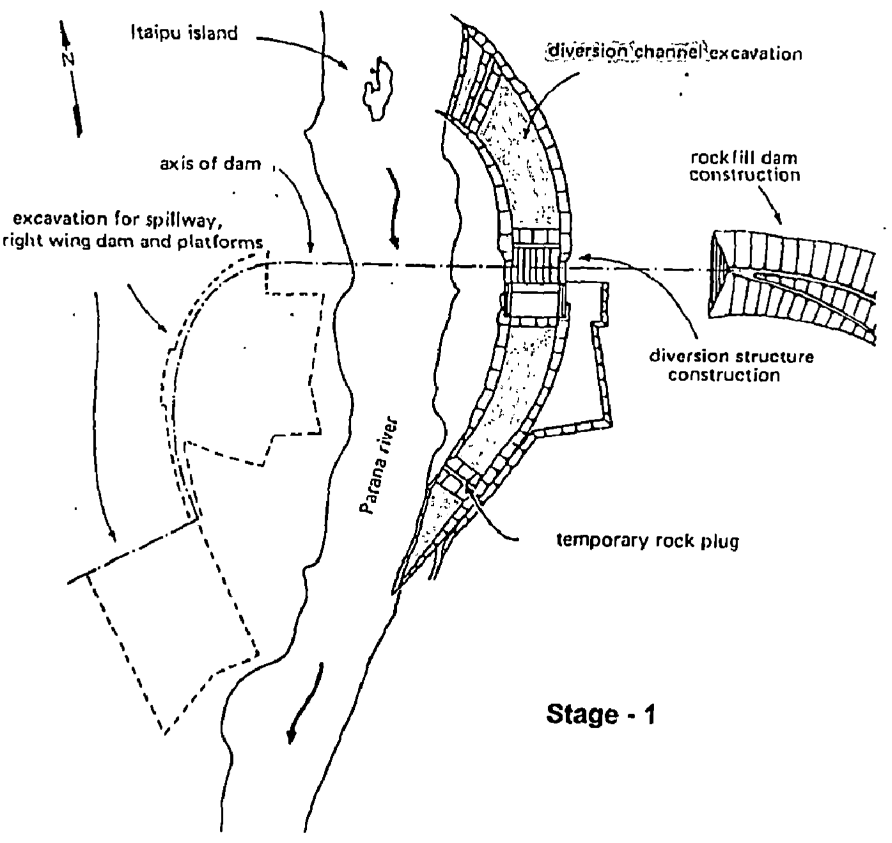

Fig.3.2.7: Itaipu dam – River diversion stage - 1

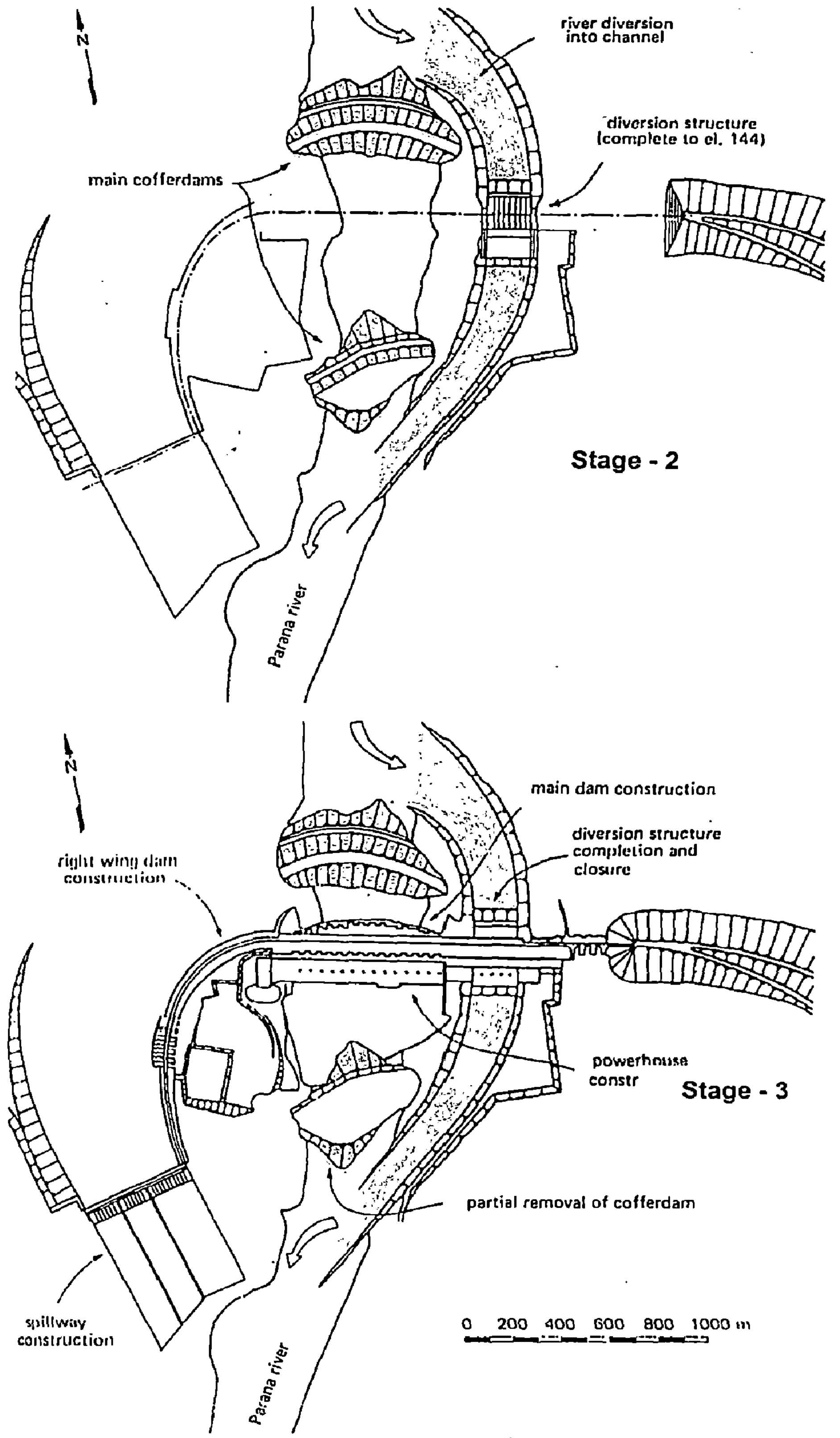

Fig.3.2.8: Itaipu dam – River diversion stages – 2 & 3

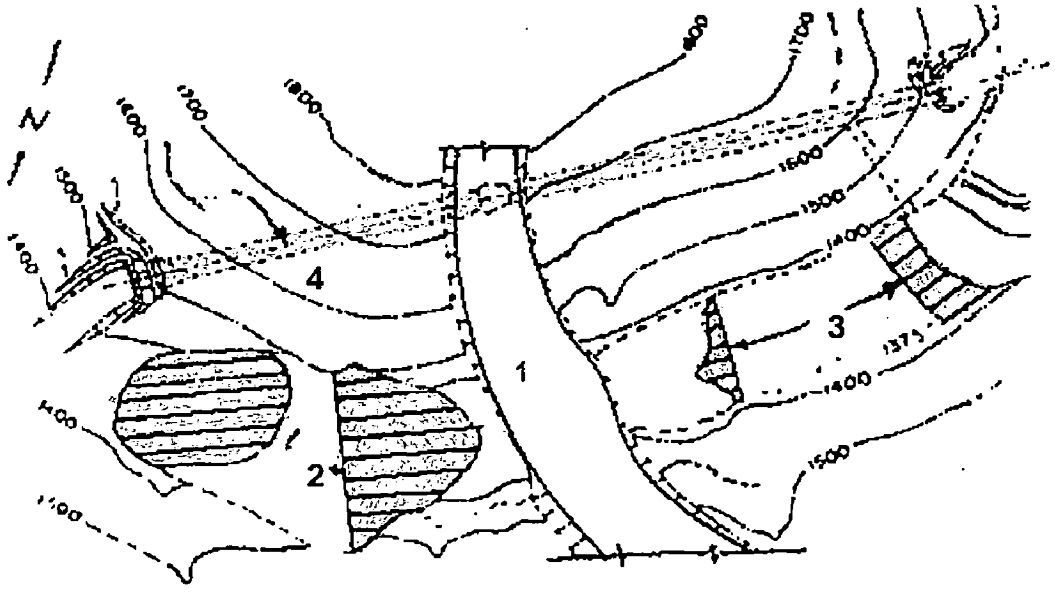

Fig.3.2.9: New Bullards bar dam – Layout of diversion tunnel and cofferdams

1. Top of main dam
2. Top of upstream cofferdam
3. Top of downstream cofferdam
4. Diversion tunnel

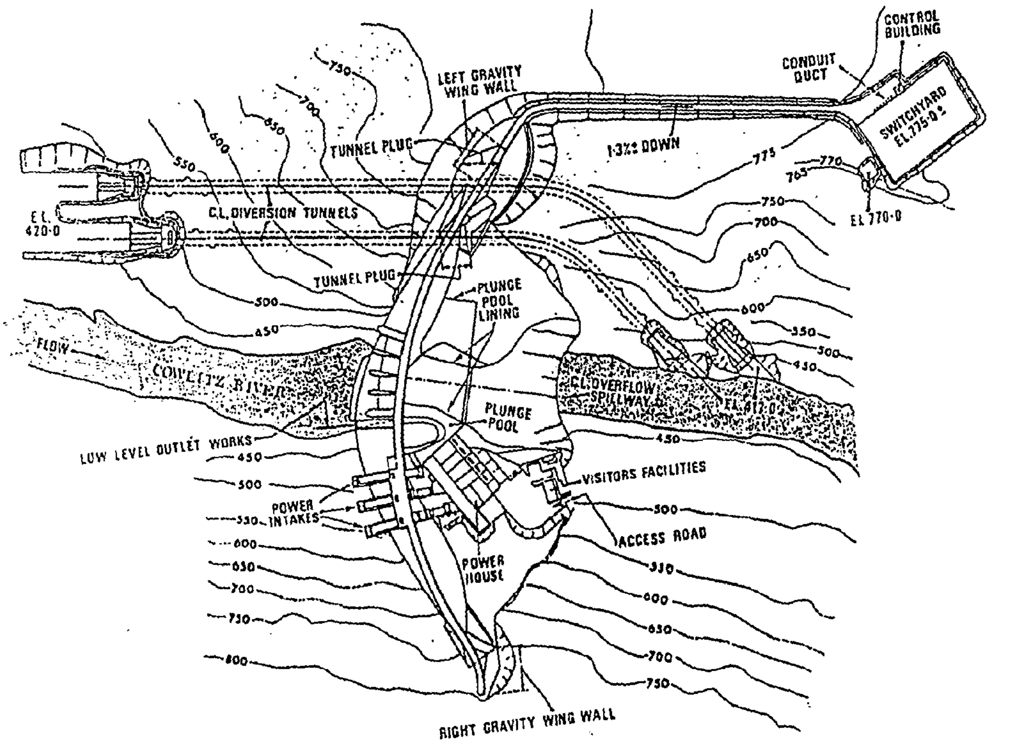

Fig.3.2.10: Mossyrock dam – General layout

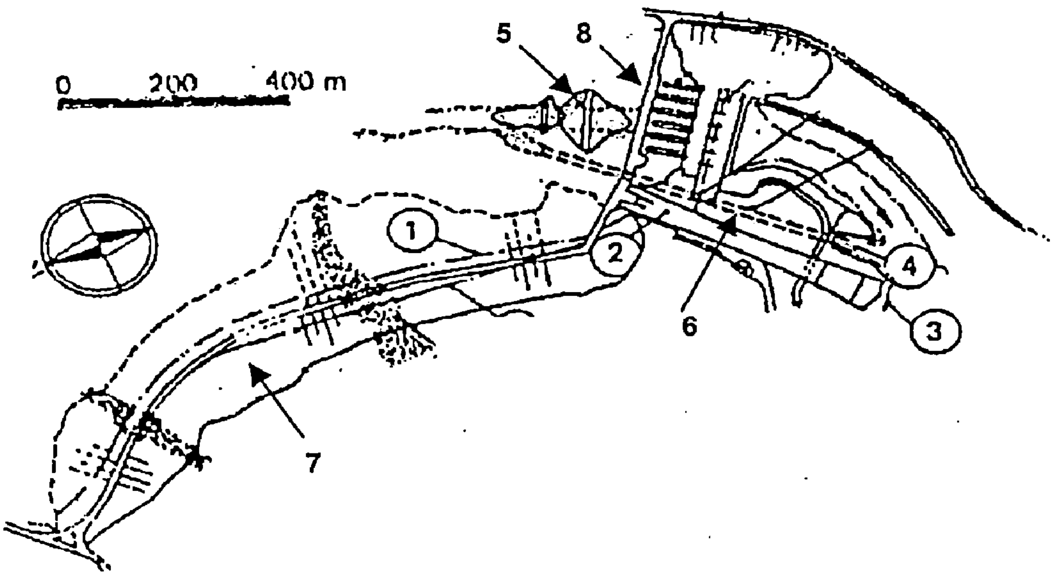

Fig. 3.2.11: General arrangement of Revelstoke

1. Reservoir shoreline	2. Spillway
3. Tail water shoreline	4. Diversion outlet
5. Upstream coffer dam	6. Diversion tunnel
7. Earth fill dam	8. Concrete dam

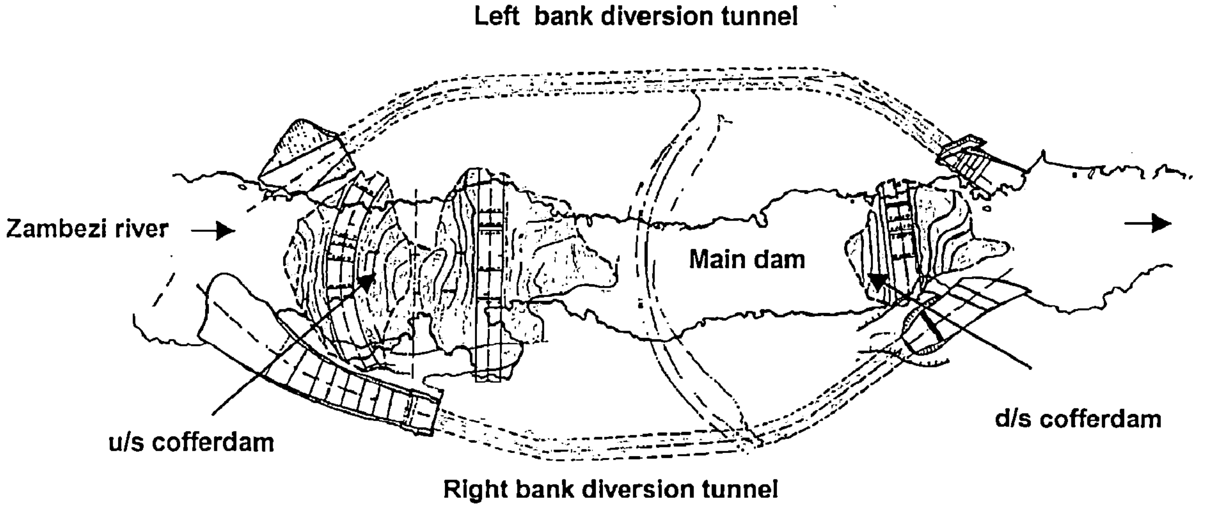

Fig. 3.2.12: Cabora Bassa – diversion scheme

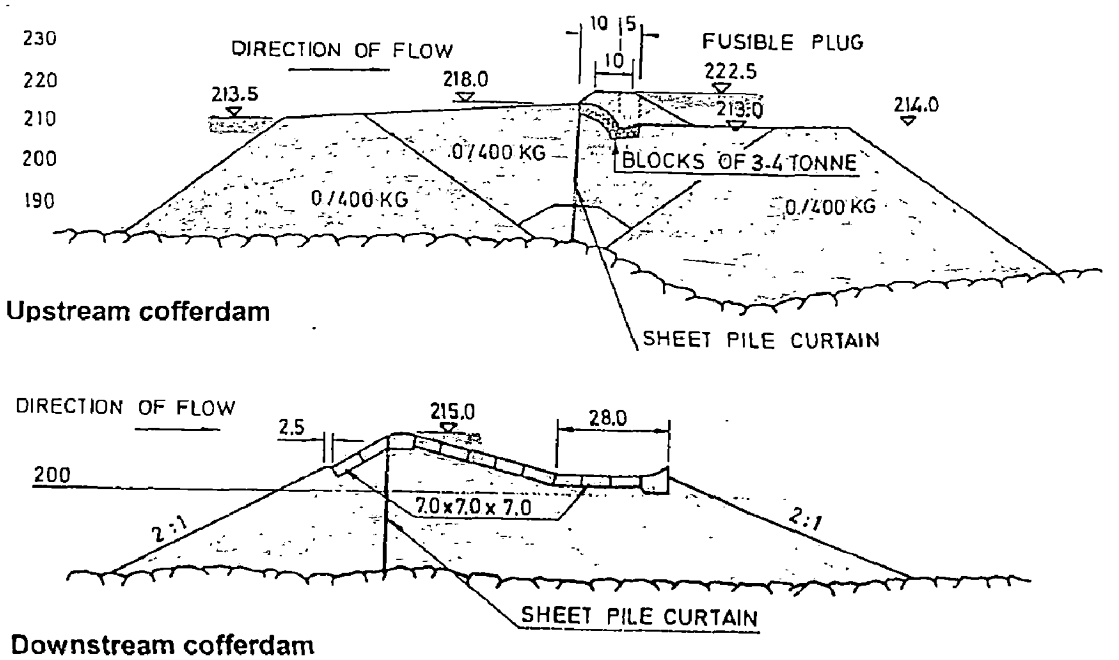

Fig. 3.2.13: Cabora Bassa – Cross section of upstream and downstream cofferdams

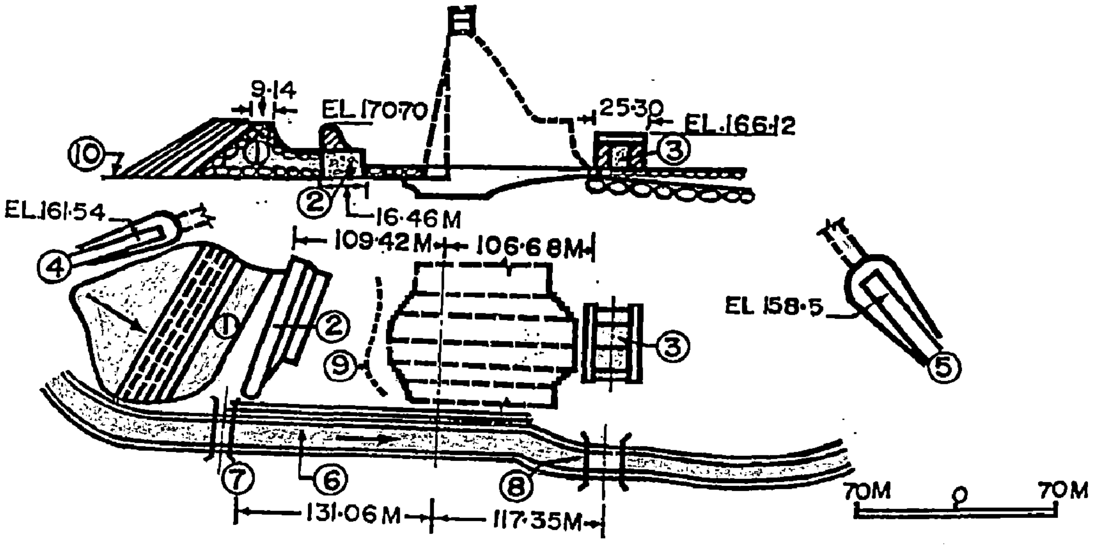

Fig. 3.2.14: Srisailam dam – Layout of diversion works

1. Upstream rock fill cofferdam
2. Upstream colcrete wall
3. Downstream cofferdam
4. Diversion tunnel entry
5. Diversion tunnel exit
6. Diversion channel
7. Upstream temporary bridge
8. Downstream temporary bridge
9. Edge of foundation pit
10. River bed

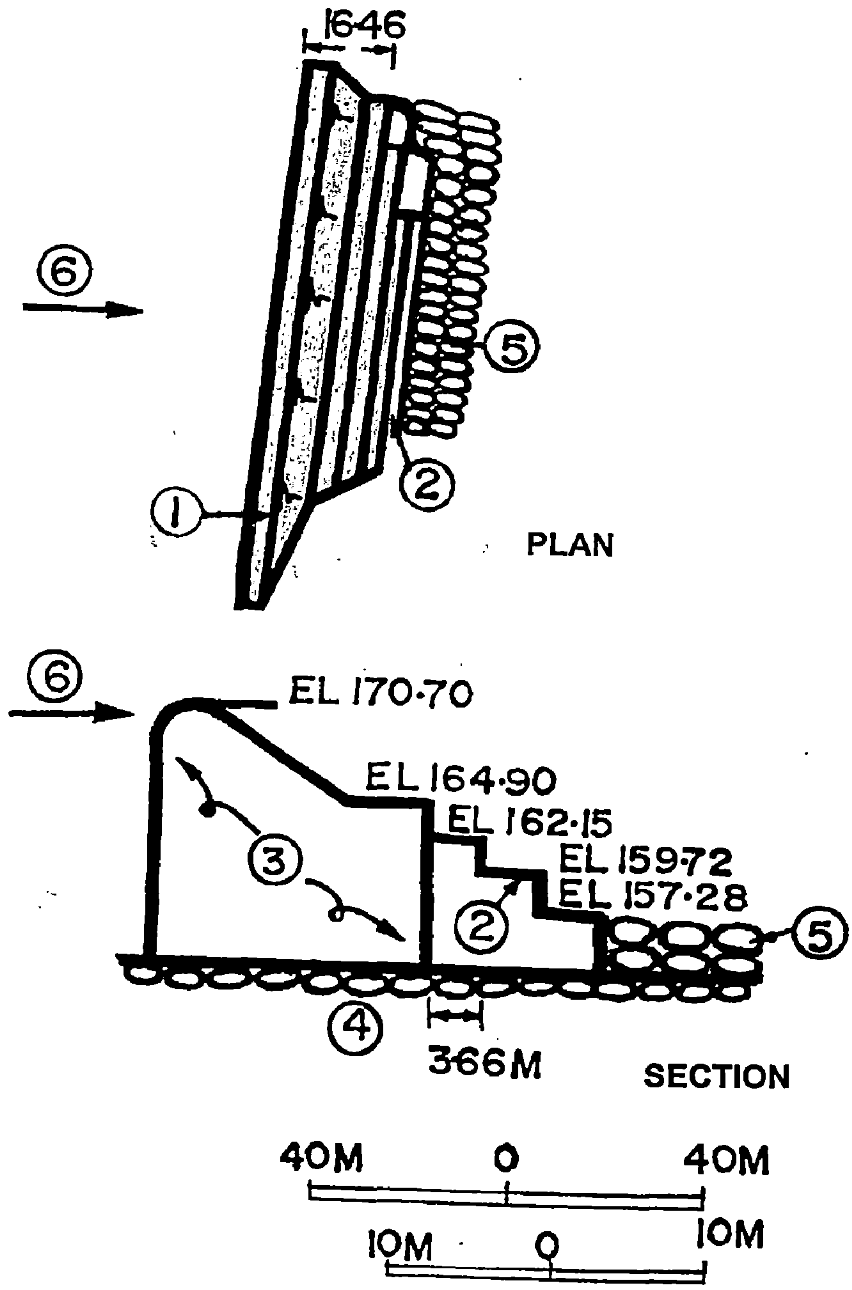

Fig. 3.2.15: Srisailam dam - Protection to the upstream cofferdam

1. Upstream cofferdam
2. Stepped concrete apron
3. Uptream colcrete cum masonry dam
4. Bouldery bed
5. Gabions
6. Flow

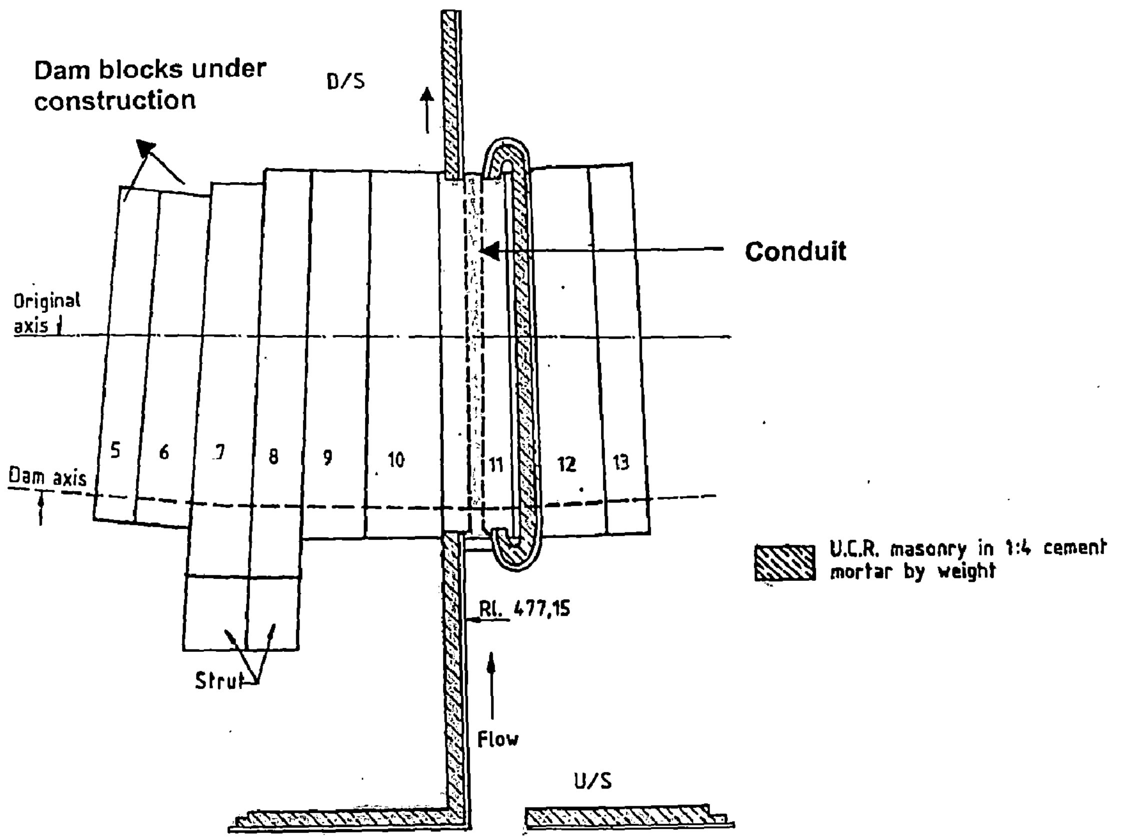

Fig. 3.2.16: Supa dam - Original diversion scheme

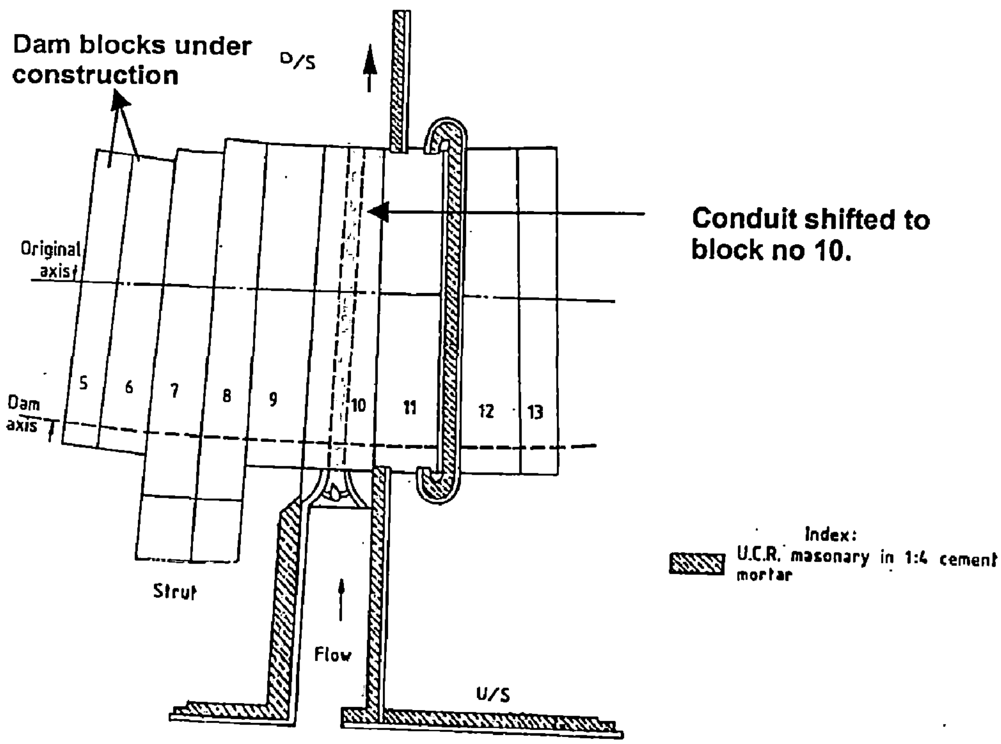

Fig. 3.2.17: Supa dam - Revised diversion scheme

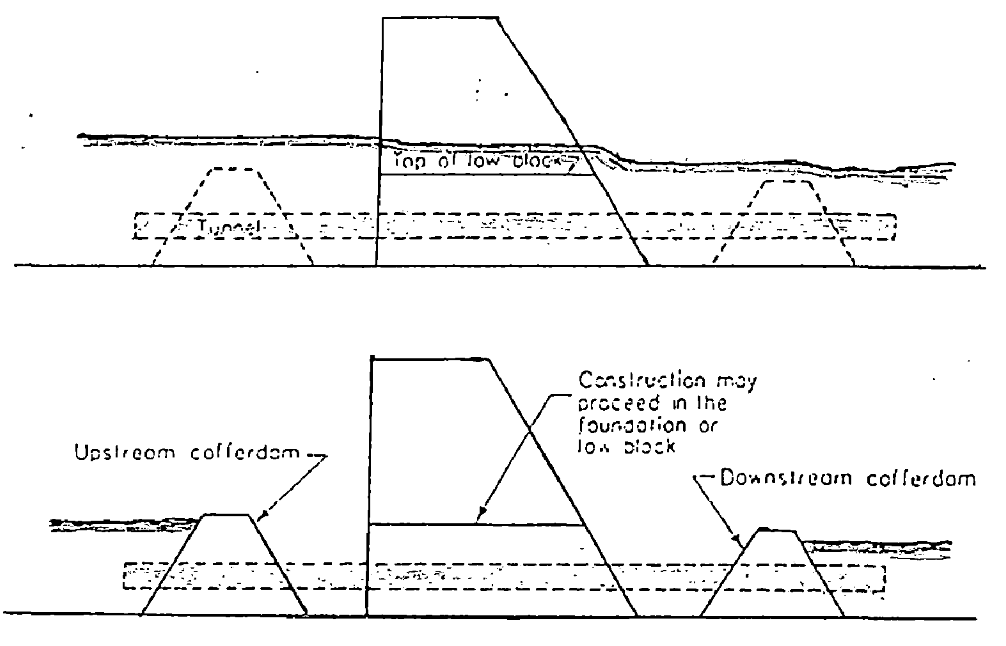

Fig. 3.3.1 Discharge during flood stages and low flow periods for a concrete dam

REVIEW OF DIVERSION ARRANGEMENTS ADOPTED FOR SOME EARTH/EARTH & ROCKFILL DAMS

4.1 General

For earth & rockfill.dams built on narrow river channels, the usual practice is to have one stage river closure and divert the river flow through the tunnels or through the culverts under the dams.

Concrete Dams can undergo overtopping during construction without much damage the overtopping of earth dams during construction, even with small rates of flow can destroy the part that has already been constructed and cause life losses downstream.

Therefore, the diversion arrangements should be designed and constructed so as not to allow overtopping. Design flood with return period ranging from 1 in 100 to 1 in 1000 or more in some cases are usually to be adopted for diversion. The best method for construction of an earth fill dam is to divert the natural river flow both for monsoon and non monsoon through temporary channel and execution of the main works is done all at once across the river without any hindrance throughout the time schedule.

But in wider valleys the diversion flows are likely to be too large to be economically carried in tunnels or conduits. A temporary channel involving a gap through the dam itself may be one of the options to divert that flows, while remaining part of the embankment is being constructed. Before the stream is diverted the foundation preparation required for the dam should be completed in the area where temporary opening will be left through the embankment. The stream is then channeled through this area, after which the foundation work completed. The portion of the embankment to either side of the diversion opening may then be completed.

The opening channel should conform to the following [USBR, 1976]

i. The side slopes should not be steeper than 4V : 1H to facilitate filling of the gap at the end of the construction period and to decrease the danger of cracking of the embankment due to differential settlement and also to provide a good bonding surface between the previously constructed embankment and the material to be placed.

ii. The bottom grade should be the same as the original streambed so that erosion in the channel will be minimized.

iii. Sufficient width of opening should be adopted so that it can handle the design flood. The width will also be governed by considerations of the equipment capabilities for filling the gap in the time available.

The period of closing the gap should be so selected that large floods are least likely to occur in this period. Construction equipment should be mobilized so that the gap can be filled as quickly as possible to an elevation, which will permit discharge of a flood through spillway. The average rate embankment placement must be such that the gap can be filled faster than the water rises in the reservoir. Care must be taken during the filling of the gap so that quality of work is not sacrificed to the exigency of the situation.

The sequence may be repeated although two or three stages may be sufficient in most cases. The portion of the cofferdams in contact with flowing water must be protected with heavy rockfill, gabions etc. and also the flow velocities are kept to reasonable values to avoid erosion problems.

4.2 Diversion arrangements for Earth/Earth & RockFill Dams

4.2.1 Oroville Dam

a. Project description

The Oroville dam of the Feather River Project is a major unit of the California water plan of the California department of water resources. Oroville dam located near the town of Oroville in Buttecounty, is a zoned rolled fill type dam about 235 m high with a crest length of 1697 m. It is the highest dam in the United States and the highest fill type dam in the world. The reservoir has a total capacity of 4298 Mm^3.

The drainage area above the dam site is predominantly steep mountainous terrain capable of producing flows ranging from a few hundred cumecs to over 5664 m^3/s within a short period of time. The left abutment of the dam consists an underground power plant, which has a total line capacity of 644 MW. And on the right abutment there is a combined spillway and outlet. General plan of works provided at Oroville Dam is shown in Fig. 4.2.1.

b. River diversion scheme

The diversion works were designed for the standard project flood of 12,460 m^3/s. This flood is produced by a 96-hour rainstorm with an average precipitation of 36.3 cm on the drainage area. Its frequency is 1 in 400 years.

The diversion of the river discharge during construction was achieved by two 10.7 m diameter concrete lined tunnels, each approximately 1370 m long, located on the left bank. The circular section was selected rather than a horseshoe because it provided distinct advantages both in construction and structural strength. The entrance to Tunnel 1 is a rectangular to circular bell mouth with a large center divider. The entrance of Tunnel 2 was meant to eliminate special coffer damming when the closure was required. The 126.5 m high upstream cofferdam formed part of the upstream face of the main dam.

Both the diversion tunnels have been plugged near their midpoints after the completion of the dam. Upstream portion of Tunnel 1 is abandoned and that of the other has been used to supply water to the river outlet works, which are meant to meet water requirement downstream during the power plant shutdown. However, both tunnels below their plugs are converted into tailraces for the underground power plant. The horizontal as well as vertical alignments of the tunnels have been dictated by this requirement.

4.2.2 Tarbela Dam

a. Project description

Tarbela dam on Indus River is the highest dam in Pakistan and comprises a 143 m high Earth and Rockfill dam for irrigation and power purpose with 2100 MW installed capacity. As and incidental benefit, it will afford a measure of flood control. The project works consist of the main embankment dam, two auxiliary dams on the left bank, a side valley and a saddle service and auxiliary spillways, four irrigation and power tunnels and power plant.

The Indus River has a seasonal flow pattern, peaking beyond 8400 m^3/s in mid summer from a combination of snow-melt and monsoon rains and tailing off in winter to much lower flows of the order of 400 m^3/s to 500 m^3/s.

b. River diversion scheme

The dam is located just downstream of confluence of the three tributaries of this river, consequently to divert the river flow into diversion conduit during construction of the main dam, upstream cofferdam should be built across these three streams. For considerations that overtopping during construction would not be allowed the diversion sequence and construction planning of this project was done in the three stages as described below.

The construction of the dam is planned in three stages, each separated from the previous stage by an operation to redirect the river and contain it. The diversion is divided accordingly: During stage I the diversion channel, cofferdam A and the buttress dam, the key to the redirection of the river flow from channel to the tunnels, are built, cofferdam C is started but the river remains in its natural channel (Fig. 4.2.2).

The diversion channel has a bottom width of 198 m throughout most of its length, 4.8 km long, and is designed to carry 21,500 m^3/s, 10-15% more than the estimated maximum daily discharge of 100 years record. At the downstream end, the sheet pile cellular cofferdam was built which constitutes the right wall of the diversion channel. The design was appropriate because of the limited space available for a cofferdam structure to protect the powerhouse area against a flood flow of 21,500 m^3/s in the diversion channel. The cofferdam consists of twenty 22.5 m diameter cells with connecting circular arc segments, allowing a 17 m wide major haul road to be located on top. The total weight of steel sheet piling utilized was 6.5 million Kg.

The buttress dam consists of twenty-seven 1.5 m thick buttresses between the two abutments, topped by a bridge and working platform, and twenty-eight 5.8 m-wide by 9.75 m high gates, stop logs, and an upstream face slab. The abutments are 212 m apart, the height of the dam above the diversion channel floor is 28 m, and the base width is about 46 m.

The four diversion tunnels have internal diameter of 13.73 m each concrete lined. Later tunnels nos. 1 and 2 would be converted to power tunnels, and nos. 3 and 4 for irrigation releases. These tunnels in pairs have 21.5 m difference in elevations, the first two with their lower intakes will pass some of 1700 m^3/sec, each, with head water at the top of the buttress dam, all four tunnels will pass flow of 4900 m^3/s.

In stage II the diversion channel is opened and the river passes through the buttress dam, the main upstream and downstream cofferdams are constructed and the four tunnels are completed (Fig. 4.2.3), Stage II continued for 3 years with the river confined to the diversion chanñels. Stage III (Fig. 4.2.4) begins in the sixth year of construction when the plug of unexcavated material in the tunnel approach is removed followed by permanent closure of all the 28 openings of the buttress dam whereby the entire river flow passes through the tunnels and the closure section of the embankment is constructed. Out of the four tunnels two (1 and 2) are to be finally used as power tunnels and the remaining two (3 and 4) as irrigation outlets.

4.2.3 Mangla Dam

a. Project description

Mangla Dam project, which will provide a major reservoir on the River Jhelum in west Pakistan, will enable high flows in the Jhelum to be stored and subsequently transferred east wards to meet the irrigation requirements. The dams were built on weakly cemented sandstones and clays of the siwalik formation at the point where the river flows out of the Himalayas foothills on to the alluvial plains of the Punjab.

The River Jhelum at Mangla has a catchment area of approximately 33700 sq. Km. Its minimum flow is about 170 m^3/s and its maximum recorded flow is estimated to have been approximately 31000 m^3/s. The river flows are characterized by the very large floods that have occurred during the monsoon season i.e. July to Sept. Thereafter river flows gradually subside. In late February, March and April snow melt in the mountains causes an increase in base flow and storms at this period have produced floods of up to 7000 m^3/s. There is normally a high base flow of 1700 m^3/s or more during May and June but recorded floods during these two months have lower peak flows than during the spring. From a maximum probable storm study carried out for the catchment, it was deduced that the conditions causing the great monsoon floods could exist from late in June. This deduction was proved correct when a flood of 23400 m^3/s occurred in July 1959.

b. River diversion scheme

To allow Mangla Dam to be built across the riverbed, temporary waterways had to be provided. In deciding their capacity it had to be borne in mind that overtopping of the partly completed dam would result in wide spread flooding downstream and very heavy damage. After careful consideration it was decided that diversion arrangements should be such that the maximum recorded flood at any time of the year could be safely discharged, due account being taken of the lag effect of available storage upstream of the dam.

From preliminary consideration it was concluded that it would be impracticable to build the full section of Mangla Dam across the river bed to elevation required during the six months between the end of a monsoon and the onset of the spring floods. It was necessary to concentrate instead on constructing a "closure dam" on the upstream side of the core of the main dam.

For any scheme for river diversion and reservoir impounding, the requirements were as follows:

i. The works must at all times be safe against floods equal in severity to the maximum recorded flood for that period of the year;

ii. Irrigation supplies to the Upper Jhelum Canal and to other canals supplied by the Jhelum river must be maintained at all times, except for very restricted periods;

iii. The works must be so designed that construction could be programmed to allow water to be impounded to conservation level by and two of the 100 MW turbo-alternator sets would be ready for operation.

After considering many alternative schemes, an economic solution to the problem of diverting the river was developed (Fig. 4.2.5). The method finally adopted involved diverting the river through five 9.15 m dia and 580 m long tunnels constructed approximately normal to the centerline of the intake dam on the left bank and providing storage in the reservoir to absorb the peak flows of incoming floods by constructing a closure dam across the river to a height of 64 m above river bed. The 2 m free board to the crest of the closure dam was considered adequate. The choice of tunnel layout was greatly influenced by the necessity to provide tunnels for the turbo alternator sets and the irrigation outlets of the power station. This advantage was decisive in locating the hydroelectric scheme on the left bank.

It was proposed that the river should be diverted into the tunnels by means of a cofferdam during the latter part of September or early October when the river discharge is normally relatively low. This was to be immediately followed by the extension of the closure dam across the river bed, which not only had to be completed to full height 64 m above river bed level before the onslaught of the monsoon the following July but also had to rise fast enough not to be overtopped by winter floods.

4.2.4 Pong Dam

a. Project description

The Pong Dam located in the Himalayan foot-hills in district Kangra of Himachal Pradesh, on the River Beas is the highest earth core-cum-gravel shell dam so far constructed in the country. It has a central impervious core with sand and gravel shell zones on either side. It is 132.6 m high from the deepest foundation level and about 100.6 m high above the river bed. The length at crest level is 1950.7 m and width at the top 13.7 m except at special sections where it is more. The overall base width at deepest river bed level (excluding toe weights) is 610 m. About 35 million cu m of fill has been used in the construction of embankment.

b. River diversion scheme

i. Diversion tunnels

After careful consideration, it was decided that design flood for diversion stage should have the probability of 1 in 1,000 years with a peak discharge of 22940 m^3/s and volume of 3.950 million cu. m. In view of the fact that the diversion flood is much more than the maximum recorded flood the chance of any floods greater than design diversion flood is very small and was accepted as construction risk. For diversion of river Beas during construction of dam, 5 tunnels each of 9.14 m finished diameter have been provided, two of these tunnels (T1&T2) will latter be permanently used as outlet tunnels for irrigation purposes and the other three (P1, P2&P3) as power tunnels. The tunnels had an aggregate length of 4,774 m. Fig. 4.2.6 shows the general layout of dam and appurtenant works.

The number of diversion tunnels was fixed primarily on the basis of rate of fill placement for the dam and flood routing studies based on actual flow of wettest year on record (1942) in conjunction with design flood with a frequency of 1 in 1000 years.

The lining of tunnels with concrete was fully mechanized. The concrete was pumped behind the steel forms with the aid of pump-cretes and pipelines. Special steel gantry forms were designed and fabricated for this purpose. The diversion works also involved construction of riverbed entry for each tunnel, stilling basins, exit channels, and permanent intakes for ultimate utilization of the tunnels. All these were completed by the end of 1969, when the flows of River Beas were diverted through the tunnels.

ii. Cofferdams

In order to divert the river flows for the construction of the dam and its appurtenant structures, coffer dams were constructed of random fill material, both upstream and downstream outlet and penstock tunnels and dam. For construction of spillway, stilling basin and power plant, cofferdams were constructed on downstream side to block river entry. The wrest levels of all such cofferdams were kept on the basis of observed highest flood level at the site with provisions of sufficient free board.

iv. Diversion schedule

Diversion schedule of river was linked with the raising of dam in stages to a safe height to avoid over-topping. As per programme based on available resources, the dam was envisaged to be completed in five stages, viz., 0 (zero) or pre-diversion stage, and I, II, III, IV stages; each to be completed in one working season after diversion. However, during actual execution, adverse geological features were encountered in the foundations necessitating extra quantum of work and a little less progress than anticipated. The construction stages were recast and the dam was completed in six stages against five planned initially.

After completion of the first stage, the monsoon flows of River Beas were passed through five tunnels and also partly completed dam. In order to minimize the severity of the flow conditions and extent of scour of the fill, the top level of fill placement for the first stage was kept at the natural river bed elevation. An apron consisting of boulders in wire crates was provided at the downstream end of fill placed to act as downstream toe protection. After monsoons, it was noticed that but for some pot-holes on the left side, the partly completed dam section stood the on-slaught of floods well.

4.2.5 Ramganga Dam

a. Project description

Ramganga River project is situated about 3 km upstream of Kalagarh village in Pauri Garhwal district of Uttar Pradesh, India. The project essentially comprises an earth and rock fill dam 127.5 m high and 630 m long at crest, known as main dam. Another 71.35 m high and 562 m long dam known as Saddle Dam has been constructed for plugging a low saddle on the left rim of the reservoir.

The project is primarily intended to provide irrigation benefits. A power house has also been constructed at the toe of Main dam. The installed capacity of the power house is 198 MW (3 x 66).

Ramganga River is not snowfed. The river catchment has an average precipitation of 1552 mm. The annual precipitation varies from 652 mm to 2436 mm. Catchment area of the river is 3134 Km^2 above the dam. Keeping in view the magnitude of the risk involved during construction period of 3 to 4 years 1 in 1000 year flood of 9708 m^3/s was adopted as diversion flood.

b. River diversion scheme

As the river at dam site is in a narrow gorge, diversion was not considered feasible through a part of the dam site. More over the construction of the dam required 3 to 4 years. Therefore, both monsoon and non-monsoon flows were required to be diverted for the duration of construction of the dam. Tunnels were, therefore, the only logical choice for diverting the flow. The size and number of tunnels will govern on the height of the first stage dam. It was decided to compute the height of the first stage dam on the basis of 1000 years flood frequency. The 1000-year flood with a base flow of 626 m^3/s when routed through the diversion tunnels raised the pond level to 337.6 m. keeping a free board of 2.56 m, the top elevation of first stage dam was fixed at 340.16 m i.e. at a height of 77 m from the river bed level.

The river takes a double right angled turn at the site of construction. The topography, therefore, warranted the right flank as the best choice to locate the tunnels. Layout of the diversion tunnels has been shown in Fig. 4.2.7. The chute spillway was also planned to be located on the right flank of the main dam. It was, therefore, economical to have the layout of the diversion tunnels in such a manner that they discharge in the stilling basin of chute spillway. It was therefore essential and economical to construct the stilling basin prior to the construction of the spillway.

Three different schemes were examined for diversion of flow during construction period, and finally the diversion was required to be made through two circular tunnels of 9.45 m diameter each. The section is of the tunnels has been kept circular for structural reasons.

To divert the non-monsoon river flow into the tunnels and to construct the first stage dam, a cofferdam was required. The cofferdam was designed to withstand a non-monsoon flood of 1 in 6 years frequency. Huge quantities of earth, mixed with boulder stripped from the main dam site and riverbed gravel were used for making the cofferdam.

Tunnel – I has been designed for a maximum discharge of 1656 m^3/s with a velocity of 23.3m /s and tunnel II for 1586 m^3/s with a velocity of 22.6 m/s.

With a flood frequency of 1 in 1000 years, a reservoir level of 337.6 m would have been attained. The first stage dam was, therefore, required to be constructed to this height with a suitable free board. Under this scheme, the alignment of the tunnels was kept in such manner that Tunnel-I (Eastern tunnel) was as close to the power house as possible with a separate stilling basin and Tunnel-II (Western tunnel) discharged in the stilling basin of chute spillway. Tunnel-I is straight where as Tunnel-II has a bend at about 30 m from the outlet end and the bend is 7^0 41' which has a radius equal to five tunnel diameter. The inlets of the two tunnels were kept close to each other. After the tunnels had served the diversion requirement, Tunnel-I was converted into power tunnel by plugging it near the inlet and Tunnel-II into irrigation outlets.

Construction of diversion works and the main dam in Ramganga project were done successfully with the above diversion arrangement. The first stage dam is part of the main dam.

4.2.6 Brownlee Dam

a. Project description

Brownlee hydroelectric project is situated in the upstream end of the Snake River gorge about 70 miles from Weiser, Idaho. The Earth and rock fill dam has a crest length of 425 m and a maximum height as measured from bedrock to the top of 121.95 m. In section the dam has a sloping impervious core flanked by sand and gravel filter zones and outer zones of rock. General plan of the project is shown in Fig. 4.2.8.

At the dam site the low water season usually starts in late June or early July and five to eight months. During this season, the flow varies form 227 m^3 /S to 566 m3/s and seldom exceeds 680 m^3/s. Normal high flood flows usually occurring in April or June with peak discharges from 1416 m^3/s to 1982m^3/s.

b. River diversion scheme

A wide berm of height 21m was constructed infront of the dam with material excavated from the core trench and with waste material from other excavations. This berm has a dual purpose to serve as a cofferdam, and to stabilize the foundation upon which the upstream rockfill is resting. Careful studies indicated that with a relatively small risk diversion could be accomplished by constructing only one tunnel of 758 m length with 11.5 m by 12.7 m modified horse shoe section through the right abutment and allowing flood waters in excess of the capacity of the tunnel to flow over the partially completed fill.

During dry season the flow was diverted through the tunnel after completion of the upstream cofferdam closure. But during wet season intense rainfall on the barren hills immediately upstream from the site caused a flood peak to occur at the site very suddenly.

Before water reached the top of the upstream cofferdam the cofferdam was opened and water allowed to-overflow the fill through a gap about 76 m wide in the central portion. The flood reached a peak of 1982 m^3/s and an estimated 1133 m^3/s to 1416 m^3/s flowed over the fill.

During the subsequent dry season the river flow had receded to 566 m^3/s and the upstream cofferdam was again closed. Cleanup and inspection of the core zone revealed that the diversion scheme had been sound. There was no apparent loss of rock in the main rockfill zones. The original compacted impervious core had not been appreciably disturbed by the river waters only an insignificant amount of original core had to be removed before construction could be resumed. Diversion over the partially completed rockfill dam was considered to be a complete success.

4.2.7 High Aswan Dam

a. Project description

The High Aswan Dam, 110 m high, is located on the Nile River near Aswan in the U.A.R. It has grout curtain, which is the deepest and largest in the world. The climate of the region is warm and dry along most of the year but during summer it is very hot and dry.

The flow of the Nile fluctuates extremely between drought and flood. It is generally full flood at Aswan during August and September due to the rainy season on the Abyssinean Plateau. The maximum flood with the highest daily peak of 14,000 m^3/s occurred in 1878 and the maximum ten days average discharge was found to be 13,200 m^3/s.

The geology of the dam site is a complex one. The right bank of the river contains granites, metamorphic shales and gneiss igneous rocks. Rockfill sluiced with sand, compacted sandfill and clay core make the major zones of the section. The underwater construction of rock and sand zones is a notable feature of this dam.

b. River diversion scheme

The dam is located in the reservoir area of the old Aswan dam and the valley at the site is very wide. Existence of a low ravine in the right abutment and the need to quarry a substantial volume of rock for the dam fill led to the choice of an open canal for diversion of the river flow.

As a flood protection measure, a maximum discharge of 11,000 m^3/s only was permitted to flow downstream of Aswan and the excess, if any, was to be stored in the existing reservoir. The diversion discharge was, therefore, kept as 11,000 m^3/s.

Upstream and downstream cofferdams and the open canal with six tunnels made the diversion arrangement. The purpose of the two cofferdams was only to provide still water zone for the main body of the dam to be constructed between them.

The final dam incorporated both the cofferdams. In addition to their use as diversion structures, the canal and tunnels now deliver water to the turbines and also pass flood through the emergency spillways of the hydroelectric station. The construction of upstream cofferdam was started leaving a 120-m wide gap (closure section) along the west bank. This gap was subsequently closed when the diversion through the channel took place.

4.2.8 Ord River Dam

a. Project description

The Ord River Dam is the major engineering feature in the development of the Ord Irrigation Project in the north of the State of Western Australia. The Ord River drains a catchment area of 46,000 Sq. km and has a maximum probable flood of 71,000 m^3/s. The dam was the rockfill type with thin sloping clay core of 99 m high above the lowest foundation, and 335 m long at the crest of the dam. The reservoir formed by the dam has an active storage of 5278 Mm^3.

At the dam site, there are two distinct seasons, a wet season and a dry season. The wet season extends from about December to April and the rains of this season occur somewhat irregularly from monsoons and the occasional tropical cyclone.

b. River diversion scheme

Construction of the embankment required two dry seasons, which necessitated the diversion of one wet season's river flow. Conventional diversion works were clearly unpractical and so the embankment had to be designed to pass one wet season's flow over the partly completed structure. A 4.43 m diameter diversion tunnel was constructed in the right abutment to deal with most of the dry season flows. The capacity of the diversion tunnel with an upstream water level 6 m above normal streambed level, was 70 m^3/s.

Hydraulic model studies were carried out to determine stable slopes, degree and type of protection, and the height to which the first season's rockfill could be constructed so as to remain stable when overtopped.

These model studies showed that the partly completed embankment would be stable if built approximately to the original streambed level with the top and downstream face of the structure protected by a 1.8 m thick layer of 1 m rock. The model was run at various tail water levels to ensure that the downstream face would be stable under the most severe conditions. At low flows, the stability of the rockfill on the downstream face was critical due to turbulence, whereas at high flows the structure was completely drowned and the critical feature was the high velocities across the top surface of the embankment.

During the construction period after the tunnel has been completed the small dry season flow was diverted through it, the foundation preparation and placing of the initial fill was carried out. When the structure had been built to streambed level, the 1.8 m layer of rock armouring was placed over the completed top surface and downstream slope of the embankment. Over the core area, the armoring was underlain by a 1 m thick layer of filter material as a building zone. Large rocks were individually placed by bulldozer, in contact one with the other, and spaces remaining were filled with smaller stones.

As an added safety measure, the downstream slope was protected with a steel grid of 25.4 m diameter bars on a 1.3 m x 0.45 m spacing, securely anchored in the fill and the foundation, with a 152 mm x 152 mm mesh behind the grid (Fig. 4.2.9). Special care was taken at the downstream crest were selected large rock was placed for 10 m along the surface of the embankment. The maximum depth of flow recorded was 10.5 m over the embankment. A flow of some 5,600 m^3/s was passed successfully over the structure during construction.

4.2.9 Balimela Dam

a. Project description

The Balimela Project on the Sileru River in the state of Orissa in India involves the construction of a 70 m high earth dam and 3 earthen dykes on the saddles in the left abutment hill. The spillway is located on a saddle to the right. The stored water is to be utilized to an extent of 50 percent by diverting to another valley and thus creating a drop of 275 m for power generation. The other half is to be used in power generation at 3 successive power stations on the sileru river it self.

b. River diversion scheme

The diversion arrangement was decided to handle only the dry water flow with the provision that the floods during monsoon season may overtop the partial embankment dam. This was necessitated due to the following considerations that:

i. The quantum of work to be done in one working season was considered much more than could be handled by the available construction facilities at site.

ii. To build the diversion arrangements with sufficient capacity for diverting monsoon flood was not economical, compared with the cost of the main dam.

iii. The site of the dam lies in the backwater of an existing dam on the Sileru river 16 km downstream, the minimum water depth at site being 11 m and rising to 21 m during floods. So the lower dam could be relied on to control the overflow during floods.

The diversion arrangements and construction sequence of the works that had been done with the following steps (Fig. 4.2.10).

To isolate the working area corresponding to the base of the earth dam a diversion tunnel of 45 m^2 area with design discharge of 280 m^3/s was excavated through the right abutment hill, with open connecting channels on the upstream and downstream. These channels were provided with cofferdams of natural ground and compacted earth fill for avoiding flooding of the tunnel during construction. While this work was in progress, earth was placed on the two.abutments, leaving the river portion and providing longitudinal slope of 1:3.

The construction of the main dam was decided to be split into four working seasons. In the first non-monsoon season the cofferdams in the channel were excavated and cofferdams were made in the river to divert water through the tunnel. Both the upstream and downstream cofferdams were constructed by dozing rockfill and earth mixture from the two ends. The water level was lowered in the final stages by reducing the pool level by about 1.2 m. The tunnel thus by passed the river flow through out the non-monsoon period and took care of 280 m^3/s flood with a free board on the upstream coffer dam of 2 m. This discharge corresponds to 1 in 20 non-monsoon flood at site.

The river cofferdams were initially planned to be a part of the dam section, but were later shifted about 300 m away from the dam with the following in view :

i. The material to be used in cofferdam was tunnel muck which had more than 50% of finer material and was expected to give flatter slopes by under water dumping. This material was readily available from the tunnel excavation and proved the most economical use.

ii. The area between the cofferdam and the dam base could be used as dump area for the overburden on rock which had to be excavated at a fast rate.

iii. Seepage into the placement area was very much reduced due to longer distance of the water pool from the working areas.

iv. Easier approach to lower levels was possible due to flatter gradients on haul roads.

v. The upstream location of the cofferdam would also save from submergence a construction bridge across the river thus avoiding dislocation of traffic between the two banks.

vi. Shifting of the downstream cofferdam lower down also assured a pool downstream of the dam section and shifting of the erosion to the coffer dam instead of the dam fill.

After the completion of the dam to EL. 408.5, the upstream cofferdam was lowered to EL. 411.48 and the cofferdam to EL. 408.50. Water was allowed to rise to a level of 416.04 by controlling the overflow at the lower dam, thus flooding the entire area of dam and cofferdams and bringing the flow back to river section. Water level EL of 416.04 was thereafter maintained during floods. A maximum flood 480 m^3/s was actually observed at site and the flow conditions remained without excessive turbulence at any location. The inspection of the fill after the monsoon, has shown hardly any erosion of the fill and the material of the dam has proved to be erosion resistant.

The overflow on the partly constructed Balimela Dam has shown that the compacted clayey material has ample erosion resistance. Although high velocities were not encountered due to low floods and cushion of water above the fill, the absence of erosion on the compacted fill is of importance in planning river diversion during construction.

4.2.10 Ukai Dam

a. Project description

Ukai Dam in Gujarat state, India forms the headworks of a large multi-purpose project envisaging benefits of irrigation to an area of nearly one million acres, generation of a block of 110 MW of power and flood control benefits. The dam is located near village Ukai at a distance of 100 km east of Surat. The dam is 4926.75 m long and is of a composite earth cum masonry type. The maximum height of earth dam is 68.58m and that of masonry dam is 64.92 m. The dam will impound a lake having gross storage capacity of 851 x 10^3 ha.m with a waterspread of 510 Km^2. The earth dam needed handling of 23.5 million m^3 of earth, which is the largest quantity laid for any of the completed projects in India.

b. River Diversion Scheme

Earth work was considered too large to be tackled in one season of construction consisting of 8 months. Considering magnitude of work involved and sequence of excavation of cut off trench (which had foundation nearly 21.3 m below water level in the river) and of the building up of the section of the earth dam and the overall feasibility of preparing dam, it was decided that the laying of earth work should be spread over a minimum period of 3 working seasons. Therefore, the construction planning had to be so oriented that in the two intervening monsoons the river was either suitably diverted or allowed to pass over the incomplete earth dam. The alternative of negotiating the monsoon floods over partly completed earth dam was ruled out at ukai because of the exceptionally heavy floods that could be expected as also very high fall of nearly 45 m. The flood peak to be catered for in the design of the diversion works was as high as 45,000 m^3/s. No protective works over the earthen dam, howsoever, strong, would withstand the enormous velocities that would be generated. The river, therefore, has to be suitably diverted such that the earth dam was at no time overtopped during its construction and that the earth dam was safe from high velocities created during floods.

Ukai dam site is not located in a mountaineous narrow gorge having steeply rising rock cliffs on either flank of the river. The dam site was overlain by overburden on either flanks, and in the river channel proper, sandy and gravelly deposits formed a thick mantle over the bed rock. Thus from geological, topographical or hydrological considerations, tunnels for diverting such huge floods were ruled out. Excavation of the diversion channel was the only alternative since depth of alluvial soil had to be removed form the left terrace to reach hard bed capable of providing non-erodible strata. Normally, during construction stage, flood corresponding to 75 to 80% of the design flood is assumed. However in case of Ukai if higher floods should occur, the earth dam in the river gorge would get overtopped, wiping out the entire civilization including the city of Surat. It was therefore decided that the dam should be designed to negotiate the flood of 49,500 m^3/s during all stages of its construction.

i. Evolution of dimensions of diversion channel

The dimensions of the diversion channel and the design as well as location of protective works were, therefore, evolved initially on the basis of this flood discharge. The following considerations had to be accounted for while determining the dimensions of the diversion channel (Fig. 4.2.11) :

- The section should be as narrow as possible so that the quantities of excavation of diversion channel could be handled in a period of 3 to 4 working seasons, which would also enable synchronous utilization of excavated materials in the earth dam zones to be specially designed for the purpose.
- The quantities of rock excavation near about the bed should be capable of being handled in more or less a single working season.
- The affluxed water levels in the river on the upstream due to construction of flow area in the channel during diversion as compared to the flow area of the parent river should not be so high as to result in quantities of earthwork for the diversion dam exceeding about 2.26 million m^3.
- The affluxed levels of the pool upstream when the maximum discharge is negotiated should also not be so high as to generate shooting velocities, which would render the task of constructing matching protective works difficult and very expensive.

Several alternative sections of the diversion channel were studied to evaluate the interplay of the various factors discussed. A width of 235 m reduced excavation quantities materially without increasing the afflux in the river pool. The width of 235 m was suitable from the consideration of raising the diversion dam in a single season and also from the consideration of velocities generated in the channel vis-à-vis the protective works. Typical cross section of the diversion channel is shown in Fig. 4.2.12.

ii. Fair weather channel

The arrangement permitting timely diversion of the fair weather flow of the river to enable raising of the diversion dam in the river channel to EL. 83 m was also given due considerations. This flow could have been allowed to flow in a sheet form in the diversion channel itself but this could have caused considerable inconvenience to the main work of spillway as well as other ancillary works connected with the diversion channel. From hydraulic considerations, a channel located in the center was found to be an ideal location to make it self-flushing consistent with economy. The fair weather flow therefore was confined to a cunette excavated along the centre line of the diversion channel. An 18 m wide cunette with an entry level of EL. 45.11 m and exit level of EL. 43.28 m was found to be satisfactory from economical as well as functional considerations (Fig. 4.2.12).

The fair weather channel crossed the spillway section in masonry construction through five construction sluices each of size 2.13 m x 2.74 m. The sluices on completion of spillway work were to be plugged for the full length of the spillway body and the plug spillway contact was to be grouted by piping system embedded in advance.

The hydro dynamic model studies depicting negotiation of high floods showed that the diversion dam would be badly scoured unless protective guide bund on the right side of the diversion channel was constructed. A solid gravity spur was accordingly designed on non-erodiable bed. The orientation and shape of the spur were evolved after elaborate model studies and it was located at a distance of 76 m downstream from the nose and oriented 68^0 degrees normal to the bank could provide the best deflecting action and impart satisfactory protection to the right bank upstream of the spillway.

4.3 Summary of the Review, study and the inferences

4.3.1 Summary of the review

The review of diversion arrangement adopted for existing earth/earth & rockfill dams is summarized in Table 4.3.1. and important inferences are drawn from the experience of diversions shames which have been studied.

4.3.2 Study and the inferences

4.3.2.1 Inferences drawn from the review of diversion arrangements adopted for existing Earth/Earth & Rockfill Dams.

In case of earth/earth & rockfill dams, overtopping during construction can destroy the part that has already been constructed and causes property & life losses downstream. But in some cases where it may not be economical to provide a tunnel or conduit large enough, consideration was made of permission of part of the flood discharge exceeding the design standard to overtop the uncompleted dam body and for this appropriate protection measures were adopted based on hydraulic model test results (Ord River, Pong). This type of operation may be feasible during the first season but may entail too much risk of major damage or complete loss of the completed fill if used during the second or subsequent high-flow seasons.

- In some cases a portion of the dam is omitted to provide a channel for flood flow passage and remainder of the embankment was placed to a height that will be safe from overtopping (Balimela, Brownlee).
- From the review it can be inferred that earth/earth & rockfill dams particularly present certain advantages because the u/s cofferdam (or both u/s an d/s) is generally incorporated with the main dam and raised simultaneously with it (Mangla, Pong, Ramganga, Oroville, High Aswan). In such instances, the saving is two fold-the amount saved by reducing the cost of embankment material required, and the amount saved by not having to remove the cofferdam. (Fig. 4.3.1).

Type of Valley

In case of steep sided rock valleys the most common method of river diversion adopted is through tunnels constructed through the abutments. The diversion is effected by construction of cofferdams both on the upstream and downstream of the construction area (Oroville, Ramganga, Balimela, Pong).

Method of Diversion.

On the basis of review it has been noticed that, on narrow river channels, the most common practice used is to have one stage river diversion through the tunnels or channels (Ord River, Balimela, Ramganga). But in case of wide valleys the diversion was carried out in stages (Ukai, Tarbela, Mangla, Pong).

Diversion Flood.

The selection of design flood for diversion mainly depends on the amount of risk. In Case of earth/earth & rockfill dams where the area exposed is greater, the damage or loss of partially completed work will be more pronounced on account of overtopping, the importance of eliminating the risk of flooding is relatively great. As in the case of Ramganga and Pong dams, a 1 in 1000-year, for Oroville dam a 1 in 400 year recurrence interval flood and incase of High Aswan 10% more than the maximum flood of 100 year record has been adopted as diversion flood keeping in view the magnitude of risk involved. And in most of the other dams reviewed, diversion arrangement was designed for maximum monsoon flow (Mangla, High Aswan, Ukai).

TABLE 4.3.1: SUMMARY OF DIVERSION ARRANGEMENTS ADOPTED FOR SOME EARTH/EARTH& ROCKFILL DAMS

S. No.	Name of Dam	Type of Dam	Height (m)	Country	Diversion Flood (m^3/s)	Return Period (yrs.)	Type of Valley	Type of Rock	Diversion Arrangement		Remarks
									Waterway	Height of u/s Coffer Dam (m)	
1.	OROVILLE	Earth	235.0	USA	12460	400	Wide	Amphibolite	TUNNEL(2) Dia: 10.7m	126.5	U/s cofferdam was built as part of main dam. Single stage diversion method was applied
2.	TARBELA	Earth & Rock Fill	143.0	Pakistan	21500	10% greater than of record for 100years	Wide	Gabbro, Lime Stone, Schist	TUNNEL(4) Dia: 13.73 m Div. Channel Width: 198 m	30	A buttress dam of 30 m height with 28 opening constructed across diversion channel to redirect the river flow from channel to the tunnels.
3.	MANGLA	Earth	137.65	Pakistan	31000	73	Wide	Sandstones & Clays	TUNNEL(5) Dia: 9.15 m	64	A closure dam of 64 m height was constructed on the u/s side of the core of the main dam to ensure that core trench could be excavated behind closure dam.
4.	PONG	Earth Core - Gravel Shell	132.6	India	22940	1000	Wide	Sand Rock & Clay Shale	TUNNEL(5) Dia: 9.14 m	56	According to the diversion scheme, the flood flows were allowed to pass over the incomplete section of the dam.
5.	RAMGANGA	Earth & Rock Fill	127.5	India	9708	1000	Narrow	Sand Rock & Clay Shale	TUNNEL(2) Dia: 9.45 m	77	The upstream cofferdam is part of the main embankment. Single stage diversion method was adopted
6.	BROWNLEE	Earth & Rock Fill	121.95	USA	5500	Designed for peak wet season flow	Narrow	Basalt	TUNNEL(1) Modified H.S., 11.5 × 12.7 m		Monsoon flow was diverted over partially completed embankment of height 33.5 m.
7.	HIGH ASWAN	Earth & Rock Fill	111.0	UAR	11000	Designed for max. monsoon flow	Wide	Granite	Diversion Channel Width : 61m	48.5	Diversion channel consist of 3 parts. (i) u/s canal, 848.5 m long (ii)Control tunnel(6nos.) spillway, 227 m long. (iii) d/s canal, 358 m long.
8.	ORD RIVER	Earth & Rock Fill	99.0	Australia	70	Designed for dry season flow	Narrow	Granite	TUNNEL(1) Dia: 4.43 m		During wet season, a flow of 5600 m^3/s was passed over partly completed dam with protective measures.
9.	BALIMELA	Earth	70.0	India	280	20	Narrow	Charnockite Khondolites	TUNNEL(1) Dia: 7.75 m	18	During monsoon, flow was allowed to pass over partly constructed dam of 13 m height.
10.	UKAI	Earth cum Masonry	68.58	India	49500	Designed for max. monsoon flow	Wide	Basalt	Diversion Channel Width: 235 m		On closure of diversion channel flow was passed over masonry blocks.

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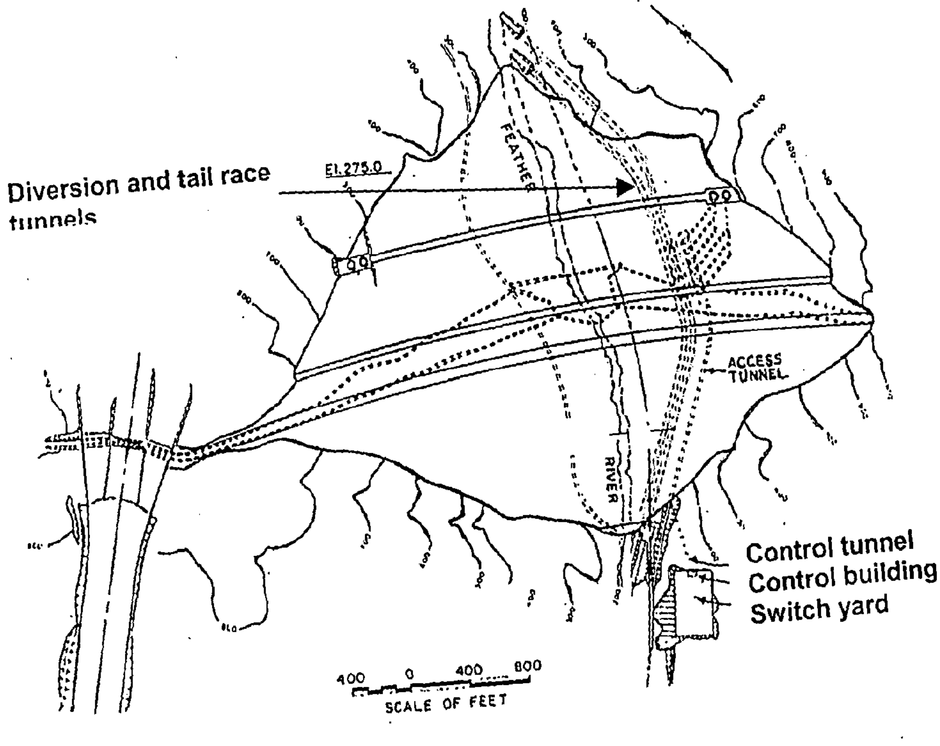


Fig. 4.2.1: Oroville dam – General plan

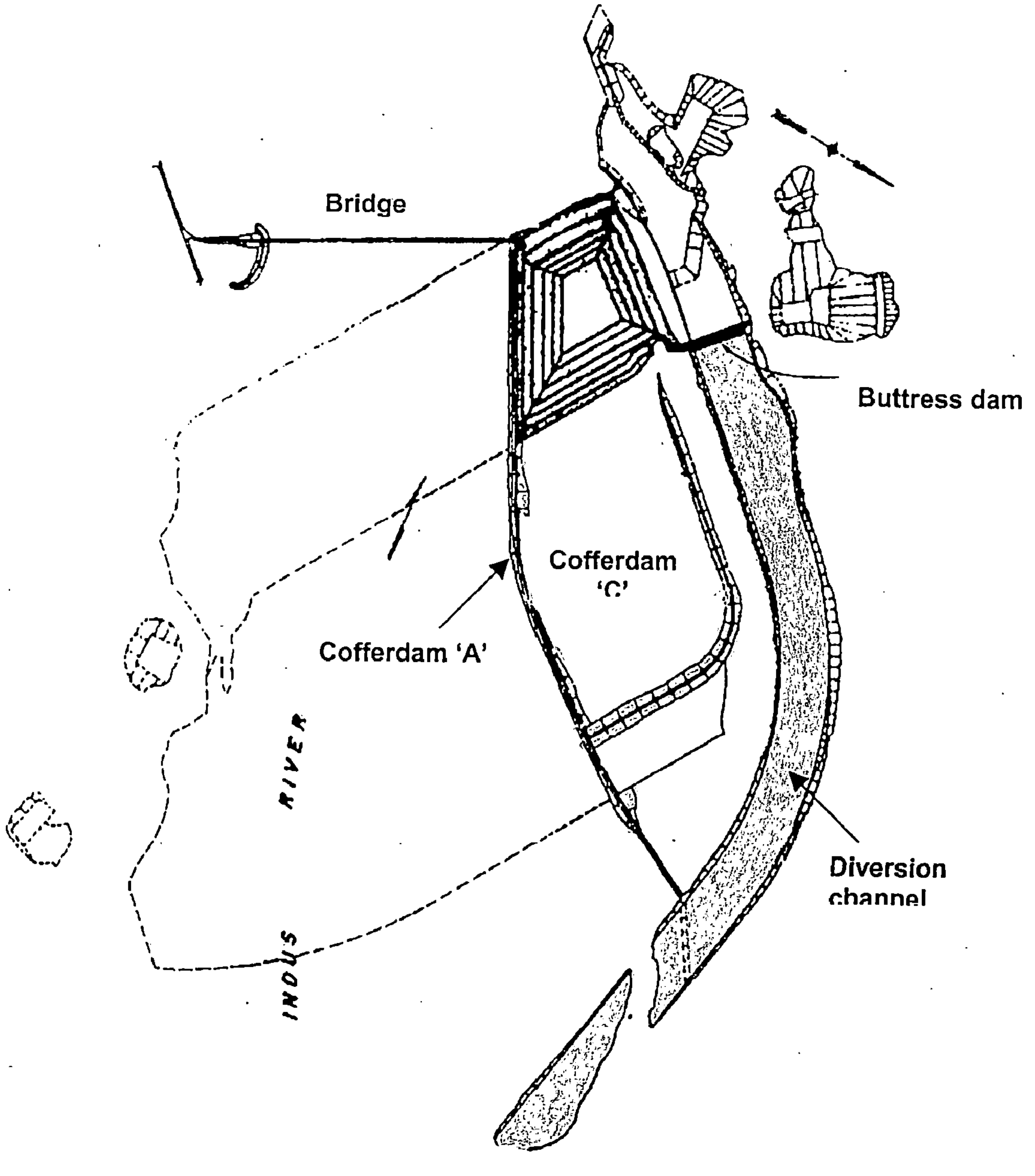


Fig. 4.2.2: Tarbela dam – River diversion stage 1

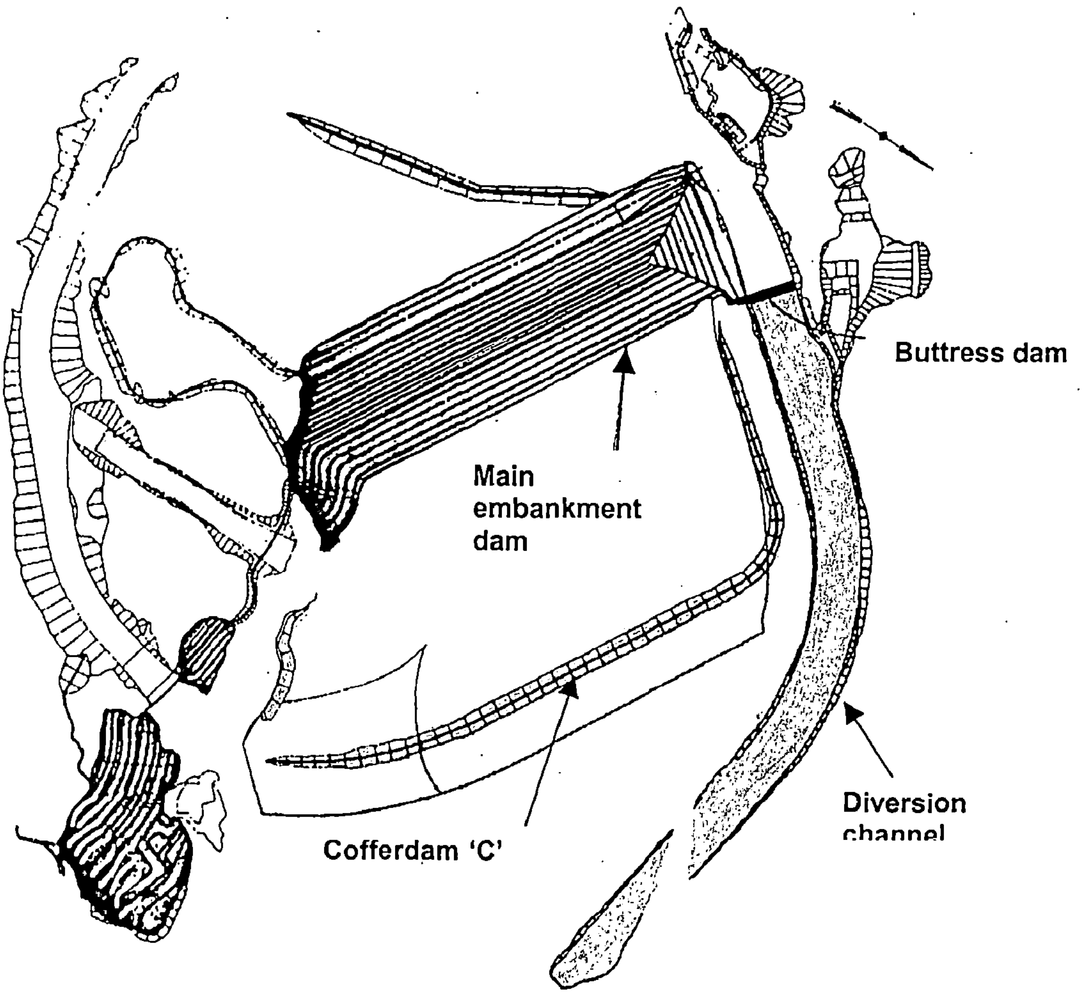


Fig. 4.2.3: Tarbela dam – River diversion stage 2

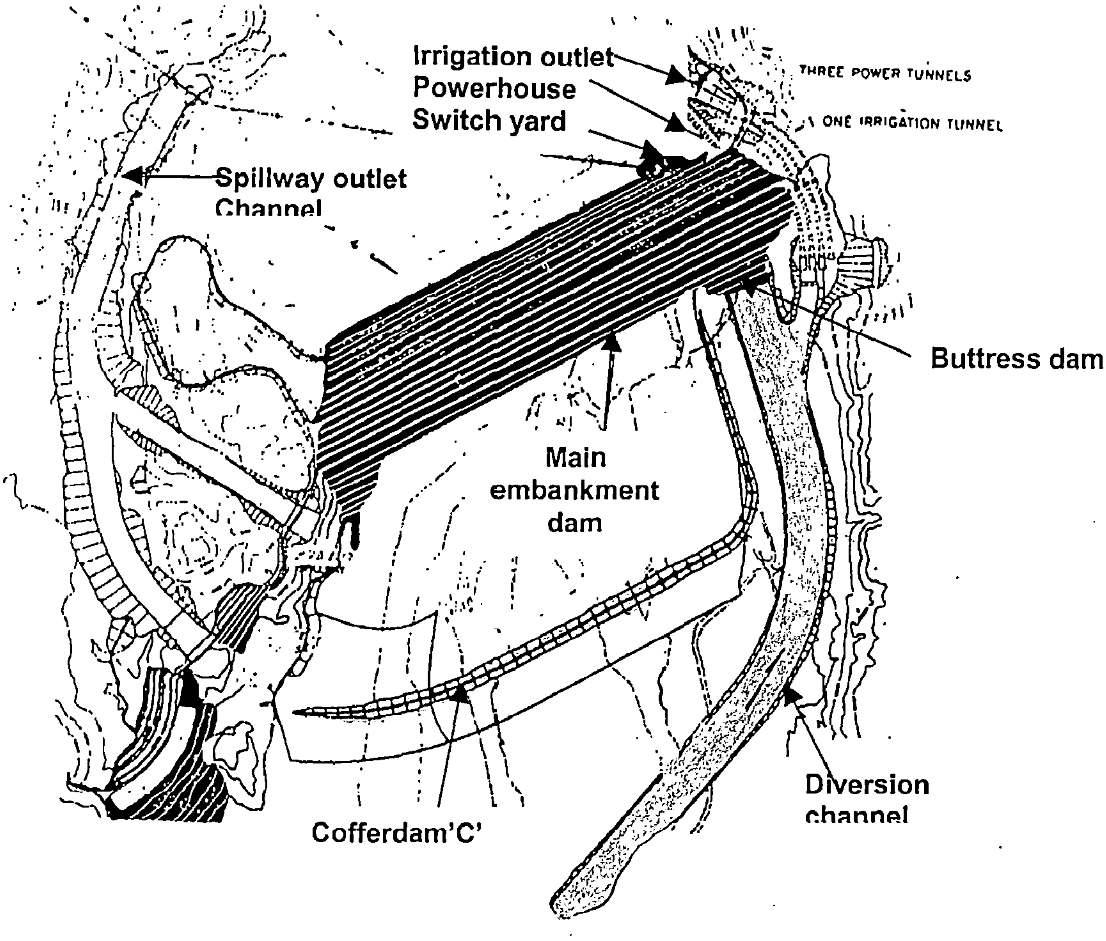
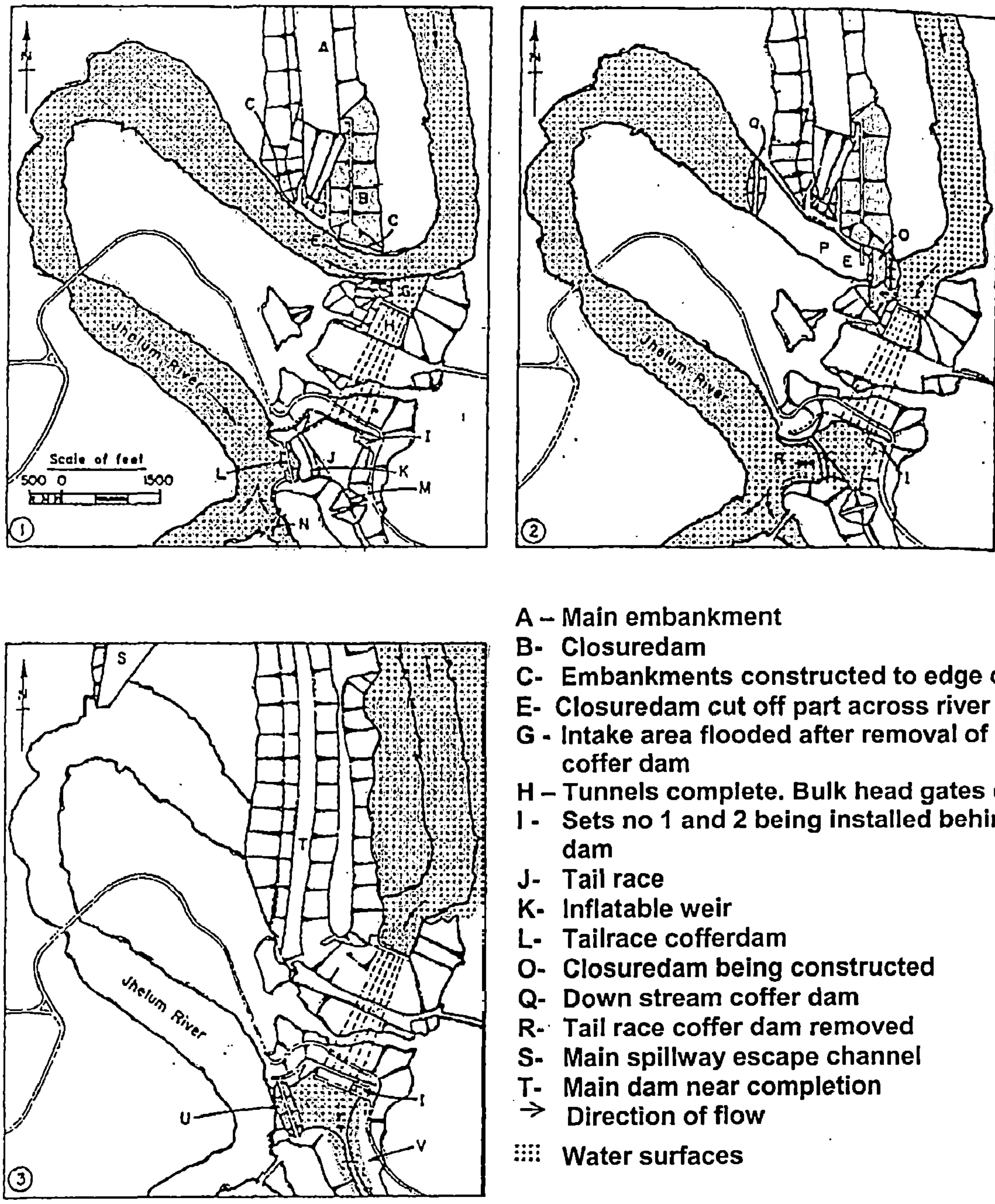


Fig. 4.2.4: Tarbela dam – River diversion stage 3



A – Main embankment
B- Closuredam
C- Embankments constructed to edge of river
E- Closuredam cut off part across river channel
G - Intake area flooded after removal of bed rock coffer dam
H – Tunnels complete. Bulk head gates closed
I - Sets no 1 and 2 being installed behind coffer dam
J- Tail race
K- Inflatable weir
L- Tailrace cofferdam
O- Closuredam being constructed
Q- Down stream coffer dam
R- Tail race coffer dam removed
S- Main spillway escape channel
T- Main dam near completion
→ Direction of flow
:::: Water surfaces

Fig. 4.2.5: Mangla Dam – Stages of diversion

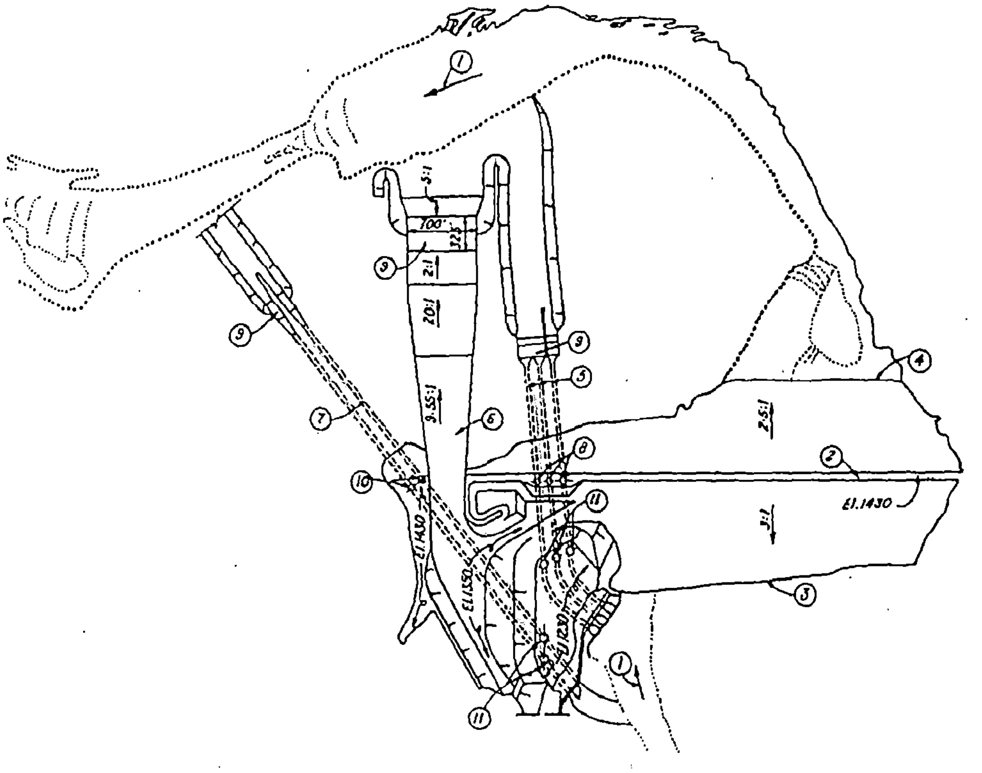


Fig. 4.2.6: Pong Dam – General layout

1. Beas River
2. Top of dam
3. Heel of dam
4. Toe of dam
5. Penstock tunnel
6. Spillway
7. Outlet tunnels
8. Emergency gate shafts
9. Stilling basin

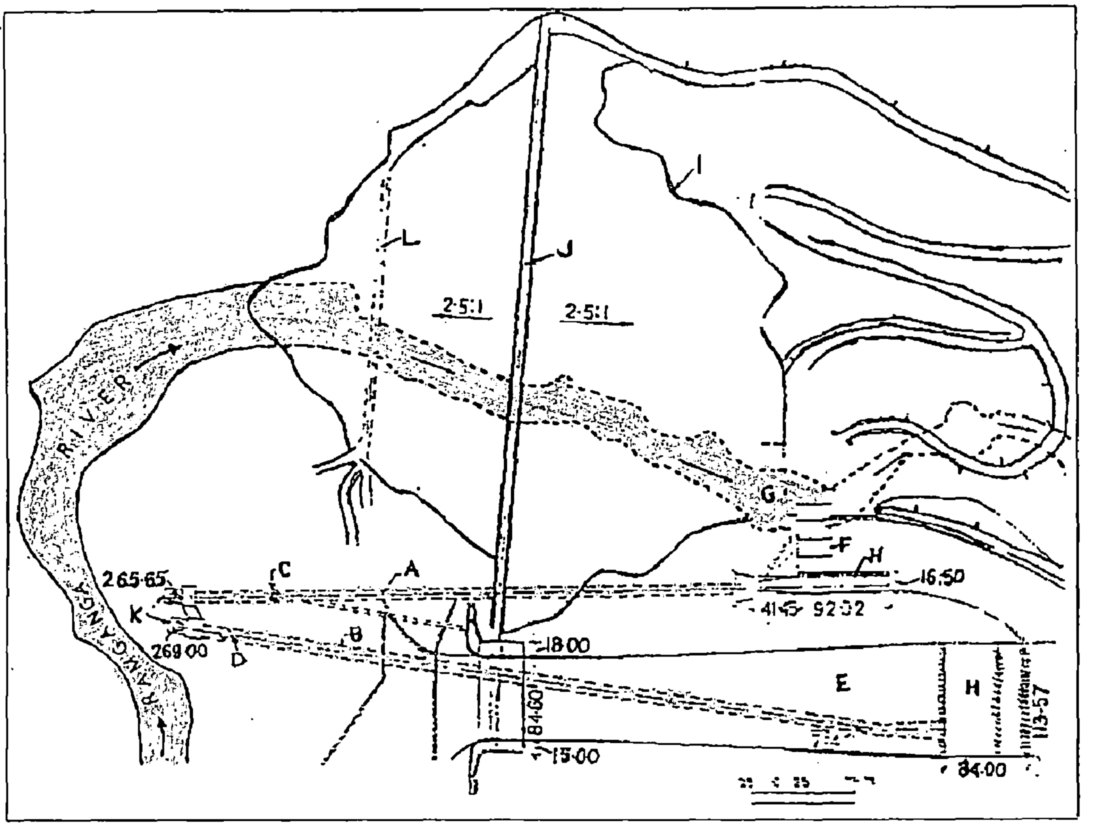


Fig. 4.2.7: Ramganga Dam –Layout of diversion tunnels

A- Tunnel 1
B- Tunnel 2
C- Power intake
D- Auxiliary intake
E- Chute soillway
F- Power house

G- Switch yard
H- Stilling basin
I- Dam toe
J- Dam axis
K- Tunnel inlets
L- First stage dam axis

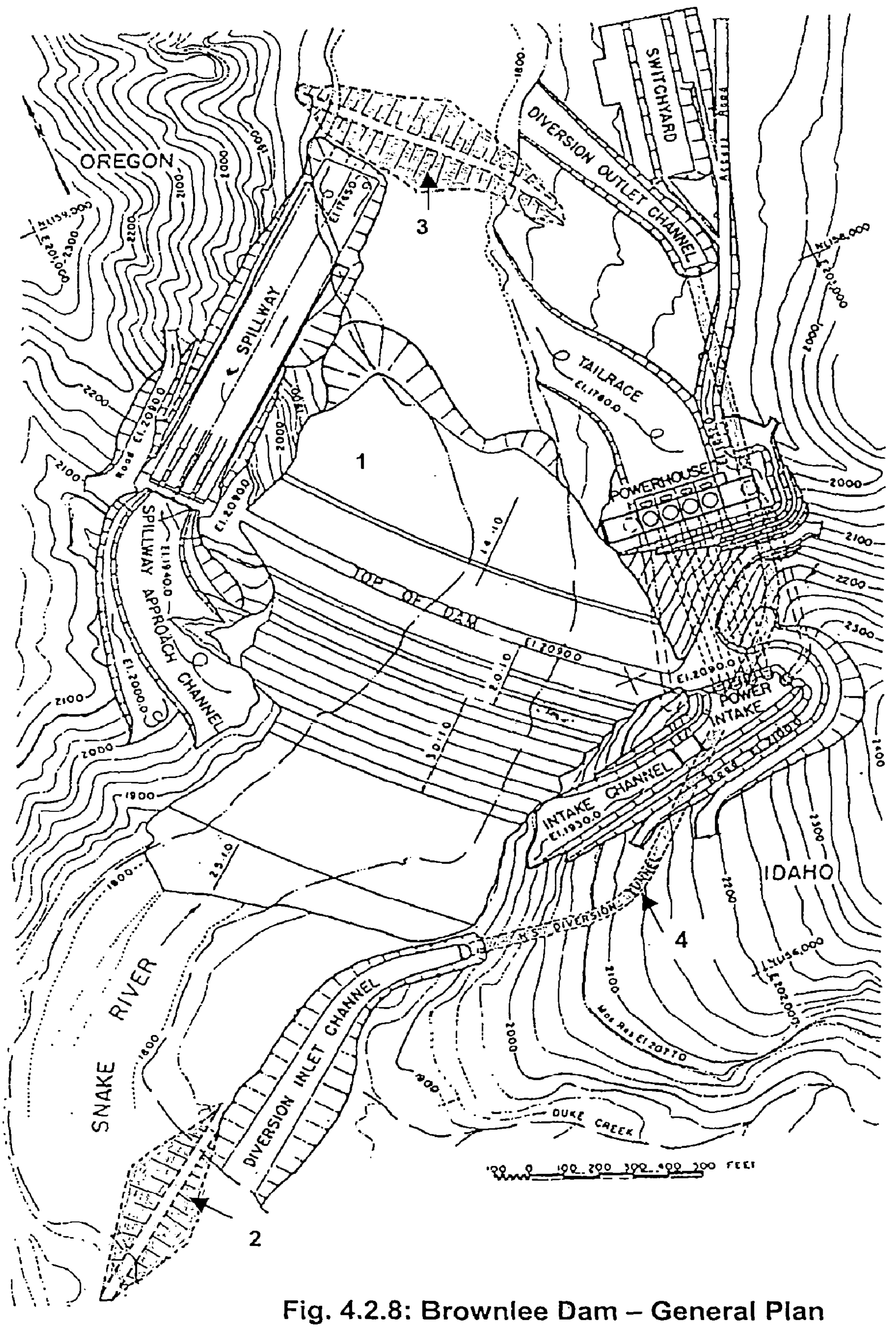


Fig. 4.2.8: Brownlee Dam – General Plan

1. Main dam
2. Upstream cofferdam
3. Downstream cofferdam
4. Diversion tunnel

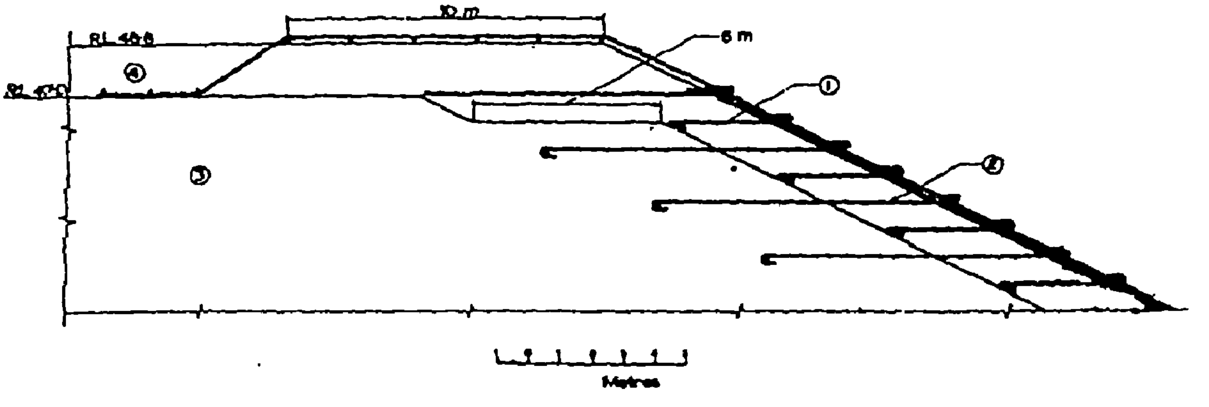


Fig. 4.2.9: Ord River Dam – d/s Protection of Partially completed embankment

1. Anchor 4.2 m long
2. Anchor 10 m long
3. Rock fill
4. Rock armouring

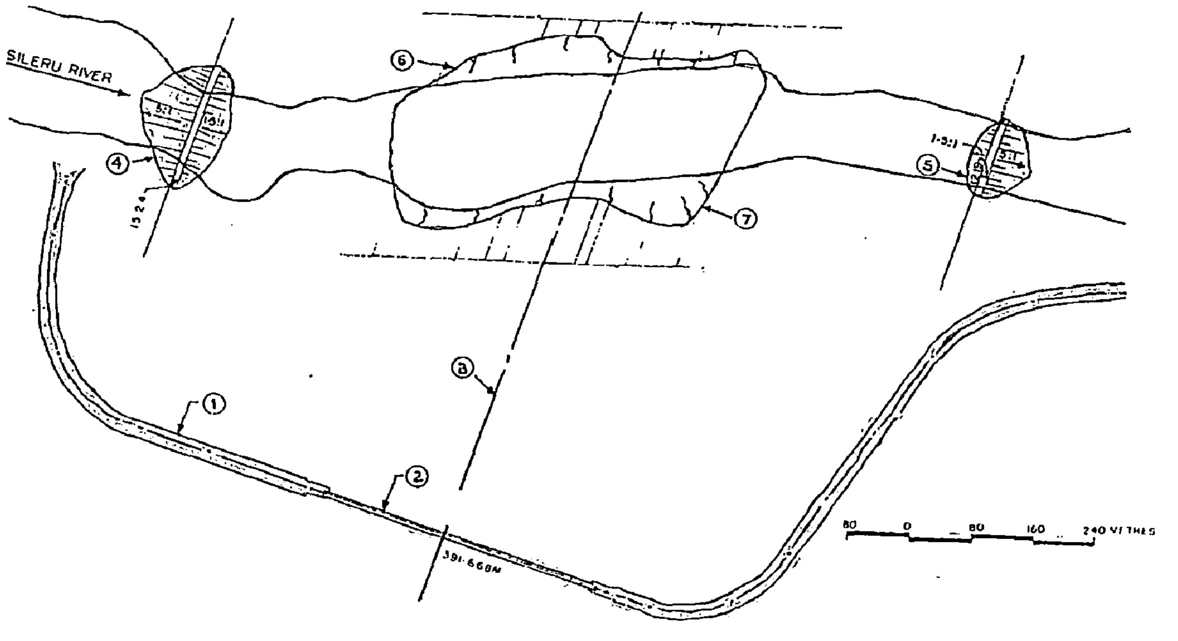


Fig. 4.2.10: Balimela Dam – Layout of Diversion works

1. Diversion channel
2. Diversion tunnel
3. Axis of dam
4. Upstream cofferdam
5. Downstream cofferdam
6. Upstream toe
7. Downstream toe

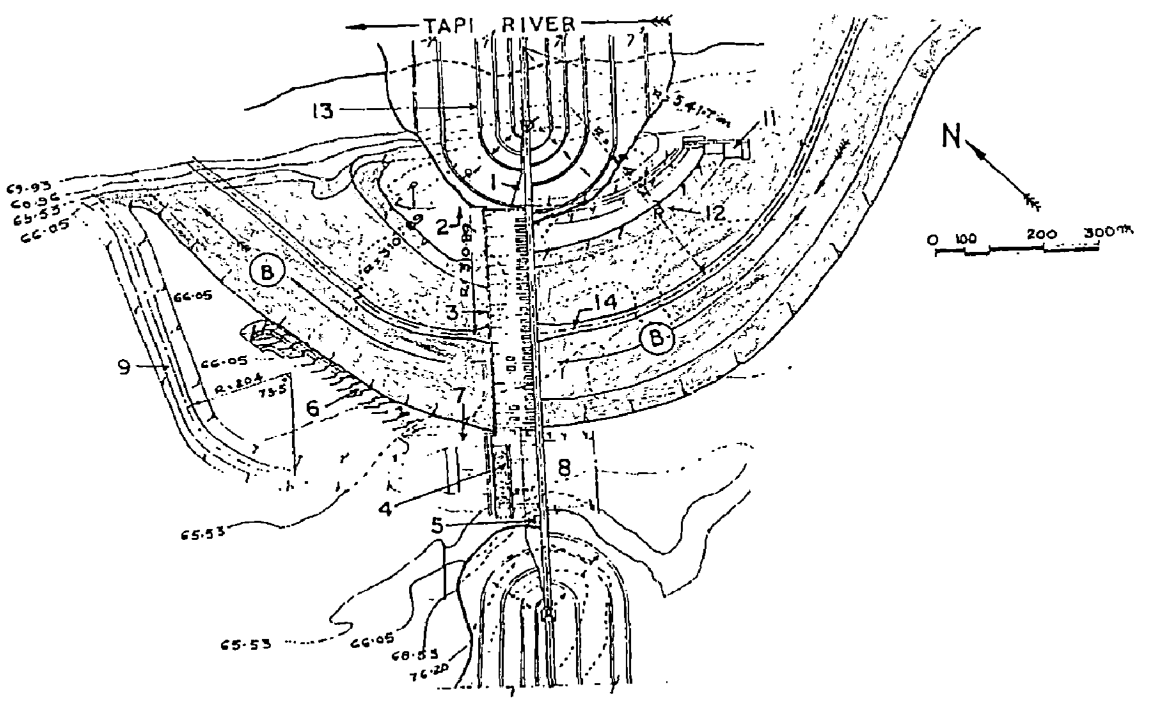


Fig.4.2.11 : (a) Layout plan of masonry dam and diversion channel
(b) Diversion Channel

1. R.T. Dam
2. Downstream ratining wall
3. Spillway
4. Main power house
5. Left transition dam
6. Downstream divide bund
7. Divide wall.
8. Approach
9. Tail race channel
10. Spur
11. Guide bund
12. Earth Dam in river channel
13. Fair weather channel

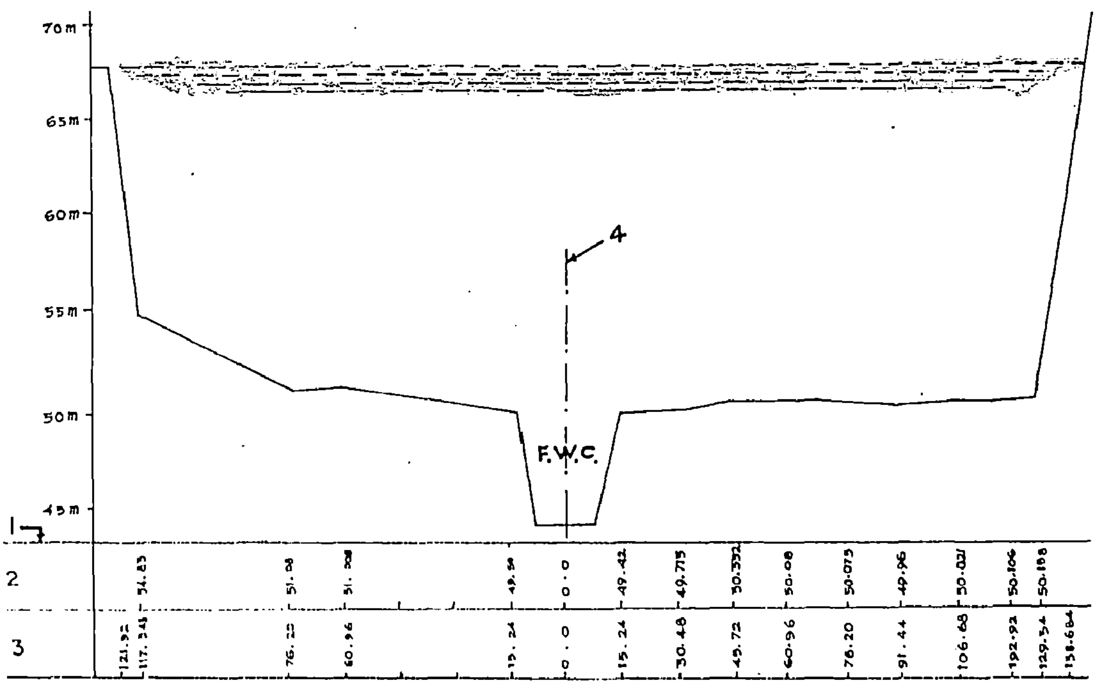


Fig. 4.2.12: (a) Typical cross section of diversion channel

1. Datum
2. Diversion channel rock before flood 1969
3. Distance in meters
4. Centre line of diversion channel

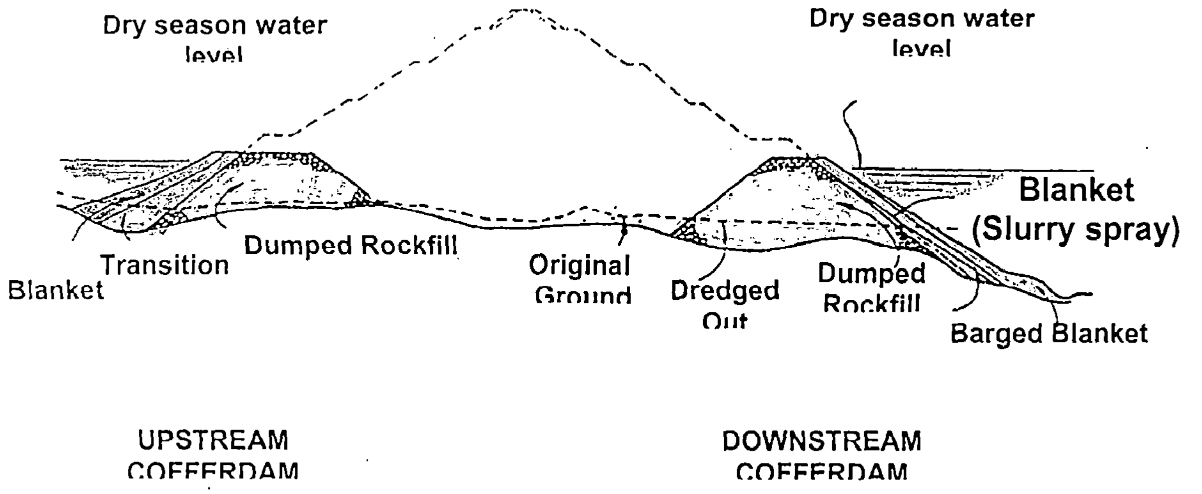


Fig.4.3.1: u/s and d/s cofferdams built as part of main dam

CASE STUDY OF DIVERSION ARRANGEMENT ADOPTED FOR TEHRI EARTH & ROCKFILL DAM

5.1 Project Features

Introduction

River Ganga and its tributaries constitute one of the three large systems of Northern India which are fed from the Great Himalayas and account for 35 percent of India's utilizable surface water resources which are assess4d around 70 M ha m. The river system has substantial hydro power potential, besides being a vast source for irrigation in Gangetic Plains extending over 1500 km in length. The hydro-power potential of the Ganga and its tributaries has been assessed as 16500 MW. So far about 1500 MW of this potential has been exploited mostly on the southern tributaries of the river originating in Vindhayan ranges in Central India. There are no storages on the main river and its two arms Bhagirathi and Alaknanda. To harness the hydro potential of the river, a major storage dam is being constructed on Bhagirathi at a point 1.5 km downstream of Tehri town in the State of Uttaranchal. To meet the peaking power needs, another dam is planned downstream of Tehri at Koteshwar. It will cater for pumped storage operation also. The entire Tehri Hydro Power Complex, as envisaged, would have an installed capacity of 2400 MW and would provide substantial benefits by way of irrigation and domestic water supply.

The Project

The river Bhagirathi originates from Gaumukh glacier, in Himalayas at an elevation of about 6900 m and meets another major tributaries Alaknanda at Deoprayag to from river Ganga. The religious towns of Rishikesh and Hardwar are situated in the foot hills along the Ganga about 70 and 100 kms below Deoprayag respectively. Below Hardwar several canal systems take off from river Ganga utilizing natural flows of the river for irrigation purpose. The average river flows in Ganga in monsoon season (July to Sept.) range between 2000 to 3000 cumecs and the lean flows are about 100 cumecs. The average river slope upto Hardwar is 20 m/km. The salient features of the project are given in Table 5.1.

To harness the potential of river Bhagirathi a multipurpose storage dam is under construction about 1.5 km downstream of Tehri town. The location map of the project is shown in Fig. 5.1.1.

The salient features of the project are :

- A 260.5 m (above deepest foundation) high earth and rockfill dam creating a live storage of 2615 Mm^3.
- An underground power house of 1000 MW (4 x 250 MW) with conventional turbine generating units.
- An other underground power house of 1000 MW (4 x 250 MW) with reversible pump turbine set (Pump storage plant).
- A 103.5 m high concrete dam (which will function as a balancing reservoir) with a surface power house of 400 MW (4 x 100 MW) at Kotehswar about 20 km downstream of Tehri dam.
- Transmission system for evacuation of power generated at Tehri and Koteshwar through 765 kV lines to Meerut.

The project would generated about 4300 million units of energy on 90% dependable and about 5000 million units on average water availability. The peak load power generation is 2400 MW. Besides, the project will provide irrigation facilities to 2.7 lac ha of area in the command of existing canal systems off taking from the Ganga downstream of Hardwar. The project will also provide about 10 cumecs of water to Delhi to meet its future domestic requirements. The total cost of project including the cost of transmission system but excluding the cost of modernizing and extending the canal distribution system is about Rs. 5060 crores.

Seismicity

The Tehri dam project area is seismically active and falls in Zone-IV of the seismic zoning map of India, which corresponds to intensity VIII on MM Scale. An evaluation of the past seismic data has revealed that most of these earthquakes have magnitude of 5-7 on Richter Scale. The dam is designed for a probable earthquake of magnitude 8.0 Site specific assessment of seismicity has been made for detailed design.

Catchment Area Characteristics

The catchment area at the dam site is 7511 sq. km. (2900 sq. miles). Out of this 2323 sq. km. (900 sq. miles) is snow bound. This catchment varies in elevation from 9600 m to 600 m in a length of 187 km. The valley is narrow and moderately forested. The catchment area below the perpetual snow line (4880 m) is divided into following the three types :

i) Agriculture land (1240 sq. km.).

ii) Forest land (2400 sq. km.).

iii) Soyam land lying between the above two types being mainly used by villagers for cattle grazing etc. (1500 sq. km.).

The spread of Tehri dam reservoir is about 42 sq. km. (4200 ha) at full reservoir level of 830 m. Length of reservoir along the river Bhagirathi is 44 km and along river Bhilangana, it is 25 km. Tehri dam will affect 22 villages fully and 74 villages partially besides submerging the Tehri Town. Out of the total submerged area, about 1600 ha is cultivated land, about 1600 ha is the forest land and the remaining is uncultivated. In addition, some area will be acquired for the Tehri Town, project colony, workshop, stores and roads etc. which affect 13 additional villages. The project will displace about 13840 families.

Hydrology

The snow-bound catchment has little rainfalls but it contributes runoff in non-monsoon period due to snow melt. The rest of the catchment has the annual precipitation varying from 101.6 to 263.0 cm. More than eighty percent of annual precipitation occurs during the monsoon period causing occasional floods. These floods often cause soil erosion bringing heavy sediment load in the river. The river discharge at dam site generally varies from 30 to 2000 cumecs, the minimum being in January and maximum in August.

The probable maximum flood (PMF) has been worked out at 15540 cumecs which corresponds to a flood frequency of in 10,000 years and has been adopted as the design flood. The routed discharge for which spillway structures have been designed has been worked out to be about 13100 cumecs.

Table. 5.1: Salient Features of Tehri Dam Project.

1. RESERVOIR	
Catchment Area	7511 Sq. Km.
M.W.L.	835 m
F.R.L.	830 m
Dead Storage Level	740 m
Gross Storage	3540 Million Cum
Live Storage	2615 Million Cum
2. MAIN DAM	
Type	Earth & Rockfill
Top Level	839.50 m Height
Height	260.50 m above deepest foundation level
Width at river bed	1141 m
Length at Top	575 m
Width at Top	20 m, Flared 25 m on abutments
3. DIVERSION TUNNELS	
Type	Horse Shoe
On Left Bank	2 Nos., 11.30 m dia, 1774 & 1778 m Long
On Right Bank	2 Nos., 11.30 m dia, 1298 & 1429 m Long
Diversion Flood	8120 Cumecs
4. SPILLWAY	
(A) Chute Spillway	
Crest Elevation	815.00 m
Design Discharge	15480 Cumecs
No. & Size of Bays	3 Nos. & 10.50 m each
No. & Type of Gates	3 Nos. Radial Gates

(B) Right Bank Shaft Spillway	
Crest Elevation	
Design Discharge	830.20 m
Intake Type	3879 Cumecs
(C) Left Bank Shaft Spillway	2 Nos. Ungated Funnel Shaped
Crest Elevation	
Design Discharge	815.00 m
Intake Type	3750 Cumecs
	2 Nos. Gate Weir type Intake
5. POWER HOUSE	
Installed Capacity (MW)	2000
Conventional Units (Nos. XMW)	4 X 250
Reversible Units (Nos. XMW)	4 X 250
Type	Under Ground 2 Nos.
Cavity Size	199 x 22 x 63 m (Stage-I), 144 x 22 x 65 m (Stage-II)
Design Head	231.5 m
Gross Head	188 m
Head Race Tunnel	4 Nos. 8.50 dia
Total Length	4441 m
6. BENEFITS	
Annual Generation	3091 Million Units. (90% availability)
Annual Irrigation	0.27 Million Hectares
7. YEAR OF COMPLETION	2004

5.2 River Diversion Scheme

Tehri dam being in a deep and narrow gorges, tunnels are the only possible means for diversion of flow during construction. There is a loop in the river Bhagirathi after its confluence with river Bhilangana at the dam site. This configuration is such that two diversion tunnels can be driven in the right bank from river Bhagirathi and discharge on the same bank of the river downstream of the dam site and the other two in the left bank from river Bhilangana and discharge on the left bank of river Bhagirathi further downstream the point where right bank diversion tunnel discharge.

The diversion of the river has been effected by 4 no. diversion tunnels of 11 m. diameter standard horse shoe-sections with combination of two cofferdams, one on the upstream end the other on the downstream for diverting the monsoon as well as the non-monsoon flow of the river through the diversion tunnels during construction of the dam. The layout of diversion arrangement is shown in Fig. 5.2.1. As soon as the right bank tunnels are ready, a closure dyke was constructed just downstream of the location of their inlets during non-monsoon period and the river flow diverted through the two tunnels. To prevent any back flow of water in the dam area, a closure dyke was also constructed just upstream of the tunnel outlets.

After the completion of the two right bank tunnels, the following scheme for river diversion has been adopted.

First Dry Period

In the first non-monsoon period (dry period) after the completion of the two right bank tunnels, the following construction work has been done in the riverbed.

(i) Construction of closure dykes just downstream of tunnel inlets and just upstream of tunnel outlets respectively, to divert the non-monsoon river flow into the tunnels and to prevent backflow in the dam area.

(ii) Riverbed excavation up to fresh rock level in the core area.

(iii) Curtain grouting under core and blanket grouting in the core and transition area.

(iv) Core contact treatment i.e. treatment of shear zones and fractures in the rock.

(v) Stripping and trenching under first stage dam

(vi) Construction of the dam section, including the core and transition zones up to original river bed level.

(vii) Provision of protection over this part section of the dam to allow flow in the flood season.

First Flood Period

Passing the flood through river section constructed upto original river bed level and through tunnels. The upstream and downstream closure dykes shall be allowed to breach.

Second Dry Period

(i) Reconstruction of upstream and downstream closure dykes which breached in the first flood period.

(ii) Construction of first stage dam up to full proposed height i.e. elevation 681.0 m, so as to divert the design flood corresponding to 1000 years return period with a peak discharge of 12848 cumecs (Fig. 5.2.2) through the two tunnels of right bank and two tunnels on the left bank which will be completed by then.

(iii) Construction of downstream cofferdam is a part of main dam safe for back water of 1000 years flood passing through the diversion tunnels. If some how the downstream cofferdam cannot be taken up in second dry season, its construction shall be taken up and completed in third working season simultaneously with the main dam construction and shall be completed before the onset of monsoon. In this case however, the downstream closure dyke shall have to be constructed once again in the third dry period before the downstream cofferdam construction is taken.

Second Flood Period

Diversion of the flood through all the four diversion tunnels..

5.3 Diversion Tunnels

Two diversion tunnels viz. T_1 and T_2 shall take off from the left bank of river Bhillangana, a tributary to Bhagirathi and are known as left bank tunnels. The rest two right bank tunnels T_3 and T_4 take off from the right bank of river Bhagirathi, a little upstream of the main dam. Out of the four diversion tunnels of 11 meter dia standard horse-shoe-section, two on the left bank i.e. T_1 and T_2 have their inverts at EL. 632.0 at inlet and at EL. 596 at outlet. The right bank tunnel T_3 is the lowest having its invert at EL. 606 at the inlet. The other right bank tunnel T_4 has invert at EL.609 at inlet and at EL. 603 at outlet. The invert level of tunnels at the inlet has been kept different for facilitating inspection and repair during lean flows and final closure after the diversion needs are over. The salient features of diversion tunnels have been mentioned in Table 5.3.1. The alignment of the tunnels T_1, T_2 and T_3 has been kept straight where as a bend having a radius of 33m and an angle of 50^0 has been provided at the inlet of right bank tunnel T_4 as shown in Fig. 5.3.1.

Geology

The project area lies in a tectonically disturbed belt. The rocks exposed in the vicinity of the dam site are alternate bands of quartzitic and phyllitic material of chandpur series. The bands are of variable thickness and have dips of 40^0 to 60^0 in downstream direction. The rocks are broadly graded as phyllites of grades I, II, III and sheared phyllite.

Construction

The excavation diameter of 12.5 m to 13 m was economically constructed using shotcrete and anchors in phyllite grade I rock and steel support in phyllite grade II & III rocks. Although a circular section is structurally more efficient and appropriate for poor rocks yet a standard horse-shoe section (Fig. 5.3.2) of 11 m finished diameter was adopted for Tehri dam on account of speedier construction and economical grounds. Longitudinal sections of tunnels T_1 & T_4 is shown in Fig. 5.3.3. For the most construction periods the right diversion tunnels will remain in operation.

Bell mouth transitions have been provided on the top as well as on the two sides with invert at the approach bed level and the section at entry conforming to a 12.65 m diameter horse shoe section. The elliptical transition on all the sides were provided in a length of 5.5 m (0.5 D) section at the end of the transition conformed to 11 m diameter horse shoe section. Fig. 5.3.4 shows the plan of inlet portals for diversion tunnels T_1 and T_2. For suppression of vortex formation in the right tunnels anti-vortex beams were provided which were effective in eliminating vortex at all operating levels. In case of left tunnels only anti-vortex beams were found inadequate in suppressing the vortices at higher reservoir levels.

All the four diversion tunnels are proposed to be utilized as spillways (Fig. 5.3.5) by connecting them with 12 m dia vertical shafts. The two left bank shaft spillways are gated whereas the two right bank spillways are ungated. The vertical shafts are connected with the horizontal diversion tunnels through an eccentrically placed reducers which causes spinning flow in the tunnels.

The spinning flow of air water mixture in the tunnel rapidly reduces the velocity. Thus with this arrangement low exit velocity are achieved which otherwise are very high in conventional shaft spillways and requires expensive energy dissipation measures at the exit.

5.4 Rating curves for Tunnels

Diversion tunnel rating curves (Fig. 5.4.1) have been developed with the help of stage discharge curve at outlet portal site (Fig. 5.4.2) as per the data collected from the office of Tehri Hydro Development Corporation, Rishikesh. Data used for tunnels is given in Appendix-I. The calculations which have been carried out for 11 m dia horse shoe tunnels are given in Appendix-II. The values of combined discharge through four diversion tunnels at different reservoir elevations is found to be same with the values supplied by the project authorities.

5.5 Cofferdam

To divert the river flow in to the tunnels the upstream cofferdam which is also referred to as first stage dam has been constructed to cater for a design flood of 1 in 1000 years frequency with a peak discharge of 12848 cumecs. This flood when routed through four tunnels of 11.0 m diameter each, rises to an elevation 676.0 m providing a free board of 5.0 m the top of the upstream cofferdam was fixed at elevation 681.0 m giving an overall height of 81 m above the river bed. Where as the height of the downstream cofferdam is 38 m. both the cofferdams have been constructed as part of the main dam (Fig. 5.5.1). Upstream cofferdam is a rockfill type structure with a well graded impervious core protected at both sides by filter material. The upstream face of the cofferdam is overlain by well graded hard blasted rock. The section of the upstream cofferdam is shown in Fig. 5.5.2.

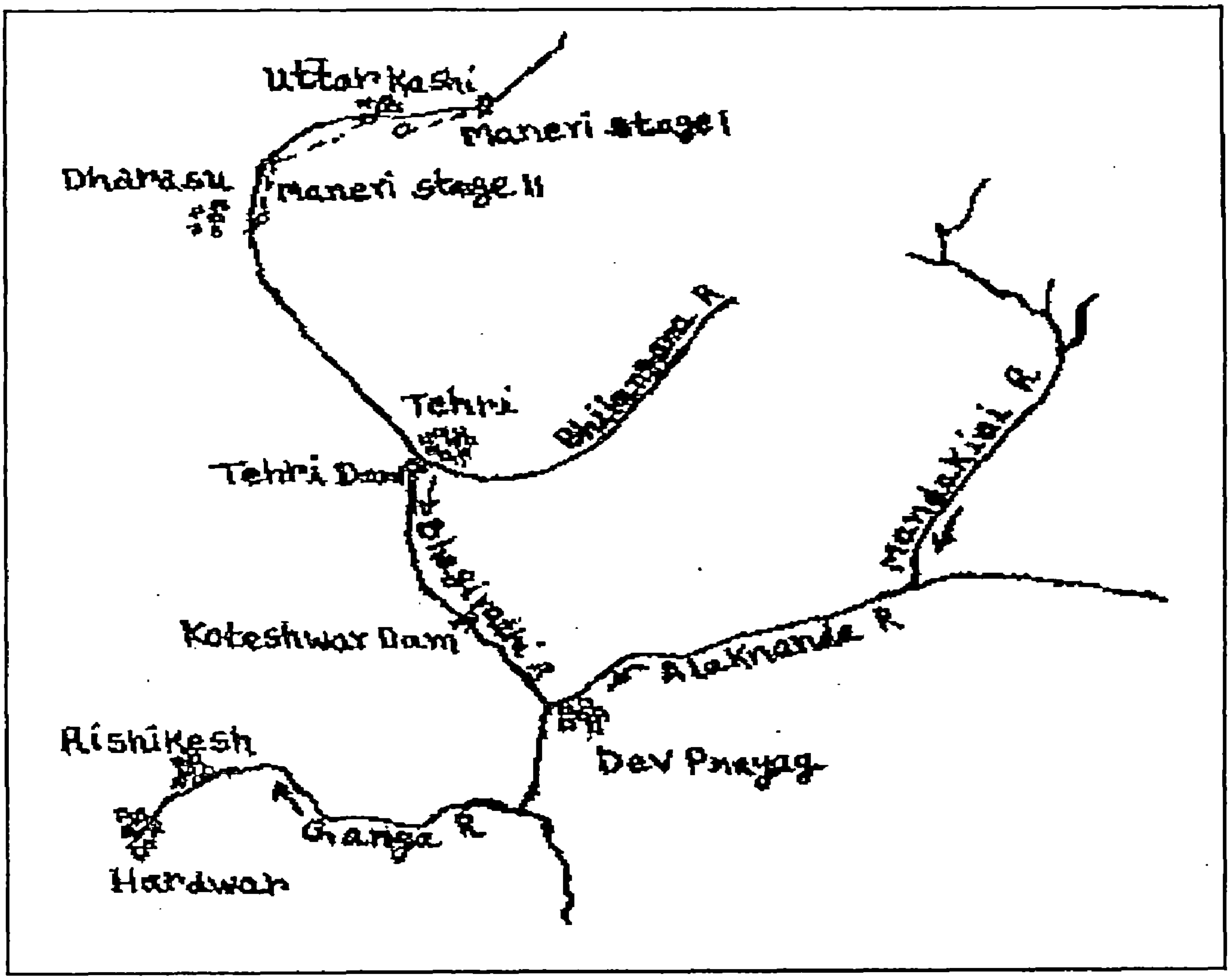


Fig. 5.1.1: Location map of Tehri Dam project

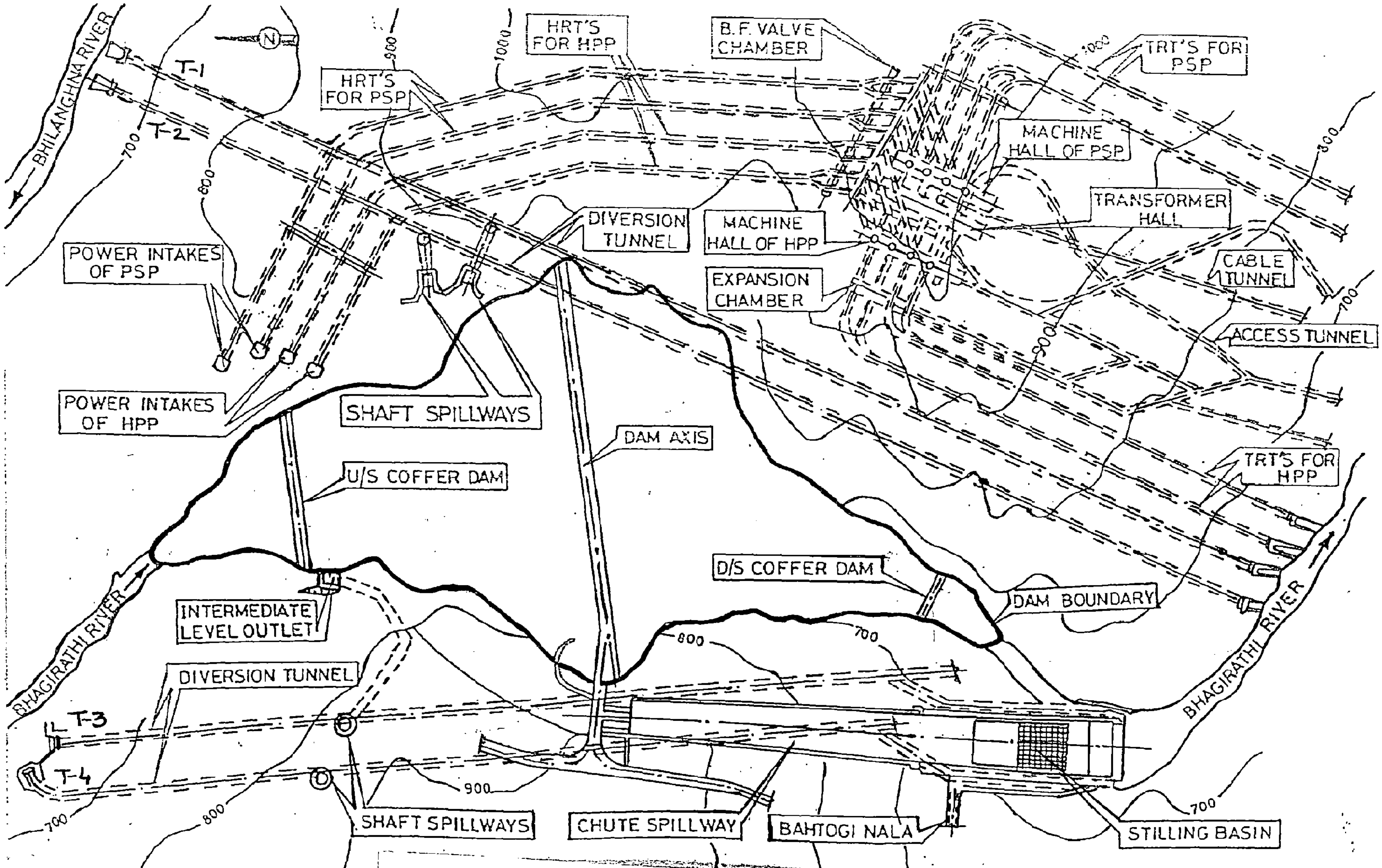


Fig. 5.1.2 General layout of project

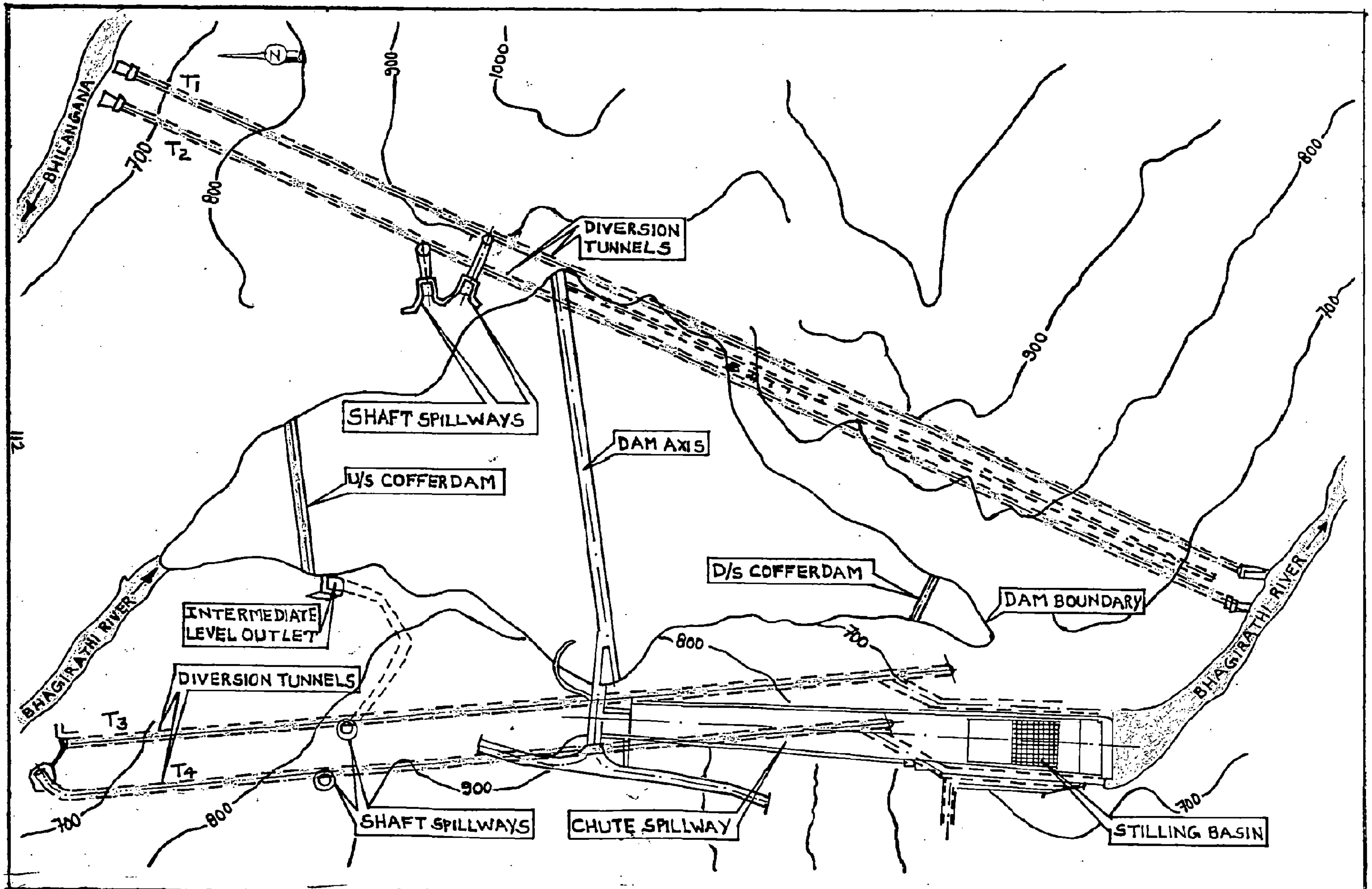


Fig. 5.2.1 Tehri Dam - Layout of diversion works

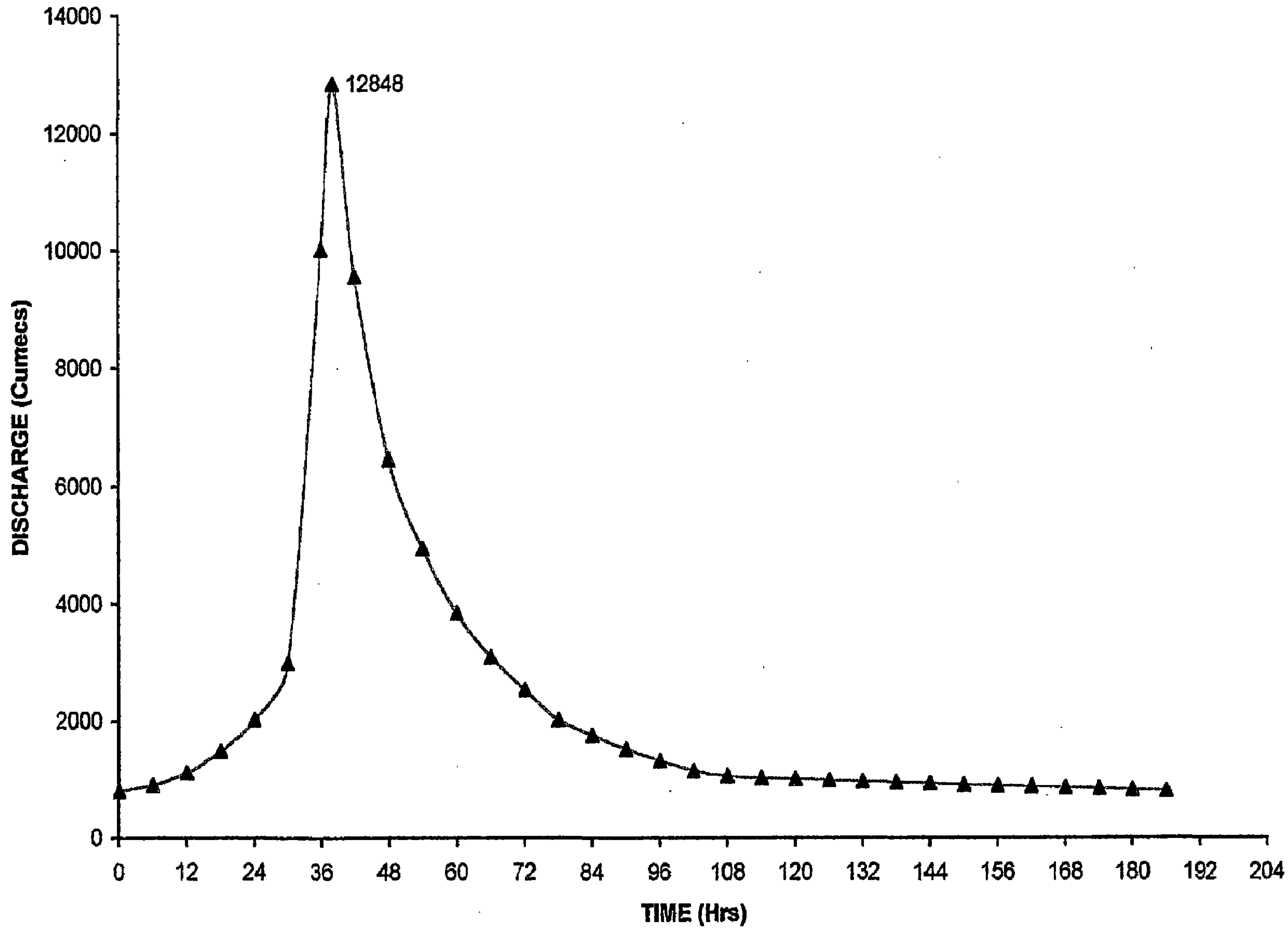


Fig. 5.2.2 1 IN 1000 YEAR FLOOD HYDROGRAPH

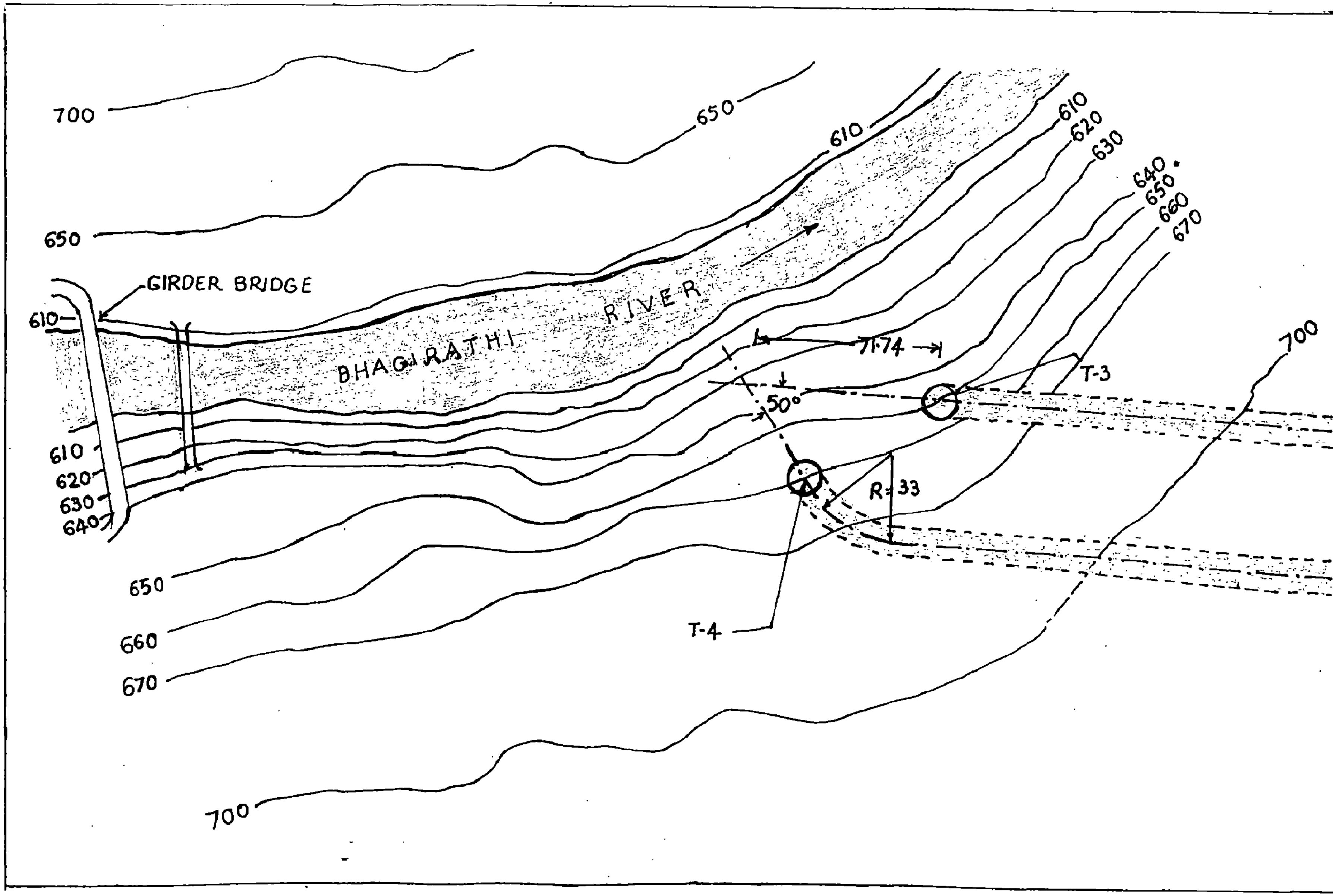


Fig. 5.3.1 Layout of Right bank Diversion Tunnels

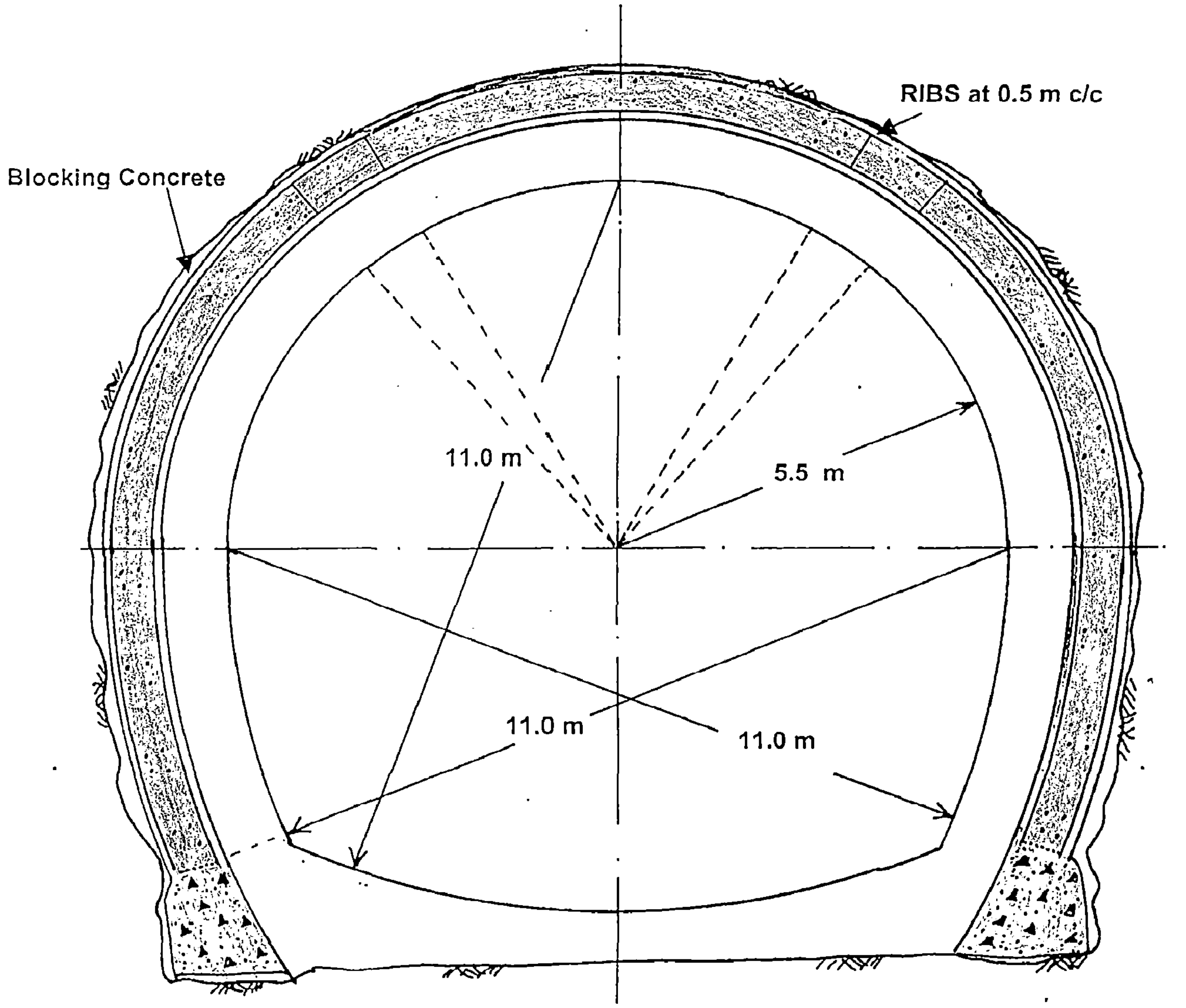
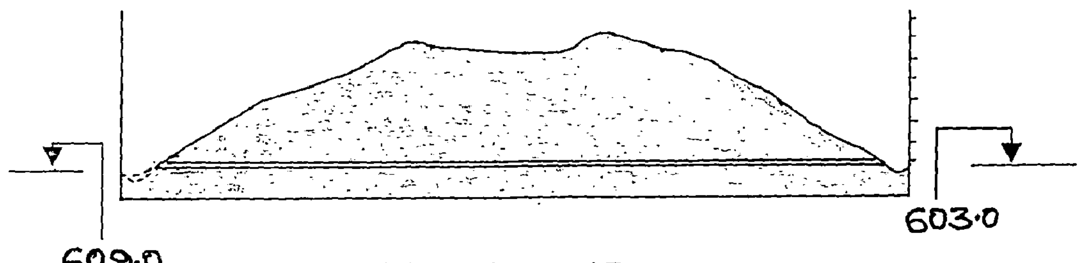
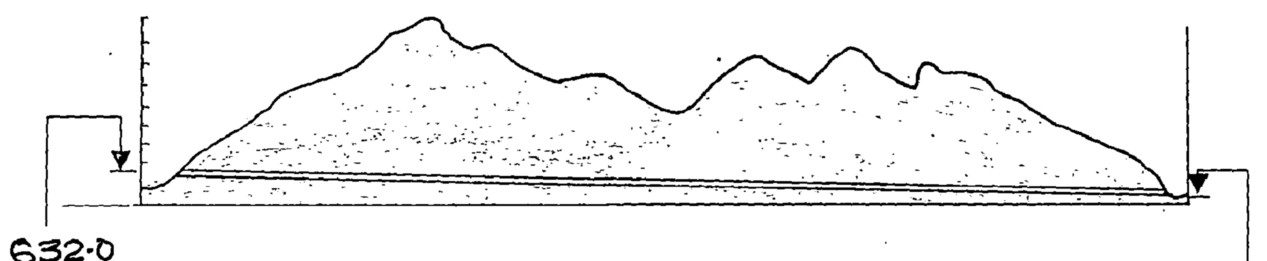


Fig. 5.3.2: Diversion Tunnel Section



Right bank tunnel T_4



Left bank tunnel T_1

Fig. 5.3.3: L-section of diversion tunnels

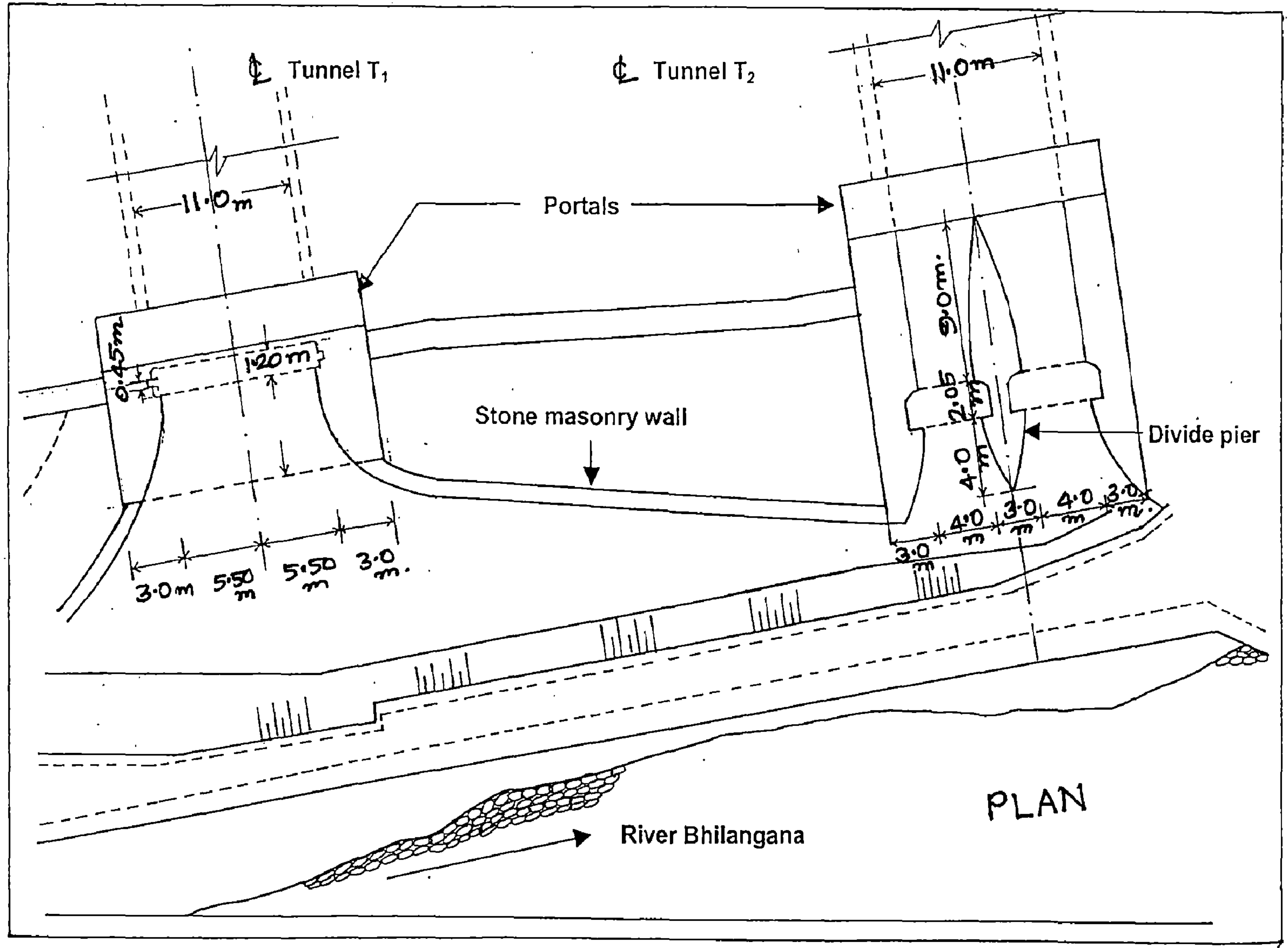


Fig. 5.3.4.: Plan of Inlet portals for tunnels T_1 & T_2

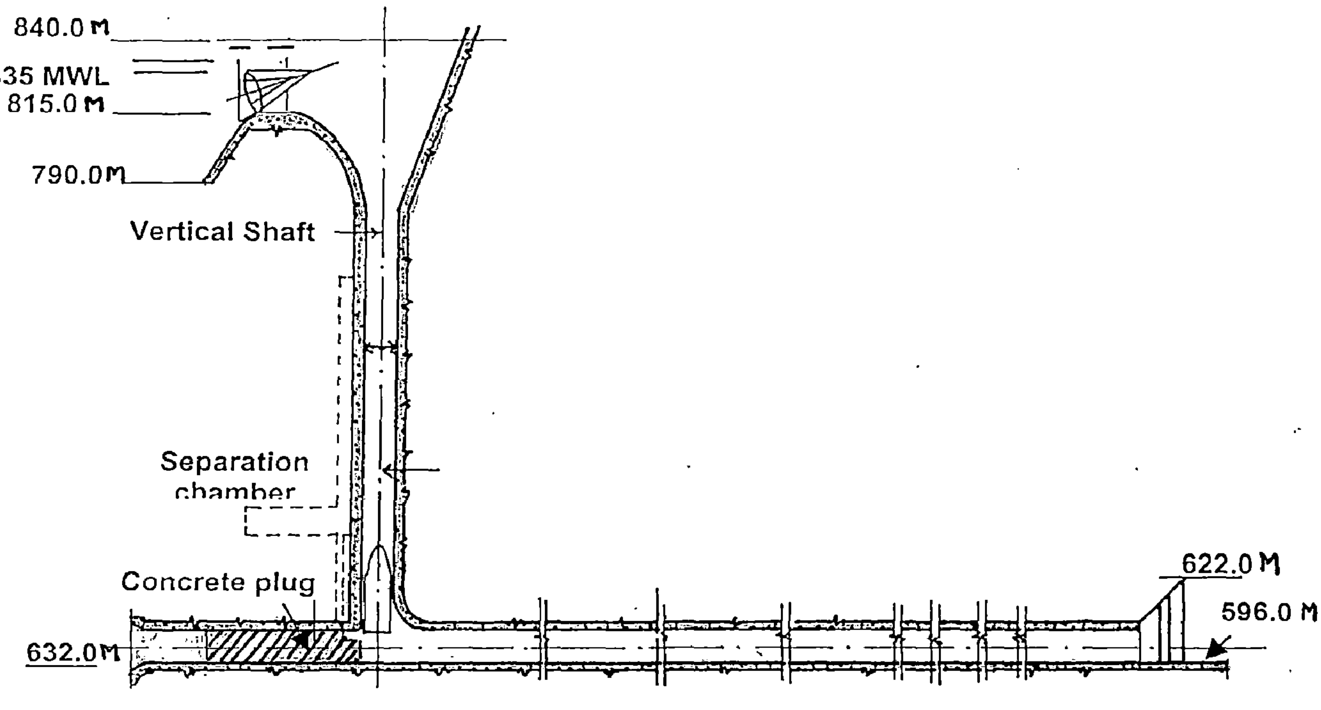


Fig. 5.3.5.: Conversion of diversion tunnel in to spillway

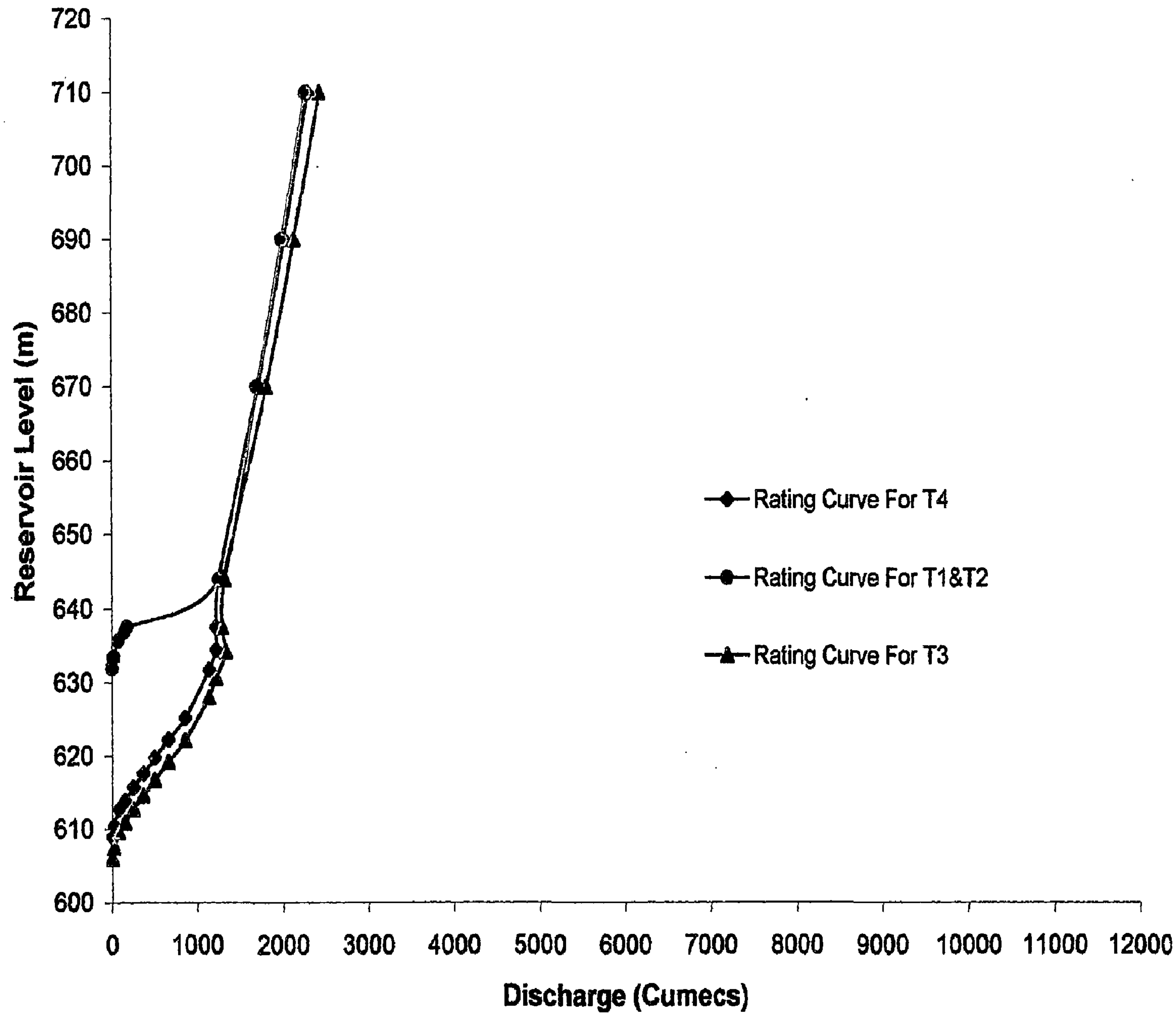


Fig. 5.4.1 DIVERSION TUNNEL RATING CURVES

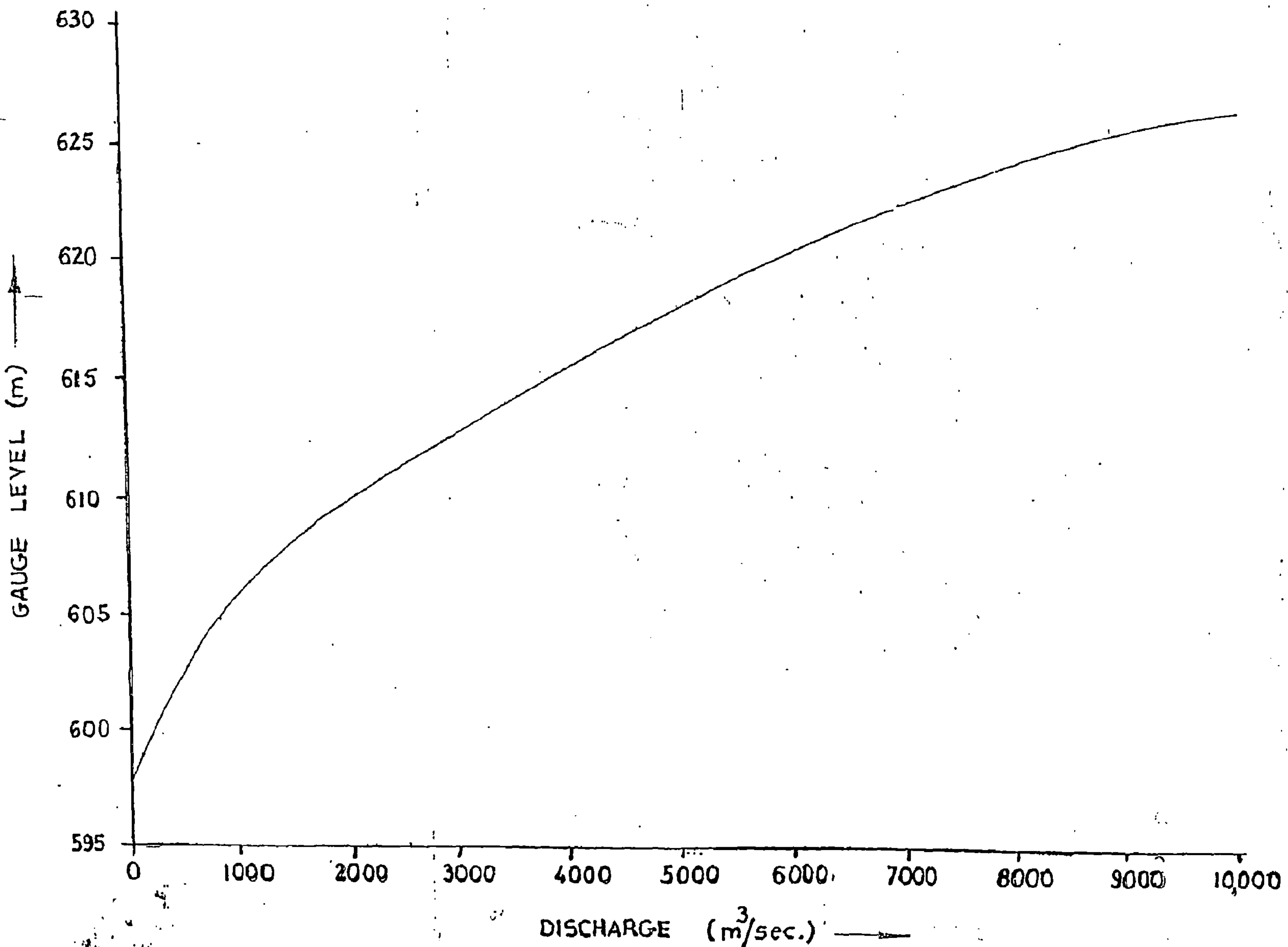


Fig-2 STAG **Fig. 5.4.2 Stage-discharge curve at tailrace**

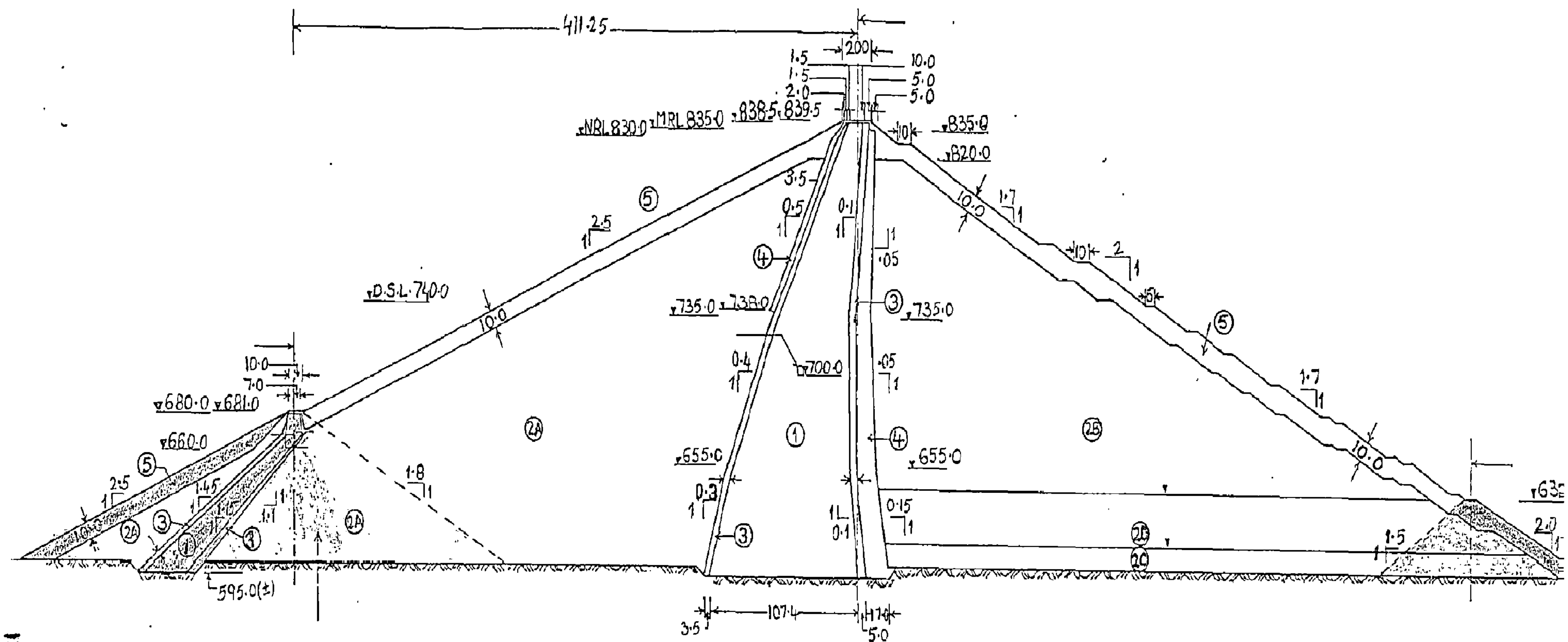


Fig. 5.5.1 Main dam section - Cofferdams built as part of Main dam

1 Well graded impervious blended core material.
2A Well graded terrace gravelly material, maximum size 600mm, fines less than 35%, still content not more than 5%.
2B Well graded terrace gravelly material, maximum size 600mm, fines less than 35%, silt content not more than 5%.
2C Well graded terrace gravelly material, maximum size 600mm, fines between 10-22%.
2D Well graded terrace gravelly material, maximum size 600 mm, fines between 10-18%.
3 Fine filter silt content less than 3% size.
4 Coarse filter sand and gravel mixture, maximum size ≤ 60 mm.
5 Well graded hard blasted rock with maximum size upto 1200 mm.

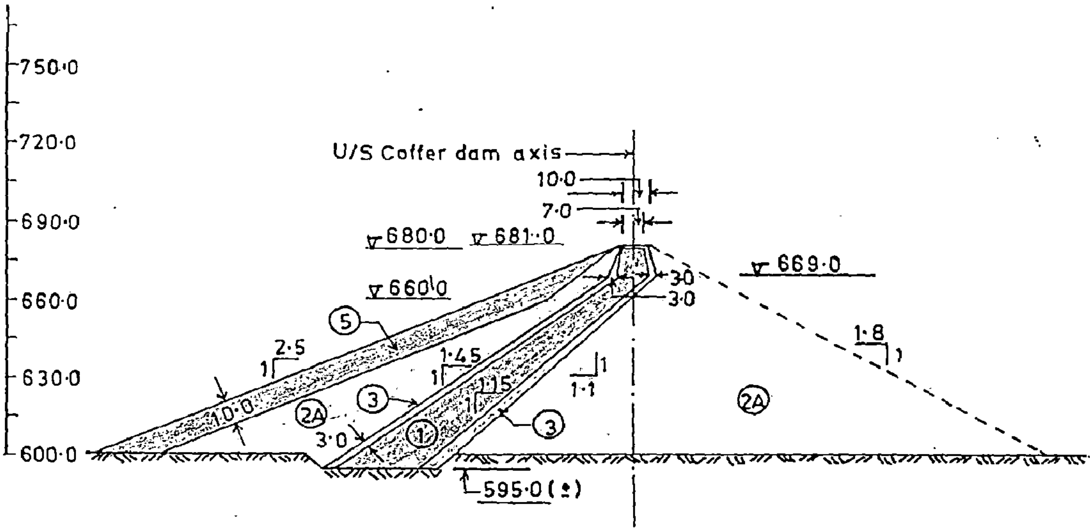


Fig. 5.5.2: Upstream cofferdam section

1. Well graded impervious blended core material

2A Well graded terrace gravelly material, maximum size 600mm

3 Fine filter

5 Well graded hard blasted rock

CONCLUSIONS AND SUGGESTIONS

6.1 General Conclusions on River Diversion

The following conclusions are made from the study presented in the preceding chapters.

i. The planning and design of river diversion works for a major dam project need careful consideration as these are costly and time consuming works.

ii. It is not easy to lay down definite rules on this problem since the diversion of the river is always subject to specific data, which one cannot easily fit into a standard pattern, such as the flow of the river and the characteristics of the terrain.

iii. Diversion is generally more of a problem with earth dams than concrete dams, since in the latter case if the cofferdam is overtopped during construction only the cofferdam will get damage but in case of earth dam apart from destroying the cofferdam, overtopping also causes serious damage to the enclosed embankment and damage to life and property below if a sudden release of the impounded water should occur.

iv. In designing river diversion works an engineer is presumed to be making a calculation of the risks involved since absolute certainty can not be afforded. Each river and each dam site represented, by their physical and technical characteristics, a unique case, subject to a judicious and detailed individual analysis taking account of all the available data. No general ruling can define the risks to be accepted in an individual case. Generally a return period of 1 in 10 to 1 in 40 years is adopted for concrete dams and 1 in 100 to 1 in 1000 years for earth and rockfill dams.

v. Usually single stage diversion with tunnels and cofferdams has been adopted. Where ever abutment rocks are not good, channels have been built as in the case of Ukai, Tarbela etc.

vi. In case of earth and rockfill dams usually cofferdams required are quit high and are not allowed to overtop and so these are made part of main dam. In a few cases the cofferdams are allowed to over top and toe of incomplete dam is strengthened to withstand damage. But such experiences are a few only.

vii. Model tests in most of the cases have been extremely useful in analyzing most of the problems related to river control and especially for diversion control structures including interaction with the river system and for closures on the natural river bed. However, these results should be used with caution because certain parameters are difficult to simulate in model tests such as seepage paths, internal stresses with in materials, junctions with the banks, mechanical effects in relating to the foundation, mesh and steel reinforcement.

viii. Cofferdams are generally made as earthen embankment, usually zoned. Some times when these are not the part of main dam these are made of concrete or stone masonry or rockfill.

ix. Diversion tunnels are usually converted into permanent structures such as intakes, spillways, emergency outlets etc. Tehri dam is an example of such uses.

6.2 Suggestions for Future Study

There is a lot of saving in cost and time if the flood is passed over incomplete earth dam. The experience of such diversions is limited. More data shall be collected to develop it as a standard approach.

Table 5.3.1 Salient Features of Diversion Tunnels

Tunnel	Cross Section	Invert Level (m)		Length (m)	Design Discharge (Cumecs)	
		Inlet	Outlet			
LEFT BANK DIVERSION TUNNELS						
T-1	11 m (HORSE SHOE)	632.0	596.0	1777.710	2180	1/49.38
T-2	11 m (HORSE SHOE)	632.0	596.0	1774.096	2180	1/49.28
RIGHT BANK DIVERSION TUNNELS						
T-3	11 m (HORSE SHOE)	606.0	600.0	1297.559	1900	1/216.26
T-4	11 m (HORSE SHOE)	609.0	603.0	1426.409	1860	1/237.73

Table 1 Rating Curve Caluculations for Tunnel T_3

Appendix-II

D = Diameter fo horse shoe tunnel
d = depth of flow in tunnel
Qc = Discharge when the crictical depth is d
hvc = Velocity head for a critical depth of d

Discharge calculations at different depths in tunnels

dc/D	dc(m)	hvc/D	Velocity Head hvc(m)	Entry Loss Cofficient	Entry Loss (m)	Reservoir Elevation (m) = 606+col.2+col.4 +col.6	Qc/D5/2	Qc (Cusecs) × 104	Qc (Cumecs)
1	2	3	4	5	6=4×5	7	8	9	10
0	0	0	0.0000	0.0000	0.0000	606.00	0.0000	0.0000	0.00
0.1	1.1	0.0351	0.3861	0.0500	0.0193	607.50	0.0879	0.0687	19.46
0.2	2.2	0.0803	0.8833	0.1130	0.0998	609.67	0.3312	0.2590	73..35
0.3	3.3	0.1247	1.3717	0.1800	0.2469	610.92	0.6777	0.5299	150.07
0.4	4.4	0.1702	1.8722	0.2495	0.4671	612.74	1.1148	0.8717	246.87
0.5	5.5	0.2183	2.4013	0.3210	0.7708	614.67	1.6361	1.2793	362.31
0.6	6.6	0.2735	3.0085	0.3950	1.1884	616.80	2.2479	1.7577	497.79
0.7	7.7	0.3443	3.7873	0.4660	1.7649	619.25	2.9702	2.3225	657.75
0.8	8.8	0.4484	4.9324	0.5000	2.4662	622.20	3.8534	3.0131	853.33
0.9	9.9	0.657	7.2270	0.5000	3.6135	626.74	5.1256	4.0078	1135.03
0.92	10.12	0.7371	8.1081	0.5000	4.0541	628.28	5.5077	4.3066	1219.65
0.94	10.34	0.8528	9.3808	0.5000	4.6904	630.41	5.9996	4.6913	1328.60
0.96	10.56	1.0446	11.4906	0.5000	5.7453	633.80	6.7114	5.2478	1486.21
0.98	10.78	1.4742	16.2162	0.5000	8.1081	641.10	8.0892	6.3252	1791.33
0.99	10.89	2.0804	22.9240	0.5000	11.4620	651.28	9.5780	7.4893	2121.01

Ref. For Col. 4 & 8 : Velocity head and discharge at critical depth in horse shoe conduits partly full (USBR, Design of small dams, 1976.)
Ref. for Col. 3 : Hand book of Hydraulics, king.

Right Bank Diversion Tunnel T_3

Length = 1297.559m

Invert level at Inlet: 606m

To Find crictical Slope (Sc)

$D^{8/3} = (11 \times 3.28)^{8/3} = 1.423 \times 10^4$

n = 0.014

(For Cement Concrete)

dc/D	Qc ($\times 10^4$) (Cusecs)	$Qc.n/D^{8/3}S^{1/2}$	Critical Slope (Sc)
0.2	0.259	0.0567	1/495.13
0.3	0.5299	0.1155	1/490.83
0.4	0.8717	0.1854	1/467.36
0.5	1.2793	0.253	1/404.065
0.6	1.7597	0.343	1/393.42
0.7	2.3229	0.42	1/337.86
0.8	3.0131	0.489	1/267.68
0.9	4.0078	0.526	1/177.96
0.92	4.3066	0.529	1/155.88
0.94	4.6913	0.53	1/131.86
0.96	5.2478	0.528	1/104.58
0.98	6.3252	0.521	1/70.092
0.99	7.4893	0.513	1/48.47

Upto dc/D =0.8, the critical slope Sc < Sb hence control point will be at inlet fro all depth $\leq$ 0.8 D for dc/D > 0.8, the critical slope is greater than bed slope and the control point will be shifted to outlet.

Table 1(b)

dc/D	Critical Slope (Sc)	Head Loss, hf = L×Sc	Reservoir Elevation (m) = Rese. Elev. In Table 1 No: 1 -606+hf+600
0.90	1/177.96	7.2913	628.03
0.92	1/155.88	8.3241	630.6
0.94	1/131.86	9.840443	634.25
0.96	1/104.58	12.4073	640.21
0.98	1/70.092	18.5122	653.61
0.99	1/48.47	26.7704	672.05

Table :2 Rating Curve for Tunnel T_3

dc/D	Discharge (Cumecs)	Reservoir Elevation (m)	
0	0	606	
0.1	19.46	607.5	
0.2	73.35	609.67	
0.3	150.07	610.92	
0.4	246.87	612.74	Table-I
0.5	362.31	614.67	
0.6	497.79	616.8	
0.7	657.75	619.25	
0.8	853.33	622.2	
0.9	1135.03	628.03	
0.92	1219.65	630.6	
0.94	1328.6	634.25	Table 1(b)
0.96	1486.21	640.21	
0.98	1791.33	653.61	
0.99	2121.01	672.05	

Table. 3 : Rating Curve for Tunnel T4

Invert level at inlet 609 m

Length: 1426.409m

Invert level at outlet: 603m

Bed Slope: 1/237.73

dc/D ①	Reservoir Elevation =609+col.7 Table 1-606 (m) ⑦	Discharge (Cumecs) ⑩
0	609	0
0.1	610.5	19.46
0.2	612.7	73.35
0.3	613.92	150.07
0.4	615.74	246.87
0.5	617.67	362.31
0.6	619.8	497.79
0.7	622.25	657.75
0.8	625.2	853.33
0.9	629.74	1135.03
0.92	631.28	1219.65
0.94	633.41	1328.6
0.96	636.8	1486.21
0.98	644.1	1791.33
0.99	654.28	2121.01

Upto dc/D =0.8, the critical slope Sc < Sb hence control point will be at inlet for all depth $\leq$ 0.8 D for dc/D > 0.8, the critical slope is greater than bed slope and the control point will be shifted to outlet.

dc/D	Critical Slope (Sc)	Head Loss hf = L×Sc	Reservoir Elevation (m) = Rese. Elev. In Table 1 -606+hf+603
0.90	1/177.96	8.0153	631.76
0.92	1/155.88	9.151	634.43
0.94	1/131.86	10.818	638.23
0.96	1/104.58	13.64	644.44
0.98	1/70.092	20.351	658.45
0.99	1/48.47	29.429	677.71

Table : 4 Rating Curve for Tunnel T4

dc/D	Discharge (Cumecs) FROM Table 3	Reservoir Elevation (m)
0	0	609
0.1	19.46	610.5
0.2	73.35	612.7
0.3	150.07	613.92
0.4	246.87	615.74
0.5	362.31	617.67
0.6	497.79	619.8
0.7	657.75	622.25
0.8	853.33	625.2
0.9	1135.03	631.76
0.92	1219.65	634.43
0.94	1328.6	638.23
0.96	1486.21	644.44
0.98	1791.33	658.45
0.99	2121.01	677.71

Table 5. Rating Curve for Tunnels T1 and T2

Bed Slope of T_1 : 1/49.38 **Bed Slope of T_2 : 1/49.28**

dc/D	Reservoir Elevation (m) (632+Col. 7 in Table 1. - 606)	Discharge (Cumecs)
0	632	0
0.1	633.5	19.46
0.2	635.67	73.35
0.3	636.92	150.07
0.4	638.74	246.87
0.5	640.67	362.31
0.6	642.8	497.79
0.7	645.25	657.75
0.8	648.2	853.33
0.9	652.74	1135.03
0.92	654.28	1219.65
0.94	656.41	1328.6
0.96	659.8	1486.21
0.98	667.1	1791.33
0.99	677.28	2121.01

Table 2 and Table 4 are the rating curves data for T_3 and T_4 with the consideration that the flow within the tunnels is free flow with control point both at inlet and outlet portal. But when the reservoir elevations at 637.50 (water in outlet portal of RBT at elevation 614.00) both T_3 and T_4 will be in submergence conditions. Therefore from Tabl2 2 and Table 4 all the point upto $d_c/D = 0.94$ have been plotted.

In case of tunnels T_1 and T_2, since the tunnels crown level at inlet portal are at elevations 643.00, the flow within the two tunnels remains free flow when the reservoir elevation is at 637.50. So all the point upto $d_c/D = 0.3$, have been plotted and interpolations are made for value for elevation 637.50 from Table 5. Further points will be plotted after considering the submergence conditions as per calculations given after.

Calculation of discharges under submerged condition

Tunnel T_1

Length = 1777.710 m

Coeff. Of roughness n = 0.014

(Cement Concrete)

Friction coeff, $f = \frac{185\,n^2}{D^1/3}$ (for circular conduits)[Ref. USBR, Design of Small Dams, 1974]

Where D is the equivalent dia for circular section

$$D = \frac{11}{0.987} = 11.145\,\text{m} = 36.56\,\text{ft}$$

$$F = \frac{185 \times (0.014)^2}{(36.56)^{1/3}} = 0.01106$$

If V is the velocity at the outlet of tunnels in m/s.

$$\text{Friction loss} = h_f = \frac{fLV^2}{2gD}$$

$$= \frac{0.01106 \times 1777.71 \times V^2}{11.145 \times 2g}$$

$$\text{Entry loss} = \frac{0.5V^2}{2g}$$

$$\text{Total loss} = (1.764 + 0.5)\frac{V^2}{2g} = \frac{2.264\,V^2}{2g}$$

Tunnel T_2 Length = 1774.096 m

$$h_f = \frac{0.01106 \times 1774.096}{11.145} \times \frac{V^2}{2g} = 1.761\,\frac{V^2}{2g}$$

Entry loss = 0.5 $V^2/2g$

$$\text{Total loss} = (1.761 + 0.5)\,\frac{V^2}{2g} = 2.261\,\frac{V^2}{2g}$$

Tunnel T_3

Length = 1297.559 m

$$h_f = \frac{0.01106 \times 1297.559}{11.145} \times \frac{V^2}{2g} = 1.288 \frac{V^2}{2g}$$

Entry loss = $0.5 \frac{V^2}{2g}$

Total loss = $(1.288 + 0.5) \frac{V^2}{2g} = 1.788 \frac{V^2}{2g}$

Tunnel T_4

Length = 1426.409 m

$$h_f = \frac{0.01106 \times 1426.409}{11.145} \times \frac{V^2}{2g} = 1.416 \frac{V^2}{2g}$$

Entry loss = $0.5 \frac{V^2}{2g}$

Loss due to bend = Kb $V^2/2g$

(Radius of bend R_b = 33 m)

Angle of bend = 50^0 K_b = 0.198 [Varshney, R.S., Hydropwer Structures, 1986]

Loss due to bend = 0.198 $V^2/2g$

Total loss = $(1.416 + 0.5 + 0.198) \frac{V^2}{2g} = 2.114 \frac{V^2}{2g}$

Calculation of discharges at Reservoir level 637.5 m

Calculation of discharges

Right bank tunnels T_3 and T_4.

Assuming the tail water level at the out let of the Right bank tunnel 614.0 m

Flow energy head H_T = 637.5 – 614 = 23.5 m

Tunnel T_3

$$V_3 = \sqrt{\frac{23.5 \times 2g}{2.788}} = 12.86 \text{ m/s}$$

$Q_3 = 0.8293 \times 11^2 \times 12.86 = 1290.49 \text{ m}^3/\text{s}$

Tunnel T_4

$$V_4 = \sqrt{\frac{23.5 \times 2g}{3.114}} = 12.168 \text{ m/s}$$

$Q_4 = 0.8293 \times 11^2 \times 12.168 = 1221.07 \text{ m}^3/\text{s}$

Calculation for left bank Tunnels T_1 and T_2.

The discharge corresponding to 637.50 m for tunnel T_1 and T_2 can be obtained by interpolation from table.

Res. Level (m)	Discharge (m^3/s)
636.92	150.07
638.74	246.87

Discharge corres ponding to Res. Elevation 637.50 m

$$Q_1 + Q_2 = 150.07 + \frac{637.50 - 636.92}{638.74 - 636.92} \times (246.87 - 150.07)$$

$$= 180.918 \text{ m}^3/\text{s}$$

$$Q = 1290.49 + 1221.07 + 180.918 + 180.918$$

$$= 2873.4 \text{ m}^3/\text{s}$$

From stage discharge curve at outlet portal of left bank tunnel (Fig. 5.4.2) the water level in the river corresponding to 2873.4 m^3/s is 612.5 m

$\therefore$ water level in river at outlet of RBT = W.L for LBT + 1.5 m

$= 612.5 + 1.5 = 614.0$ m.

Calculation of discharges at Reservoir level 644.0 m

Calculation of discharges.

Right bank tunnels T_3 and T_4

Assuming the tail water level at the outlet of the Right Bank tunnel 619.5 m.

Flow energy head $H_T = 644 - 619.5 = 24.5$ m.

Tunnel T_3

$$V_3 = \sqrt{\frac{24.5 \times 2g}{2.788}} = 13.131 \text{ m/s}$$

$Q_3 = 0.8293 \times 11^2 \times 13.131 = 1317.66 \text{ m}^3/\text{s}$

Tunnel T_4

$$V_4 = \sqrt{\frac{24.5 \times 2g}{3.114}} = 12.424 \text{ m/s}$$

$Q_4 = 0.8293 \times 11^2 \times 12.424 = 1246.78$ m/s

Calculation for Left Bank tunnels T_1 and T_2

Tail water at outlet of left bank tunnel = 619.5 – 1.5 = 618.0 m

$H_T = 644 - 618 = 26$ m

Tunnel T_1

$$V_1 = \sqrt{\frac{26 \times 2g}{3.264}} = 12.50 \text{ m/s}$$

$Q_1 = 100.35 \times 12.50 = 1254.52 \text{ m}^3/\text{s}$

$$V_2 = \sqrt{\frac{26 \times 2g}{3.261}} = 12.51 \text{ m/s}$$

$Q_2 = 100.35 \times 12.51 = 1255.10 \text{ m}^3/\text{s}$

$Q_1 + Q_2 + Q_3 + Q_4 = 5074.06 \text{ m}^3/\text{s}$.

From stage discharge curve at outlet portal of left bank tunnels (Fig. 5.4.2) the water level in the river corresponding to 5074 m^3/s is 617.0 m. Average fall in water level in the river between the outlets of LBT and RBT is taken as 1.50 m.

$\therefore$ water level in river at outlet of RBT = W.L for LBT + 1.5 m

= 618 + 1.5 = 619.50 m

Calculation of discharges at Reservoir level 670.0 m

Calculation of discharges

Right bank tunnels T_3 and T_4

Assuming the tail water level at the outlet of the right bank tunnel 624.0 m.

Flow energy head, $H_T = 670 - 624 = 46$ m

Tunnel T_3

$$V_3 = \sqrt{\frac{46 \times 2g}{2.788}} = 17.99 \text{ m/s}$$

$Q_3 = 0.8293 \times 11^2 \times 17.99 = 1805.51 \text{ m}^3/\text{s}$

Tunnel T_4

$$V_4 = \sqrt{\frac{46 \times 2g}{3.114}} = 17.02 \text{ m/s}$$

$Q_4 = 0.8293 \times 11^2 \times 17.02 = 1708.36 \text{ m}^3/\text{s}$

Calculation for Left Bank tunnels T_1 and T_2

Tail water level at outlet of left bank tunnel = 624 – 1.5 = 622.5 m

$H_T = 670 - 622.5 = 47.5$ m

Tunnel T_1

$$V_1 = \sqrt{\frac{247.5 \times 2g}{3.264}} = 16.897 \text{ m/s}$$

$Q_1 = 100.35 \times 16.897 = 1695.6 \text{ m}^3/\text{s}$

$$V_2 = \sqrt{\frac{47.5 \times 2g}{3.261}} = 16.91 \text{ m/s}$$

$Q_2 = 100.35 \times 16.91 = 1696.44 \text{ m}^3/\text{s}$

$Q_1 + Q_2 + Q_3 + Q_4 = 6906.25 \text{ m}^3/\text{s}$

From stage-discharge curve at outlet portal of left bank tunnels (Fig. 5.4.2) the water level in the river corresponding to 6906 m^3/s is 622.5 m

$\therefore$ water level in river at outlet of RBT = W.L for LBT + 1.5 = 622.5 + 1.5 = 624.0 m

Calculation of discharges at Reservoir level 690.0 m

Calculation of discharges

Right bank tunnels T_3 and T_4

Assuming the tail water level at the outlet of the right bank tunnel 626.0 m.

Flow energy head $H_T = 690 - 626 = 64$ m

Tunnel T_3

$$V_3 = \sqrt{\frac{64 \times 2g}{2.788}} = 21.22 \text{ m/s}$$

$Q_3 = 0.8293 \times 11^2 \times 21.22 = 2129.66 \text{ m}^3/\text{s}$

Tunnel T_4

$$V_4 = \sqrt{\frac{64 \times 2g}{3.114}} = 20.068 \text{ m/s}$$

$Q_4 = 0.8293 \times 11^2 \times 20.068 = 2015.10 \text{ m}^3/\text{s}$

Calculation for Left Bank tunnels T_1 and T_2

Tail water level at outlet of left bank tunnel = 626 – 1.5 = 624.5 m

$H_T = 690 - 624.5 = 65.5$ m

Tunnel T_1

$$V_1 = \sqrt{\frac{65.5 \times 2g}{3.264}} = 19.842 \text{ m/s}$$

$Q_1 = 100.35 \times 19.842 = 1991.19 \text{ m}^3/\text{s}$

$$V_2 = \sqrt{\frac{65.5 \times 2g}{3.261}} = 19.85 \text{ m/s}$$

$Q_2 = 100.35 \times 19.85 = 1992.10 \text{ m}^3/\text{s}$

$Q_1 + Q_2 + Q_3 + Q_4 = 8126.76 \text{ m}^3/\text{s}$

From stage-discharge curve at outlet portal of left bank tunnels (Fig. 5.4.2) the water level in the river corresponding to 8126.76 m^3/s is 624.5 m

$\therefore$ water level in river at outlet of RBT = W.L for LBT + 1.5 = 624.5 + 1.5 = 626.0 m

Calculation of discharges at Reservoir level 710.0 m

Calculation of discharges

Right bank tunnels T_3 and T_4

Assuming the tail water level at the outlet of the right bank tunnel 627.25 m

Flow energy head $H_T = 710 - 627.25 = 82.75$ m

Tunnel T_3

$$V_3 = \sqrt{\frac{82.75 \times 2g}{2.788}} = 24.132 \text{ m/s}$$

$Q_3 = 0.8293 \times 11^2 \times 24.132 = 2421.61 \text{ m}^3/\text{s}$

Tunnel T_4

$$V_4 = \sqrt{\frac{82.75 \times 2g}{3.114}} = 22.8189 \text{ m/s}$$

$Q_4 = 0.8293 \times 11^2 \times 22.8189 = 2289.88 \text{ m}^3/\text{s}$

Calculation for Left Bank tunnels T_1 and T_2

Tail water level at outlet of left bank tunnel = 627.25 – 1.5 = 625.75 m

$H_T = 710 - 627.25 = 83.25$ m

Tunnel T_1

$$V_1 = \sqrt{\frac{83.25 \times 2g}{3.264}} = 22.504 \text{ m/s}$$

$Q_1 = 100.35 \times 22.504 = 2258.27 \text{ m}^3/\text{s}$

$$V_2 = \sqrt{\frac{83.25 \times 2g}{3.261}} = 22.514 \text{ m/s}$$

$Q_2 = 100.35 \times 22.514 = 2259.31 \text{ m}^3/\text{s}$

$Q_1 + Q_2 + Q_3 + Q_4 = 9229 \text{ m}^3/\text{s}$

From stage-discharge curve at outlet portal of left bank tunnels the water level in the river corresponding to 9229 m^3/s is 625.75 m

$\therefore$ water level in river at outlet of RBT = W.L for LBT + 1.5 = 625.75 + 1.5 = 627.25 m