A STUDY OF FOUNDATION TREATMENT OF CONCRETE GRAVITY DAMS - A CASE STUDY OF LAKHWAR DAM USING FINITE ELEMENT METHOD

A DISSERTATION

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By

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CANDIDATE'S DECLARATION

I hereby certify that the work, which is being presented in the dissertation entitled **"A STUDY OF FOUNDATION TREATMENT OF CONCRETE GRAVITY DAMS – A CASE STUDY OF LAKHWAR DAM USING FINITE ELEMENT METHOD"** in partial fulfillment of the requirement for the award of the Degree of Master of Technology in Water Resources Development (Civil) submitted in the department of Water Resources Development Training Centre, Indian Institute of Technology Roorkee, is an authentic record of my own work carried out during a period from July 2002 to December 2002 under the supervision of Dr. B.N. Asthana, Emeritus Fellow, and Dr. R.P. Singh, Professor, Water Resources Development Training Centre, Indian Institute of Technology Roorkee, India.

The matter embodied in this dissertation has not been submitted by me for the award of any other degree.

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SYNOPSIS

Foundation of a dam must be considered as an integral part of the dam, and for safe design it is necessary that it should be given as much an attention as the dam itself. It plays an important part in controlling the stability of a dam entirely.

A good foundation is of ample strength to withstand the weight of the structure and to prevent sliding. It must be tight enough to prevent excessive leakage, uplift must be reduced as much as possible, and discharge from the overflow or outlets must not damage it.

The stability of a concrete gravity dam is controlled principally by the geological structure and geomechanical properties of the foundation material. Knowledge of the characteristics of foundation is, therefore, essential for design of a dam.

Geological investigations are required to determine the general suitability of the site and to identify the structure of the foundation, then geomechanical tests of material are to be carried out.

In most cases, the natural characteristics of the foundation do not meet the requirement of a good foundation. Foundation treatment is, therefore, a must to improve the natural characteristics of the foundation.

Review of the foundation treatments carried out at the existing concrete dams revealed that foundation treatments could be categorized into impermeabilization treatments and treatment of weak zones.

Impermeabilization treatments consist of consolidation grouting, curtain grouting and drainage system. At design stage, the dimensions of these treatments are approached from the available empirical formula or from the previous experiences. The depth of curtain grouting of most dams is generally obtained by USBR formula. Other dimensions, like number of row and space of curtain grouting holes, depth and space of consolidation grouting, number of row, depth and space of *drainage holes are finalized at site depending on site conditions. There is no generally acceptable empirical formula for these dimensions.*

Review of 38 numbers of existing concrete dams revealed that only in 10 dams the depth of curtain grouting provided is within the range of the USBR formula.

In this study, all the above dimensions are approached using the parameter of dam height and type of foundation rock of the existing concrete dam, and then a number of inferences are made.

The treatment of weak zones basically is by backfilling them by concrete. The type of backfilling concrete could be a dental treatment, a deep backfilling concrete, a concrete key/strut or a grid of shear keys. The choice and dimension of the type depend on the characteristics of the foundation and should be decided through the analysis of dam-foundation behavior. This type of analysis is feasible by Finite element method (FEM).

Analysis of dam-foundation behaviour of Lakhwar gravity dam has been carried out using finite element method in this study. The results have revealed that the characteristics of the Slate rock, one of the rock within the foundation for which the modulus of deformation is five percent of that of the dam material (concrete), plays an important role in the deformation of the dam. The existence of shear zone and treatment of it have proved relatively inconsequential.

Further, as treatment of the entire Slate rock to improve the characteristic up to the requirement is likely not feasible as the Slate is closely jointed and grouting is unlikely, hence the type of dam at Lakhwar site should be reviewed and other alternatives could be considered.

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CHAPTER I

INTRODUCTION

1.1. Background of Study

There are very few areas in Civil engineering where the relationship between the foundation and the structure itself is as strong as in area of Dam engineering. The foundation of a dam must be considered as an integral part of the dam and for safe design it is necessary that it should be given as much attention as the dam itself, particularly for those located in complex geological setting. Certainly the foundation is the part of the structure, which is least knowable in advance and least visible after project completion.

The design of dams is largely governed by the geological and lithological structure of the foundation on which they will be set, engineering properties of their rock such as strength and deformation, permeability, resistance to internal erosion, tendency of the foundation material to deconsolidation in case of distressing and others. Designing and construction of dams on complicated foundations, as a rule require the detailed site investigation, examination and additional study of the rock in the excavated foundation pits.

Most of the failures of the dams have occurred because of the failure of their underlying strata. The material underlying the base of a dam must be strong. The cause of the failure of St. France dam, a concrete dam in California, was found to be the presence of conglomerate in one abutment which was weakened after exposure to moisture from the reservoir. Austin dam in Texas failed because the large cavities had been dissolved in its limestone foundation [Garg, S.K., 1976]. Also the failure of Malpasset dam, a concrete arch dam in France, was caused by the break of the foundation due to uplift pressure [ASCE, 1974].

No two foundations are exactly alike. Each foundation presents its own separate and distinct problem requiring corresponding special preparation and treatment. The foundation of concrete dam is usually rock foundation. The rock mass behavior is much different from that of soil. The rock mass contains joints, fissures, foliations, bedding planes, shears and faults.

A good foundation for concrete dams should be able to support the dam without unacceptable deformation, should not permit objectionable leakage through untreated joints or fissures and should not deteriorate with time under sustained reservoir loading. In nature rocks are seldom free from flaws in one form or the other and do require treatment to improve their strength and to render them impermeable by closing discontinuities by effective grouting.

In view of the importance of foundation treatment for concrete dams, 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 foundation condition in designing a concrete dam particularly in foundation treatment.

ii. To summarize various methods of foundation treatment.

iii. To draw the important inferences from the experiences of foundation treatment executed at existing concrete dams

iv. To study and analyze the dam-foundation behavior of Lakhwar gravity dam using Finite Element Method.

1.3. Methodology of Study

The details of methods of foundation treatment of concrete dams are discussed from standard texts and from papers presented in *CBIP, Workshop on investigation and treatment for dam foundations, ICOLD,* and *Water Power & Dam Construction.* The experiences of foundation treatment executed at high dams are reviewed from these literatures, and then the important inferences are made.

Foundation condition of Lakhwar dam is examined. The 2-dimensional stresses and deformations analysis using finite element method is carried out.

Finally, based upon the above study, inferences are made regarding various parameters of foundation treatment and from the analysis of Lakhwar dam.

CHAPTER II

REQUIREMENTS OF A GOOD FOUNDATION FOR CONCRETE DAMS

2.1. Need for Foundation Investigation

The term "foundation" as described herein includes both the valley floor and the abutments. Although the foundation is not actually designed, certain provisions for treatment are made in designs to ensure that the essential requirements will be met.

The stability of concrete dams is controlled principally by the following geomechanical properties of the foundation material:

i. Compressive strength.

ii. Shear strength.

iii. Deformation modulus.

iv. Permeability.

Knowledge of the foundation properties is, therefore, essential for design of the dam. A foundation investigation is then required to examine the geological structure and to determine the physical properties of the existing foundation.

2.2. Foundation Investigation

2.2.1. Objective of investigation

Foundation investigation is an expensive and time-consuming phase of project development. Moreover, it may indicate whether the project is economically and technically feasible or not. Hence, it should be planned and executed so that the probable soundness of the project will be determined as early and as inexpensively as possible.

The objective of foundation investigation of a dam is to provide the data necessary to properly evaluate the geological condition of an existing dam foundation. It is therefore, the term of foundation investigation is frequently referred as geological investigation.

Geological investigations are performed to determine [Japan National Committee of ICOLD,1978]:

i. Nomenclature of rock and classification of soil-bed comprising the ground.

ii. Stratigraphy and geological structure of the ground.

iii. Properties of the ground.

iv. Extraordinary phenomena in the ground.

However, there is no simple rule to determine the extent of investigation, which is necessary in any particular case. The extent of investigation will vary depending on the stage of investigation, but in any case the above-listed items are to be included.

2.2.2. Stages of investigation

To accomplish the above objective, the investigation is generally divided into three stages as described herein [USBR,1974].

a. Reconnaissance stage

It is undertaken primarily to support a decision on whether to proceed with more detailed investigation on the basis of rough data and shortcut studies.

In this stage, the investigation includes a preliminary selection of the site and type of the dam. All available geologic and topographic maps, photographs of the site area, and data from field examinations of natural outcrops, road cuts, and other surface conditions should be utilized in the selection of the site and preliminary evaluation of the foundation.

The amount of investigation necessary for this stage will vary with the anticipated difficulty of the foundation. In general, the investigation should be sufficient to define the major geologic conditions with emphasis on those, which will affect design.

The geologic history of a site should be thoroughly studied, particularly where the geology is complex. Study of the history may assist in recognizing and adequately investigating hidden but potentially dangerous foundation conditions. Core drilling during this stage may be necessary in more complex foundation. Basic data that should be obtained during the reconnaissance investigation, with refinement continuing until the construction is complete, are:

i. Dip, strike, faults and shears.

ii. Depth of overburden.

iii. Depth of weathering.

iv. Joint orientation and continuity.

v. Lithology throughout the foundation.

vi. Physical properties of the foundation material.

b. *Feasibility stage*

It determines the scope, magnitude, essential plan and feature, and the approximate benefits and costs of the project with sufficient dependability to support project authorization or approval for construction.

During this stage, the location of the dam is usually finalized and the basic design data are firmed up. The geologic mapping and sections are reviewed and supplemented by additional data such as new surveys and additional drill holes. The best topography should be used. In most cases, the topography is easily obtained by aerial photogrammetry to almost any scale desired.

The drilling program is generally the means of obtaining the additional data required for the feasibility stage. The program takes advantage of any knowledge of special condition revealed during the reconnaissance stage. The drill holes become more specifically oriented and increased in number to better define the foundation conditions and determine the amount of foundation treatment required.

The rock specimens for laboratory testing during this stage are usually nominal, as the actual decision for construction of the dam has not yet been made. Test specimens should be obtained to determine more accurately physical properties of the foundation rock.

Physical properties of joint or fault samples may be estimated by using conservative values from past testing of similar materials. The similarity of materials can be judged from the cores retrieved from the drilling.

c. *Specification stage*

It supplements the feasibility stage to produce final design data required prior to the preparation of final plans and specifications after authorization or approval has been obtained and construction is imminent. Many of the smaller projects will not require, at the specification stage, any information addition to that already obtained in the feasibility studies. The larger and more difficult projects will often require extensive additional surveys and investigations. However, project size is not the sole criteria with respect to necessity for further detailed studies.

2.2.3. Method of investigation

The method of geological investigation includes [Japan National Committee of ICOLD,1978]:

a. Geological reconnaissance

The ground surface is to be reconnoitered. Observation and measurements of the items specified earlier and the topography are to be made using clinometers, hammers, barometers, tape measures and other similar implements. The stratigraphy and structure of any outcrop are to be recorded. Thus data necessary for knowledge of the general geological conditions is gathered.

Geological reconnaissance is to be reinforced by reference to any other related source of information, such as aerial photographs.

b. Detailed surface survey

Surveys in greater detail are to be made of outcrops, including measurements of the parameters used in numerical analysis of the behavior of a dam and foundation, to enable the geological conditions to be understood with greater accuracy. The locations of the outcrops are recorded by surveying. All outcrops are to be investigated as fully as possible and where access is difficult, temporary plank roads, walkways or other suitable means are to be provided, so that as far as is practicable, the necessary data may be obtained.

c. Excavation survey

Trial holes, trenches and boreholes are to be used in the excavation survey. Trial holes include the digging of horizontal and inclined adits and shafts in the ground to enable the geological characteristics of the foundation site to be investigated. In trenching, geological data is to be gathered from trenches dug just below the ground surface. Boreholes provide various data in the course of boring works and by recovered core, from which the geological conditions may be assessed.

d. *Geophysical survey*

Geophysical survey is to be used to obtain geological information through the elastic or electrical properties of the foundation. The methods, which are frequently used in this survey, are seismic method and resistivity method.

e. Tests

Information on the characteristics of the foundation material is to be obtained by field tests and laboratory tests.

2.3. Geomechanical Properties of Foundation Material

Foundation material as well as concrete material is basic material, which is encountered in design of concrete dam. The main foundation material properties, which must be accurately determined include compressive strength, shear strength, and deformation modulus. The followings are the description of the main geomechanical properties of the foundation material [Sharma, H.D., 1998].

a. Compressive strength

Compressive strength of the foundation rock or the bearing pressure of other foundation materials can be an important factor in determining base thickness of the dam. The dam base should be at a level where the maximum bearing stress is equal to the value of compressive strength of foundation material divided by an appropriate factor of safety.

The value is influenced by rock's texture, particularly by coarseness of grains. The fine-grained sandstones are stronger than coarse-grained sandstones. The value of compressive strength of basalt rocks is over 2,500 kg/cm^2, for fine grained granites is 1,600 kg/cm^2 to 2,500 kg/cm^2, for average sandstones and lime stones is 700 kg/cm^2 to 1,600 kg/cm^2, and for shales, porous sandstones and lime stones is 350 kg/cm^2 to 700 kg/cm^2.

Uniaxial unconfined compression tests are generally carried out to determine the compressive strength of rock samples.

b. Shear strength

Shear strength or shear resistance within a foundation and between a dam and its foundation depends upon the normal load, cohesion and the internal friction in the foundation and on the bond between the concrete and foundation at the dam contact. Experience shows that intact rock possesses cohesive and sliding-friction strength. Therefore its total shear strength can be determined using Coulomb's equation, which describes a linear relationship between shear resistance and normal load. For material other than intact rock, the relationship is often nonlinear.

Since shear movement will take place along joint plane and fissures, the realistic values of shear strength can only be obtained by in situ tests. There are two types of in situ load test, inclined and parallel, which are generally used. Since the dam foundation rocks will be exposed to water on filling of reservoir, it is advisable to keep the rock saturated while testing.

Link had compiled the data of in situ tests on rock mass of 55 dam foundations from 16 countries. The values of rock cohesion range from 0 to 30 kg/cm^2 and sliding friction (tan Ø' or f') from 0.30 to 2.70. In concrete to rock interfaces, the adhesive shear resistance (cohesion) ranges from 1 to 30 kg/cm^2 and sliding friction from 0.30 to 2.0. Eliminating the very low and scattered values belonging to the soft and highly weathered rocks, the majority of tests yielded an acceptable value of sliding friction from 1 to 1.30.

The practice of pressure grouting the foundation rock to a certain depth has been found to improve the sliding resistance to a moderate extent. The benefit from cement grouting mainly tends to be an evening out of deformation and an increase in deformation modulus.

c. *Deformation modulus*

A concrete dam is considered to be homogeneous, elastic and isotropic but its foundation is, in general, heterogeneous, inelastic (rigid) and anisotropic. This variation in the foundation can affect the distribution of loads within the dam. Thus, the foundation deformation characteristics should be evaluated over the entire extent of the dam contact.

The reaction of the foundation to the loads from the dam controls, to some extent, the stresses within the dam. Conversely, the response of the dam to the external loads and the foundation determines the stresses on the foundation. The proper determination of the interaction between the dam and foundation requires an accurate knowledge of the deformation characteristics of the foundation.

Deformation modulus is defined as the ratio of applied stress to elastic plus inelastic strain and is used to define the deformation characteristics of the foundation. It differs from elastic modulus in that an elastic modulus is the ratio of stress to elastic strain only.

Jacking tests are usually employed to determine the modulus of elasticity of the foundation rock in term of the Boussinesq equation, which provide a relationship between this modulus and measured load and displacement. Since these equations are not applicable to a discontinuous system, they give a modulus of elasticity for an equivalent continuum. The obvious question is whether the deformation modulus obtained by a small jacking test can be applied to the full-scale dam foundation.

Furthermore, experimental relationship between load and deformation for rocks are generally nonlinear. In other words, several values of deformability can be inferred from any one test depending on the magnitude and the sign (loading or unloading) of the applied load. In fact, these non-linier curves can be used as an additional identification index for the rock mass.

Cylindrical jacking tests are usually performed in adits, but they can also be run at the surface if the load reaction is provided by deep anchors. A number of different arrangements have been suggested and discussion as to the most suitable type of equipment is continuing, i.e. small load area and high stress, or large load area and low stress.

The latter alternative is more expensive but probably closer to the conditions that will apply once the full-scale structure is built. In fact, the crucial point of this argument is the question of the importance of scale effect on deformability.

Borehole jack (dilatometers) have been developed in several countries and have the advantage of being capable of application at depth within the rock mass. Although the stress field is entirely different from that developed by ordinary jacks, the same sort of discussion applies here as for adit tests. It would be seem however that the walls of an adit driven by blasting. This is a point in favor of borehole measurement.

The magnitude of the foundation deformation modulus is not necessarily a critical consideration. Gravity dams have been successfully constructed on low modulus foundation material such as siltstone or clay stone. The critical consideration is the variation of deformation modulus across the foundation. Abrupt change in deformation modulus can result in differential movements, which can possibly cause cracking of the dam. Therefore, proper identification and mitigation measures are required such as excavation, grouting and replacement of low modulus zone with dental concrete, etc.

The deformation modulus of rock of 50,000 kg/cm^2 and greater is very good for high concrete dam foundation, between 20000 kg/cm^2 and 50,000 kg/cm^2 is almost good, between 5000 kg/cm^2 and 20000 kg/cm^2 is bad and below 5,000 kg/cm^2 very bad for high concrete dam foundation [Singh, B., et al, 1994].

d. Permeability

Permeability will have a direct bearing on the design of the dam and can represent a significant cost of the structure. Permeability studies conducted for a dam foundation investigation will be described in detail in discussion of "foundation grouting".

2.4. Influence of Foundation Conditions in Design of Concrete Dam

The selection of the best type of the dam for a particular type calls for thorough consideration of the characteristics of each type. It relates to the physical features of the site. One of the important physical features is foundation condition of the dam site. The importance of the foundation conditions in selecting the types of concrete dam is discussed herein [Golze, A.R., 1977].

Most foundation materials are acceptable for consideration of a gravity dam under 15 m high and with a difference between head water and tail water less than 6 m. Even silt, clay, fine sand or gravel foundations can be used. However, tests should be made of bearing values as well as of consolidation characteristics. Generally, sound rock foundations of igneous, metamorphic and sedimentary rock types are generally considered the most satisfactory for *gravity dam types* having a head water – tail water differential of more than 6 m or 15 m high because they posses high bearing capacity and good resistance to erosion and seepage. The presence of faults and shears does not necessarily preclude a site, although the freer the site is of major faults and shears, the better it is. Materials which decompose or deteriorate upon exposure to water, atmosphere, or pressure are questionable foundation components and should be carefully evaluated before a gravity dam is considered.

Magnitude of deformation modulus of the foundation rock of gravity dams is not a critical consideration in the design as dams have been made on rocks of low deformation modulus after necessary treatment. For foundation other than rock, settlement and deformability characteristics should be investigated before consideration of a low gravity dam. Compressive strength or bearing capacity and shear strength of foundation materials are very important factors in determining the suitability of a site for a gravity dam. If the shear strength of the foundation material is low, an increase in volume of concrete or of foundation treatment to improve the shearing resistance may preclude the use of gravity dam under such circumstance.

Gravity dams are suitable for most canyon width and shape particularly for wide site with relatively flat canyon floor. In general, they are likely to be economical for a valley with the ratio of width between abutments to height of more than 6.

A valley where the ratio of width between abutments to height is relatively small (3 or less) and the abutments are solid rock capable of resisting arch thrust, a thin *concrete arch dam type* is adaptable to this site. However, an arch dam can be designed to give good economy for the wider site. Greater the ratio of width to height, greater the thickness of the arch dam approaching that required for a gravity dam.

The rock foundation for arch dams should be sound and durable to provide a satisfactory foundation for the life of the structure. Deformation modulus of 500,000 psi is satisfactory for an economical design of this type. The increased deformability of such a foundation may require added thickness at the concrete to rock contact and consequently lead to a greater volume of concrete. If a high dam is being considered, compressive strength of the foundation is an important consideration. A foundation with relatively low strength material may require an increase of the thickness of the dam near the contact of concrete and the rock to distribute forces from the structure over a larger area of foundation.

In a situation where the rock foundation is not strong enough to support the load of a gravity dam and the abutments are inadequate to withstand the thrusts from an arch dam, the *arch-gravity dam type* is considered. This dam type is designed in such a case to suitably apportion the load between the foundation and the abutments and in accordance with their load-bearing capacities.

The valley having gentle slope is the most desirable for *buttress dam type*. Such conditions permit more desirable connection between the valley and the dam. Generally, buttress dam should be considered only when rock foundations are available. However, low buttress dam can be constructed on poor rock or earth foundations if footing slabs are used to keep bearing pressures within allowable limits.

Buttress dams have an advantage over gravity dams where uplift pressures are concerned. The hydrostatic pressure or uplift at the base of the dam is relieved between the buttress, and small remaining uplift pressure on the buttress does not materially affect the stability of the structure. However, relatively impervious foundations are desirable because excessive seepage losses may occur over the comparatively short path of water travel under the narrow dam.

2.5. Foundation Treatment

2.5.1. Foundation shaping

If the canyon profile of a dam site is relatively narrow with steep sloping walls, each vertical section of the dam from the center towards the abutments is shorter in height than the preceding one. Consequently sections closer to the abutment will be deflected less by the reservoir loads and sections closer toward the center of the canyon will be deflected more. Since most gravity dams are keyed at the contraction joints, the result is a tension effect in a dam that is transmitted to the foundation rock.

A sharp break in the excavated profile of the canyon will result in an abrupt change in the height of the dam. The effect of the irregularity of the foundation rock causes a marked change in stresses in both the dam and the foundation, and in stability factors. For this reason, the foundation should be shaped so that a uniformly varying profile is obtained free of sharp offsets or breaks. Shaping is carried out by excavating the entire area to be covered by the dam to firm rock capable of withstanding the loads imposed by the dam and appurtenance structures.

Considerable attention should also be given to blasting operations so as not to shatter the otherwise sound rock. Much too often excavation is carried out by heavy blasting using big explosion charge. It should be stressed, however, that the final result can be detrimental for the quality of the foundation or stability of the slopes and may cost much in remedial measures such as consolidation grouting or reinforcement. Heavy blasting opens up joints (either existing or potential) in the rock mass behind the bottom and walls of the excavation, resulting in:

i. Increased deformability.

ii. Increased permeability.

iii. Increased grout takes.

iv. Decreased stability of excavation slopes.

The best answer to this problem is to limit the unit charges, use smooth or control blasting techniques in rock excavation [Londe, P., ICOLD, 1993].

The excavated surface of the rock should be rough and have protrusions to improve bonding. It generally appears as horizontal in the traverse (upstream-downstream) direction. However, where an increased resistance to sliding is desired, particularly for structures founded on sedimentary rock, the surface can be sloped upward from heel to toe of dam.

2.5.2. Treatment of faults and weak zones

Problems and treatments of faults and weak zones in dam foundation are discussed herein [Sharma, H.D., 1998].

a. Problems due to faults

Faults and weak zones exist to some extent in most of rock formations. Their size, continuity and orientation are important factors in determining the suitability of a foundation for any dam. The usual problems associated with the occurrence of faults in a dam foundation are:

i. Inadequate sliding resistance.

ii. Overstressing.

iii. Differential movements.

b. Treatment of shallow faults

In case the orientation of fault is of low-angle, the usual treatments are:

i. Excavating out the weak material and founding the dam on the excavated surface.

ii. Providing shear keys by excavating drift and backfilling with concrete.

In case the orientation of faults is of high-angle or steeply inclined, the usual treatment is by trenching out the weak material then backfilling with concrete (concrete plugging). The function of this concrete plug is to provide support to the bridge portion of the dam and transfers the stresses to the adjacent rock on either side through interfacial action. The depth of plug has to be such that the stresses in the plug are compressive and the contact stresses along the rock-plug interfaces are within the permissible limits, and there is no excessive deformation.

c. Treatment of deep fault

Very often the exploratory drilling or final excavation uncover faults, seams, and shattered or inferior rock extending to such depths that it is impracticable to attempt to clear such areas out entirely. These conditions require special treatment in the form of removing a portion of the weak material and backfilling the resulting excavations with concrete. This procedure of reinforcing and stabilizing such weak zones is frequently called *dental treatment.*

Theoretical studies have been made to develop general rules for guidance as to how deep transverse seams should be excavated. These studies based upon foundations and stress at Shasta and Friant Dams, have resulted in the development of the following approximate formulas for determining the depth of dental treatment:

$$D = 0.00656\ B\ H + 1.526 \qquad \text{for } H > 46 \text{ m, and}$$

$$D = 0.30\ B + 1.524 \qquad \text{for } H < 46 \text{ m}$$

Where:

H = height of dam above general foundation level (m)

B = width of weak zone of rock foundation (m)

D = depth of excavation of weak zone below surface of adjoining sound rock (m). In clay gouge seams, D should not be < 0.10 H.

However, the final depth of the excavation is obtained by analyzing the dam-foundation behavior.

2.5.3. Foundation grouting

a. Objective of grouting

The principal objectives of grouting in a rock foundation are to establish an effective barrier to seepage under the dam, and to consolidate the foundation by filling any opening, which could conduct water.

Spacing, length, and orientation of grout holes and the procedure to be followed in grouting a foundation are dependent on the height of the structure and the geologic characteristics of the foundation. Since the characteristics of a foundation will vary for each site, the grouting plan must be adapted to suit field conditions [USBR,1976].

b. Consolidation grouting

In general, there will be cracks, joints and seams within the foundation. Also some loosening of rock may occur during excavation due to blasting although the specification may restrict it to above 0.50 m above the final excavated surface. Consolidation grouting is a low pressure grouting (utilizing grout holes arranged in a pattern or grid) to fill voids, fracture zones and cracks at or below the surface of the excavated foundation by shallow holes to consolidate the foundation rock and make it more impermeable so as to prevent erosion of the infilling material in this zone of maximum seepage gradients. These holes are called "B" holes [USBR,1976].

The consolidation grouting has two main effects in respect of strengthening the foundation. Firstly, it reduces the irrecoverable part of the foundation deformation, and secondly it increases the rock modulus of elasticity. These two effects apply to groutable rocks with open fissures. The main advantage gained by consolidation grouting is to minimize the variation of deformation in different parts of the foundation.

"B" holes are generally drilled normal to the foundation surface unless it is intended to intersect known faults, shear zones, fractures, joints or cracks. Drilling and grouting are usually done from excavated surface, although in some cases of abutments consolidation, have been done from top of concrete placements in the dam to avoid slabbing of the rocks.

Grouting should always be done in a splitting sequence. The first drilled holes should be widely spaced and then a series of intermediate holes are drilled and grouted in sequence. Frequently, some portions of the foundation may be extensively jointed and require grouting while other portions are relatively tight. Usual practice is to continue splitting until the last drilled hole receives only small amounts of grout, and there is reasonable assurance that all seams, fractures and voids have been sealed.

Generally, for structure of more than 30 m high, it is usual to drill primary holes to depth of 6 m to 15 m depending on the height of the structure and local conditions, and to some extent in a general pattern with spacing from 3 m to 8 m. For dams less than 30 m high and depending on local conditions, the grouting is restricted to the area of the heel of the dam. In this case the upstream line of holes is located at or close to the heel of the dam to prevent leakage of grout from high-pressure curtain grout holes drilled later in the same area. In region where there may later be relatively high stress concentration, the grouting pattern should be spread to consolidate a bulb under the dam [Sharma, H.D., 1998].

According to Indian standard, IS 6066: 1994 for *Pressure grouting of rock foundations in river valley projects – recommendation (second revision)*, the choice of pattern of holes, for consolidation grouting depends on whether it is necessary to wash and jet the hole systematically. When washing has to be carried out a hexagonal pattern would be preferred as this admits for flow reversal. When systematic washing and jetting are carried out to remove all soft material in seams, it is generally unnecessary to use a primary and secondary system of holes.

When it is desirable to test the efficacy of consolidation grouting by comparing the grout absorption in primary and secondary holes, a rectangular or square pattern of holes would be preferred. This is generally the case when the joints are irregular and relatively free from infilling or it is not necessary to remove the material filling the joints.

Consolidation grout pressures vary widely and are dependent in part on the characteristics of the rock such as strength, tightness, joint continuity, stratification and the height of the rock above the stage being grouted. Grout pressures as high as practicable but which, as determined by trial, are safe against rock displacement, are used in grouting. These pressures may vary from a low of 0.6 kg/cm^2 to high range of 5 kg/cm^2 to 7 kg/cm^2 [Sharma, H.D., 1998].

c. *Curtain grouting*

Construction of a deep grout curtain near the heel of the dam to control seepage is accomplished by drilling deep holes and grouting them using higher pressure. These holes are identified as "A" holes when drilled from a gallery [USBR,1976]. Curtain grouting has now attained a high degree of efficiency with the advances made over recent years in the rheology of the material to be treated and design mixes suitable for different types of foundation, together with improved grouting procedures. In some cases, grouting can be advantageously replaced by cast in place concrete diaphragm walls in the foundation. The two systems are sometimes combined, with a diaphragm wall from the surface, continued in depth by a grout curtain.

The grout holes are angled in an upstream direction or parallel to dam axis. Horizontal grout holes are considered for abutments and are integrated into the grout curtain when conditions indicate that they will add to grouting effectiveness. The alignment of grout curtain should be such that the base of the grout curtain will be located on the vertical projection of the heel of the dam. If the holes are drilled from a gallery that is some distance away from the upstream face, the holes may be inclined as much as 15° upstream from the plane of the axis. If the gallery is near the upstream face, the holes may be vertical. When the holes are drilled from upstream fillet, they are usually vertical or inclined downstream. Characteristics of the rock seams and direction of open joints also influence the amount of inclination.

According to Indian standard, IS 11293 (part 2): 1993 for *Guidelines for the design of grout curtains,* the most common practice of grouting inclination is to drill holes inclined towards upstream at 5° to 10° with the vertical.

Uniformity of the foundation can be of assistance to the layout of grout holes. Irregular foundation may require the holes to be at various inclinations. The weak features may require intensive localized grouting. If the material in the joints is liable to be removed by seepage, the grouting may need to be more intensive than otherwise to eliminate seepage in the piping prone areas.

To facilitate drilling, pipes of 0.60 m minimum length are embedded in the floor of gallery or in the upstream fillet. When the structure has reached an elevation that is sufficient to prevent movement of concrete, the holes are drilled through these pipes into the foundation. Grouting in the valley should proceed from foundation floor to abutments.

Although the tentative grouting plan may indicate holes to be drilled on 3 m spaces of center to center, the usual procedure is, first to drill and grout the primary holes approximately 12 m apart or as far apart as necessary to prevent travelling of grout material from one hole to another hole which has been drilled but not yet been grouted. Also leakage into adjacent contraction joints must be prevented by prior grouting of those joints. The secondary holes are then drilled and grouted between the primary holes. Drilling and grouting of additional intermediate holes, and splitting the spaces between the completed holes, will continue until the amount of grout received by the last group of intermediate holes (secondary and tertiary) indicates that no further grouting is necessary.

According to Indian standard, IS 11293 (part 2): 1993, the usual practice is to try a widely spaced system of primary holes at a spacing of 6 m to 8 m, followed by secondary and tertiary holes at a progressively smaller spacing till the desired results are obtained. However, hole spacing less than 1 m should be avoided.

When the reservoir is filled and a concrete dam is placed under water load, there is a tendency for development of tensile stresses and formation of cracks in the heel area of the dam foundation. It is wise, therefore, to locate the drilling and drainage gallery in the dam following, and not far distant from it, the foundation contact. Horizontal extensions of this gallery should also be driven into the abutments. The gallery and its extensions will permit regrouting if the grout curtain is ruptured or proved to be inadequate.

d. Depth of grout curtain

The depth of grout curtain ideally is a function of both hydraulic head and the characteristics of the rock. For this reason, the recommended practice is to augment the geologic and the permeability data obtained during the design stage, with the initial grout holes made during construction and referred to as exploratory holes. Ordinarily, the exploratory holes would be located at about 40 m spacing along the line of grout curtain and taken to a depth of about ⅔ H to H, where H is the hydraulic head at the point subject to a minimum depth of 50 m. Water absorption tests (Lugeon type) are then carried out at these holes [Sharma, H.D., 1998].

On the basis of results obtained from the exploratory grout holes and the previous design drillings, the depth of primary holes would be selected for each reach of grout curtain. In a hard, dense foundation, the depth may vary from 30% to 40% of the head. In a poor foundation, the depth will be more and may reach as deep as 70% of the head. During the progress of the grouting, local conditions may determine the actual or final depth of grouting.

The USBR formula for the depth of grout curtain is:

$$D = \tfrac{1}{3}H + C$$

Where: D = depth of grout curtain (m)

H = height of the dam (m)

C = variable constant ranging from 8 m to 25 m.

According to Indian standard, IS 11293 (part 2): 1993, the following empirical criteria may be used as a guide, which is based on on going practice:

$$D = \tfrac{2}{3}H + 8$$

Where: D = depth of grout curtain (m)

H = height of reservoir water (m)

Supplementary grouting may also be required after the water load has come on the dam and observations have been made of the rate of seepage and the accompanying uplift. For high dams where foundation galleries are located at a relatively long distance from the upstream face, "A" hole grouting may be augmented by a line of "C" holes, drilled from the upstream face of the dam and inclined downstream in order to supplement the main grout curtain. The depth of these "C" holes is usually 22.50 m and their spacing is usually the same as for the "A" holes. The supplementary grout curtain formed by grouting this line of holes serves as an upstream barrier for subsequent "A" hole grouting, permitting higher "A" hole grout pressures with less chance of excessive upstream grout travel.

e. *Spacing of holes and number of row of curtain grouting*

The efficient curtain is achieved by applying the split hole method [Japan National Committee of ICOLD-1978]. The method is described in 6th paragraph of article c of this subchapter. Based on water test or grout take, the spacing between the 2 holes should be halved by additional holes as required. The additional expense is small as compared to the cost of repairing a curtain with gaps in it. A final small spacing is particularly required in erosive rocks, such like sandstone.

In the case of karstic formations, the only answer is a very close initial spacing, so as to increase the probability of crossing dissolution channels. For that specific purpose, cheap small diameter percussion holes can be drilled in a first stage. Where they meet a cavity in the ground, larger diameter holes are drilled for injecting mixes of high viscosity.

For any type of rock, the best arrangement is a curtain in one row of holes [Sharma, H.D., 1998]. The only exception is perhaps the case of karstic formations with clayey infilling, which may require a several row treatment locally. The same case with this exception is when applying the treatment of fault by grouting. The grout pressure used is lower in the outer rows and higher in the inner ones. The purpose of the outer rows is to prevent grout escaping over long distance.

f. Grouting pressure and rate of grouting

If the grouting pressure is increased, then (i) the arriving distance of grout increases to make it possible to increase the spacing of the grouting holes, (ii) grout can be injected even into fine cracks, and (iii) excess water can be removed from the injected grout. However, if the grouting pressure is too high, harmful displacement may be given to rocks or the grout may be injected into an unnecessary range. It is normally said to be reasonable to set the grouting pressure of rocks and the pressure is increased as the depth becomes larger until a predetermined depth is reached. The absolute value of the pressure varies between dams, depending upon the characteristics of rocks.

In the case of consolidation grouting for relatively shallow portions of the foundation, the pressure of 2 kg/cm^2 to 5 kgf/cm^2 is used for the first section of depth and of 3 kgf/cm^2 to 7 kgf/cm^2 for the second section [Shimizu, S., 1985].

The rate of grouting is not so important as long as the maximum grouting pressure is specified and it will be more rational to control the rate of rise in pressure. However, the control of the rate of rise in pressure is complicated and thus, the rate of grouting is normally controlled. The rate of grouting is controlled to 3 l/m per minute for hard rocks and to 2 l/m per minute for soft rocks.

g. Curtain grouting criteria

A.C. Houlsby [ICOLD, 1985,Q58,R60] wrote that curtain grouting is never uniform. Variations come from geological factors as well as from inevitable non-uniformity of grout penetration from hole to hole. The aim in curtain construction is to ensure that the weaker parts are not worse than the target standard of permeability as set for that area of the curtain. There are inevitably other sections where the curtain is stronger than really required and in the interest of economy, wastefully grout takes at these can be minimized by appropriate modifying injection techniques in relevant holes. Assessment of the weaknesses and strengths of a curtain and the implementation of consequent remedial action requires the individual examination of every grout application. It does not suffice to merely use a statistical average of a group holes. This can mask the extremes and leave weakness uncorrected.

The original concept of criteria advanced by Lugeon in 1933 suggested that for concrete dams, grouting is needed wherever the foundation is more permeable than 1 lugeon unit for dams over 30 m high and 3 lugeons for the dams lower than this. Lugeon unit (LU) is a unit in grouting practice. It is a measure of rock permeability or hydraulic conductivity obtained from pump-in water tests and is defined as being a water loss of 1 liter per meter of the hole per minute at a pressure of 10 bars (1 MPa).

F.K. Ewert [Water Power &Dam Construction, January 1992] wrote that in the course of the last 15 years or so, doubts have arisen as to whether this Lugeon criteria is the right measure, or whether it is too stringent. Inconsistent observations have been made repeatedly, causing doubts in the reliability and, thus, the expediency of this criteria.

Subsequent worldwide experience and critical review of the issue has led to relaxation of the criteria. Table 2.5.1 shows the revised criteria recommended by A.C. Houlsby. It is the curtain grouting criteria for the degree of watertightness to be achieved in case of concrete dam. The criteria mainly refers to the top stages of holes, lower stages may be more permeable taking cognizance of the longer seepage paths through them. C. Kutzner [ICOLD, 1991,Q66,R18] suggested a modified curtain grouting criteria, shown on table 2.5.2, taking into consideration the flow conditions in the water absorption (Lugeon) test.

h. Grout mix

In the final state, a grout must have sufficient strength to remain in position against the hydrostatic pressure to which it is exposed within the treated material. It must also be resistant to aggressive water. However, while it is being injected it must be a fluid of low stiffness to promote penetration. Substances meeting these two requirements are relatively rare [Japan National Committee of ICOLD-1978].

The cement-based mix is still the most widely used grout for rock treatment. The important leading parameters determining the properties of the mix are cement-water ratio and admixture content (bentonite). These parameters should be associated with grouting pressure. With respect to grouting pressure, grouts of thick consistency are now injected at higher pressure. Nowadays, there is a trend in America and Europe to a consensus on the need to open the joints in finely jointed rocks by the action of grouting pressure. This makes it essential to correlate pressure and grout take, with higher pressures being used in the holes grouted last, which will take less grout than the earlier ones.

Today, cement-rich mixes (cement-water ratio > 1.50) are used, which are almost completely stable [Japan National Committee of ICOLD-1978].

2.5.4. Foundation drainage

Grouting and drainage are two mutually complementary techniques to control seepage forces in dam foundations. Grout curtain with one or more lines of holes are designed to cut off seepage flow, and in theory, they would have to withstand full hydrostatic pressure on the upstream side because no water could pass through. There would thus be no danger to the foundation stability from seepage forces downstream of the curtain. However, in practice, there is always some leakage through the curtain as the grout cannot penetrate into the fine cracks or force out pervious sandy infilling from the large ones. Performance is also affected by local defects in the curtain [Sharma, H.D., 1998].

Seepage flow through or below the grout curtain can be intercepted by provision of drainage holes downstream of the grout curtain. The interception of seepage through drainage holes reduces uplift pressures acting on the base of the dam and within the foundation. Thus it is the drainage that is primarily effective in controlling the seepage pressure and improving the stability of the dam and its foundation, and a grout curtain is built to fill in large voids and other openings in the rock so as to cut down leakage to an acceptable value and thus reduce load on the drainage system.

To be effective, the drainage holes should not be shadowed by the grout curtain so that they are able to intercept seepage water from the flow channels through the foundation. To ensure this, the drainage holes are generally inclined by about 10° from the vertical to the downstream.

According to Indian standard, IS 10135: 1985 for *Code of practice for drainage system for gravity dams, their foundations and abutments (first revision)* the size, spacing and depth of drainage holes are assumed on the basis of physical characteristics of the foundation rock, foundation condition and depth of storage of the reservoir. The diameter of the hole is generally NX drill-bit size which is 75 mm. The spacing of the holes may be kept as 6 m center to center. The depth of the holes may be kept between 20% and 40% of the maximum reservoir depth, and between 30% and 75% of the curtain grouting depth for preliminary design. The actual spacing and depth may be determined on the basis of geological conditions.

Drainage holes should be drilled after all foundation grouting have been completed within a minimum horizontal distance of 15 m. The drainage holes shall be drilled through the drainage gallery, through previously installed metal pipe extending down to the foundation rock. Additional drainage holes or curtain grouting shall be provided, if uplift pressures higher than design values are observed. After drilling, the pipes shall be taken off at a T-joint and let to the gutter of gallery.

The upstream face of the gallery shall be located at a minimum distance of 5% of the maximum reservoir head or 3 m from the upstream face, whichever is more. A supplementary drainage gallery is sometimes provided towards the toe. The main aim of this gallery is to collect seepage water from foundation and the body of the dam. Besides, it provides space for drilling and grouting the foundations and inspection of the dam structure.

Besides from the foundation drainage gallery, the drainage holes may be drilled through tunnels in the foundation and abutments. Spacing and depth of the holes shall depend on geology condition.

James A. Rhodes and Norman A. Dixon (ICOLD,1976,Q45,R5) wrote that regarding the influence of geology on drainage system, with an idealized rock-type, the permeability characteristic of the drained rock mass should largely control the hydrostatic pressure gradient from regions of high head at the heel, reduced head at the gallery location and approximately tail water at the toe of the dam. Granite rocks are usually more difficult to drain than sandstones, for example, because of the absence of bedding planes and lower mass permeability.

Generally it can be concluded that jointing and fracturing comprise the inherent permeability properties of the rock itself, and that the selection of drain holes spacing depends upon the result of bore holes pressures tests as well as engineering geology judgment. Sedimentary rock foundations are usually more prone to greater leakage thereby allowing wider drain holes spacing.

Table 2.5.1: Curtain Grouting Criteria by A.C. Houlsby
(Permissible values of water absorption in the rock foundation of dams)

General Case	Curtain Standard (Lugeon)
Concrete dams (gravity, arch, buttress):	
- Single row curtain	3 to 5
- Multiple row curtain	5 to 7
Embankment dams (narrow core earth/rock fill):	
- Single row curtain	3 to 7
- Multiple row curtain	5 to 10
Embankment dams (wide core earth/rock fill and membrane faced):	
- Single row curtain	5 to 10
- Multiple row curtain	7 to 15
Exception	
All types of dam if foundation contains material able to be removed by seepage:	
- Single row curtain	3
- Multiple row curtain	4
All types of dams if water lost by seepage is sufficiently valuable to warrant considerable expenditure to stop it, or is environmentally hazardous:	
- Single and multiple row curtain	1 to 3

Reference : ICOLD,1985,Q58,R60

Table 2.5.2: Curtain Grouting Criteria by C. Kutzner
(Permissible values of water absorption in the rock foundation of dams)

Unit : Lugeon

Permeability condition of rock	Quasi-isotropic rock		Extremely anisotropic rock			
Flow of seepage	Free[1]		Free[1]		Partly blocked[2]	
Absorption test	Laminar	Turbulent	Laminar	Turbulent	-	-
Seepage investigation	-	-	-	-	Not made	Favorable
Homogeneous investigation	10 to 15	8 to 12	10 to 15	8 to 12	< 20	< 40
Homogeneous embankment (long seepage path)	8 to 12	5 to 8	8 to 12	5 to 8	< 15	< 30
Rockfill dam/earth core (medium sepage path)	5 to 8	3 to 5	5 to 8	3 to 5	< 12	< 20
Rockfill dam/membrane sealing (short seepage path)	5 to 8	3 to 5	5 to 8	3 to 5	< 12	< 20
Concrete dam (uplift reduction by drainage)						
Special cases :						
Risk of erosion	3 to 5	1 to 3	3 to 5	1 to 3	< 5	< 10
Risk of solution	3 to 5	1 to 3	3 to 5	1 to 3	< 5	< 10
Risk of environmental defects	3 to 5	1 to 3	3 to 5	1 to 3	< 5	< 10
Risk of unacceptable loss of water	3 to 5	1 to 3	3 to 5	1 to 3	< 5	< 10
Lower limit of groutability by use of cement (kg/m)	30	30	30		30	

1. Free : Strike of permeable layers is parallel to valley
2. Partly blocked : Strike of low permeabiliy is across the valley

Reference : ICOLD,1991,Q66,R18

CHAPTER III

REVIEW OF FOUNDATION TREATMENT OF EXISTING DAMS

3.1. General

In this chapter, the foundation treatments, which were carried out during the construction of 10 existing concrete dams in various countries, are reviewed. The dams to be reviewed are those in category of high dam and the type of foundation material is partly similar to that of Lakhwar dam, the 192 m high concrete gravity dam, for which foundation treatment analysis has been made in this study. In addition, the special types of foundation treatment of 4 concrete dams are also reviewed.

3.2. Foundation Treatment of Existing Dams

3.2.1. Sardar Sarovar dam

a. Project feature

Sardar Sarovar project envisages construction of a dam and power complex on the Narmada river near village Navagam in Bharuch district and canal system to create irrigation potential of 1,800,000 ha in Gujarat. It will also provide water for domestic and industrial use.

Sardar Sarovar dam now being constructed, will be a gravity mass concrete structure rising to a height of 163 m above the deepest foundation level, 1,210 m long at the crest and will require 7 million cu.m of precooled concrete.

b. Foundation conditions

The geology of the Indian peninsula comprises Deccan volcanism. The lava flows spread over the cretaceous sedimentary sequence (Bagh series) which are predominantly limestone and sandstone with some indurated shales and covered a large area extending into state of Maharashtra, Gujarat, Madya Pradesh, Andhra Pradesh and Karnataka. The lava flows are horizontal to sub-horizontal and have formed prominent plateaux. In many cases individual flows are separated by a layer of agglomerate or a "Red Bole" and create problems for the dam foundations because it is inherently weak. The individual flows of basalt are as thick as 56 m. The flow contact may be the tight or open with the weakest members being the agglomerate and red bole.

The geological investigation for the project commenced as far back as year 1947, since then complete geological investigations have been made for the dam site, including alternative sites considered so far and other appurtenant works.

The dam is mainly founded on basalt rock. The lava flows are 10 m to 15 m thick on average. Foundation investigations revealed the presence of a reverse fault in the deepest river portion. The fault crosses the dam below the spillway blocks making an angle of 70° with the dam axis and dipping about 60° to 70° with the horizontal towards the right bank. The average width of the fault is about 8 m. On the right side of the fault, the basalt is underlined by sedimentary rocks of argillaceous sandstone, quartzitic sandstone, sedimentary breccia and limestone. On the left of the fault zone is basalt consisting of different lava flows. The dam site is criss-crossed by number of dolerite dyke intrusions with the result that the rock in the foundation is fractured.

The modulus of deformation of basalt is 80,000 kg/cm², whereas that of the fault zone material is 8,000 kg/cm².

Investigations further revealed the presence of a thin weak layer of red bole (crushed basalt with very little clay content) between the two lava flows well below the foundation level of the dam on the left of the fault zone, and two layers of weak and friable argillaceous sandstone in between two competent rock masses on the right of the fault zone.

c. *Treatment of faults zone*

The treatment of riverbed faults was taken up in the year 1980 and completed in the year 1984. It involved 256,000 cum of precooled concrete with placement temperature of 9°C to 12°C and 53,000 tone of reinforcement steel. Concrete plug has been provided for the faults zone. The details of the treatment as shown in Figure 3.2.2 are as follows:

i. Excavation of trench of width 12.50 m (assumed width of fault zone) below the normal foundation level to a depth of 18.75 m (1.5 times the width of the fault) on the upstream and 25 m (twice the width of the fault) on the downstream along the fault and backfilling with concrete. An increasing depth of plug from the upstream to the downstream has been provided in view of the increasing stresses along the dam base under the reservoir full condition.

ii. Excavation of two shafts on the upstream and the downstream ends of the trench of depth equal to 12 m and 6 m respectively below the trench bottom and backfilling with concrete to act as seepage cut-off.

iii. Contact grouting between the side faces of the plug and the excavated slopes of the trench.

iv. Provision of a gallery in the concrete plug for drainage and inspection.

d. Treatment of weak layer

Treatment for safety against sliding of the dam on argillaceous sandstone layers on the right bank and red bole layer on the left bank was started in the year 1982-1983. The treatment along the weak layers (Figure 3.2.3) comprises of:

i. A grid of longitudinal and transverse shear keys of (3 m x 2.50 m size) spaced 7.50 m and 11.50 m respectively in the two directions for the red bole layer.

ii. A grid of similar keys (3 m x 3.50 m size) spaced 7 m and 11.50 m in the longitudinal and transverse directions respectively for the argillaceous sandstone layers.

The shear keys were constructed by excavating drifts and backfilling them with concrete. The contact of the shear keys with the rock was also pressure-grouted to ensure monolithic action and thereby mobilize maximum resistance against sliding. It was also proposed to provide peripheral drainage tunnels to control uplift pressures in the treated zone. Similar type of treatment has also been provided on the Itaipu dam, Brazil.

e. Grouting and drainage

To control uplift, there are heel gallery (1.50 m x 2.30 m) and two supplementary drainage galleries (1.50 m x 2.50 m each) located downstream of the heel gallery, as shown in Figure 3.2.3. The vertical drainage holes provided downstream of the curtain grouting, are of is 75 mm diameter with 3 m spacing, and their depth 0.25 times dam height or below the bottom of argillaceous sandstone layer. Curtain grouting holes are inclined upstream with the diameter of 35 mm, 3 m spacing and the depth is 0.25 times dam height or 30 m minimum. Consolidation grouting is 3 spacing and 30 m deep (Figure 3.2.1).

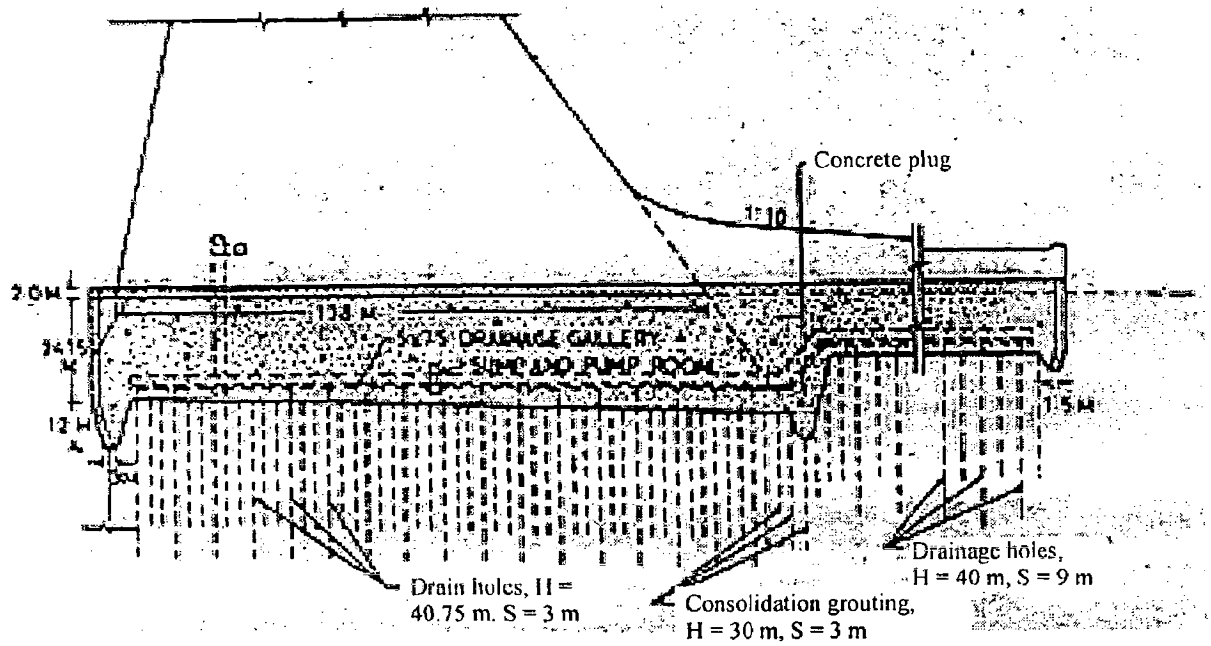


Figure 3.2.1: Sardar Sarovar Dam.
Consolidation grouting and Drainage Holes

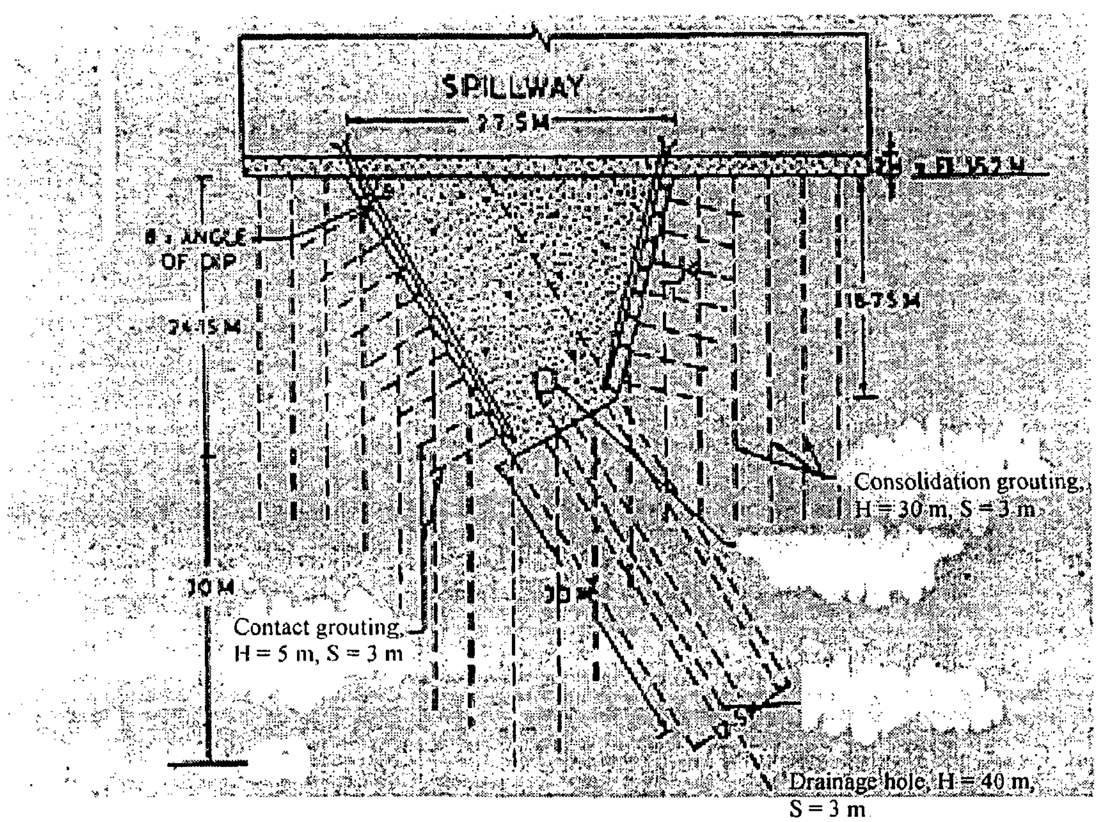


Figure 3.2.2: Sardar Sarovar Dam.
Treatment of Fault Zone

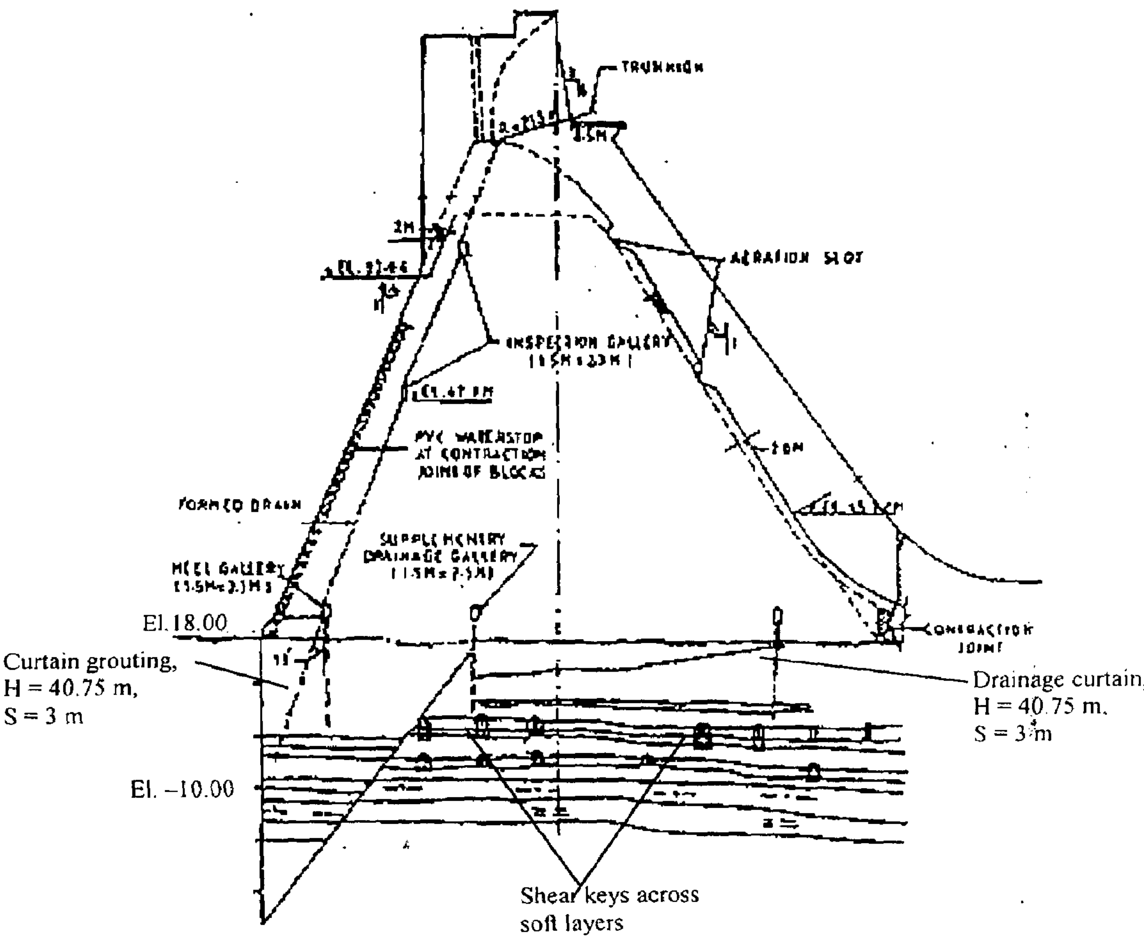


Fig. 3.2.3. Sardar Sarovar Dam.
Treatment of Soft Layer by
Shear Keys

3.2.2. Nagarjunasagar dam

a. Project feature

Nagarjunasagar dam across the river Khrisna in Andhra Pradesh state of India, with a maximum height of 124.66 m above the deepest foundation level and with the total volume of 5.61 million cu.m is the highest and largest rubble masonry dam in the world built by the largest manual labors (60,000 labors) ever engaged on the construction of any dam in the world. Total length of the dam crest is 1,450 m with the crest level of el. 184.40 m.

b. Foundation conditions

The main dam is located at latitude 16°34'23" north and longitude 79°18'47" east. The dam site lies in a hilly tract of thorn scrub jungle region through which the river Krishna has carved its course eastwards. The hills constitute the northern fringes of the Nallamalai range and are mostly flat topped.

The riverbed at the dam site is of Archaean granite-gneiss free from undesirable joints and fissures. The foliation is not distinct. The right bank is steep and of good height, while the left bank rises fairly rapidly. On both the flanks, flat bedded or gently dipping quartzites rest on the decomposed granite-gneiss.

The site selected is geologically sound for the foundation of the dam. With the types of rocks such as granites and granite-gneiss, the outcrops in the riverbed are usually found to be hard, massive and sound.

There are possibilities of weathered patches in them due to percolation along the joint planes in the rocks. Such weathered patches are eroded away and are usually concealed under water or water-borne debris. Similar situation is met with in the brecciated or sheared zones as well.

At the left flank, the depths of weathering in the granites underlying the sediments are very variable. There is a sudden drop in level of the granite-sedimentary rocks contact just upstream of the dam axis, and a band of breccia and an adjacent dyke enter into the left abutment very near to the dam axis.

In the bed of the river, hard, massive and fresh granite rock is met with at shallow depth. Except for a few pockets of weakened rock and seams of fractured rock, sound rock was met with throughout the riverbed portion, within an average depth of 3.05 m.

On the flanks, which 30.48 m to 36.58 m above the riverbed level, the granite strata are overlined with flat-bedded sedimentary rocks of quartzites and shales. In the first 9.14 m to 12.19 m depth at top, flat-bedded massive quartzites are met with. In the next 9.14 m to 10.67 m depth, thin-bedded quartzites are found.

Below those quartzites, shales are met with for 6.10 m to 7.62 m depth before the granite below could be reached. The depth of quartzites and shales overlying the granites ranges from 27.43 m on the left flank to 44.20 m on the right flank.

The quartzites are hard, compact and fine grained. They posses very good bearing strength but they are flat bedded with prominent bed joints. The shales occur as band and intercalations in-between the quartzites.

The foundations of the masonry dam in blocks no.6 to no.69 rest on granites, while those in blocks no.1 to no.5 on the left flank and no.70 to no.79 on the right flank rest on fresh quartzites.

c. *Treatment of granite rocks foundation*

In the riverbed portion, where granite rock is with in the foundation, the following treatment was imparted (Figure 3.2.4):

i. Consolidation grouting of 4 to 5 rows parallel to the dam axis at 6.10 m intervals in each row, at shallow depth and normal to the surface, except where it was required to intercept joints in the rock. The pressure was low, ranging from 5.27 kg/cm^2 to 7.03 kg/cm^2. The diameter of Wagon drill bits varies from 5.08 cm to 6.35 cm. Cement-water grout proportions vary from 1 : 3 to 1 : 10. Test holes were drilled and grouted later on to see if the grouting done earlier had filled up all the seams and joints fully. The purpose of this grouting was to improve the bearing surface of the dam foundation.

ii. One row of curtain grouting at interval of 3.05 m along a line parallel to dam axis and close to the heel, at a depth varying from 30.48 m to 45.72 m was carried out through the foundation gallery provided in the dam just at a little height above the foundation rock after the dam masonry was raised to sufficient heights of 30.48 m to 45.72 m above foundation level, to see that the top layer of foundation rock did not get disturbed due to the high grout pressures. The grout pressure varied from 17.58 kg/cm² to 21.09 kg/cm². The grout holes were drilled through 7.62 cm diameter pipes embedded in the dam masonry below the foundation gallery with a slight inclination towards the heel of the dam. The purpose of this grouting was to form a watertight curtain and cut-off percolation in the foundation rock.

iii. Provision of drainage holes in rear of the grout curtain was made to take out any water that might find its way beyond the grout curtain, and thus reduce the uplift on the base of the dam. These holes were drilled through the same foundation gallery in a line parallel to the grout curtain and at about 1.22 m in rear. In order to ensure that they remained open and did not get filled up when the surrounding area was grouted, these holes were drilled in the end after the grouting of the area around was completed. As these holes were to be of maximum possible size to be more effective, 7.62 cm (3 in) diameter holes were drilled with NX diamond bits through 10.16 cm (4 in) diameter pipes embedded in the masonry. They were drilled to a depth of about 24.38 m and placed at 6.10 m intervals.

iv. In places where concrete or masonry was laid over rock or old concrete, contact grouting was done to fill up the gap effectively and prevent separation that may take place after setting due to shrinkage.

e. *Treatment of quartzite rocks foundation*

In flanks where the dam is founded on quartzite rocks, more extensive drilling and grouting (consolidation grouting) was carried out for the foundations as indicated below (Figure 3.2.5):

i. Consolidation grouting with holes put for the whole width and length of the foundation area at interval of 6.10 m to 7.62 m, at shallow depth and normal to the surface, except where it was required to intercept joints in the rock. The pressure was low, ranging from 2.81 kg/cm² to 3.52 kg/cm². The diameter of Wagon drill bits varies from 5.08 cm to 6.35 cm. Cement-water grout proportions varied from 1 : 3 to 1 : 10.

ii. Blanket grouting of the rock upstream of the dam for a width of 15.24 m to 30.48 m in the same manner as for consolidation grouting.

iii. One row of curtain grouting similar to the one mentioned in paragraph *c* above has been provided.

iv. Drainage holes in rear of the grout curtain in the same manner as for granite rocks in riverbed portion.

v. Special treatment to the abutting rocks on either flank, where the foundation grade changes abruptly, was provided, by drilling and grouting at different levels with radial pattern holes before the masonry dam was built, and through shafts and adits left in the body of the dam later on, after the masonry was built.

e. *Treatment of weak zones*

Detailed exploration carried out during construction by intensive drilling and washing revealed the existence of fault and dyke zones below some blocks.

The followings were the treatments carried out to overcome such weak zones:

i. A wide soft area (fault) consisting of highly weathered and crushed granite varying from 1.22 m to 4.88 m in thickness and dipping towards upstream side was located in blocks 7 to 9 at a depth of 6.10 m to 30.48 m below a large mound of fresh granite rock. This fault zone was excavated inside by driving an adit of 1.52 m to 3.05 m length, and removing the weathered and soft material, and backfilled with concrete. In addition, vertical holes were drilled from the top of the fresh granite mound, and inclined holes from the sloping face extending below the fault zone and grouted after washing through away of the soft material.

ii. A dolerite dyke intercepted the foundations extending from the upstream end of block 18 to downstream end of block 23. Width of the dyke increased from 0.91 m at the upstream end to 2.44 m at the downstream end. Exploratory holes drilled into this dyke revealed that this dyke extended to considerable depths. Excavation was carried out deeper along the fault zone and a vertical shaft of about 9.14 m x 6.10 m was excavated to a depth of 9.1 m at the upstream end of the dyke, and radial grouting was done from the bottom and sides of the shaft with 9.14 deep holes. The full length of the dyke traversing the foundation area was excavated to a depth of not less than twice the width of the dyke zone. All the soft material including fractured rocks were removed until the hard granite surface exposed. The trench and shaft were then backfilled with concrete up to the general foundation level. Contact grouting was done later on after shrinkage and settlement take place.

iii. A 0.61 m to 1.22 m wide band of crushed zone was met with in block 55 traversing the full width of excavation at right angles to the dam axis with a dip of 45° towards the right abutment. On drilling exploratory holes, it was revealed that the crushed zone extended to a considerable depth at the same dip angle and in the same direction. This crushed zone was treated in same manner as the dolerite dyke in block 18 to 23, by excavating a shaft of 11.28 m x 5.18 m size to a depth of 9.14 m at the upstream end of this fault to serve as a cut-off. The shaft was drilled and grouted from the bottom and sides at different level with radial holes of 9.14 m depth. Along the length of the crushed zone, a trench was excavated to sufficient width and depth of not less than twice the width of the crushed zone. Uplift pipes were embedded along this trench before the trench and the shaft were backfilled with concrete and taken out into the foundation gallery for observation purposes at a later date.

iv. The dam is founded on quartzite rock in blocks 70A and 71 where open joints were found with the width ranging from 5.08 cm to 7.62 cm at the surface narrowing down to 2.54 cm at lower levels. This joint traversed the full width of the dam in block 71 and extended to considerable depth up to the shale strata below. A 3.05 m to 4.57 m wide open trench was excavated along this joint and a number of drill holes were put on either side of the joint, spacing as well as inclination of the holes being so fixed as to intercept the joint at different depths. The joint was then thoroughly cleaned through all these holes, grouting was done to refusal, and the trench was backfilled with concrete.

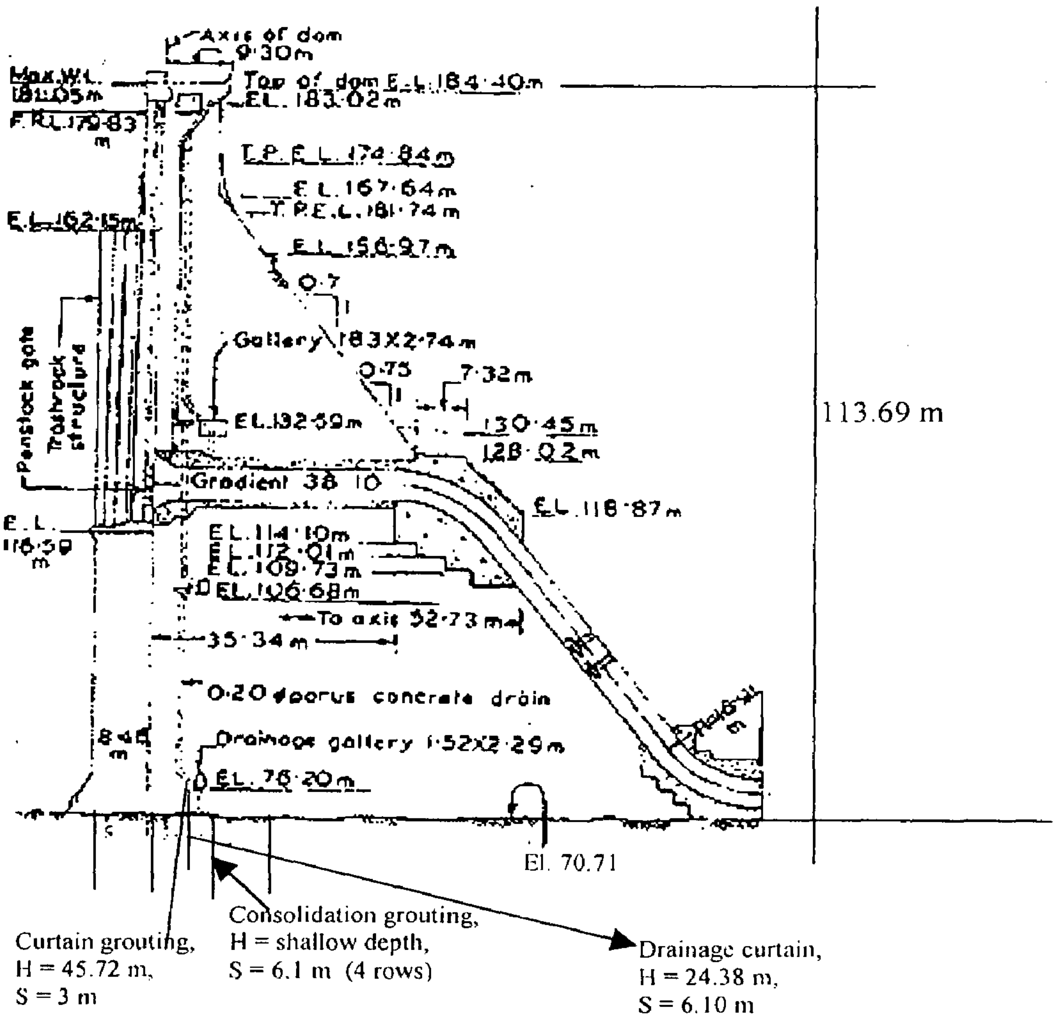


Fig.3.2.4: Nagarjunasagar Dam. Treatment of Foundation Resting on Granite Rock

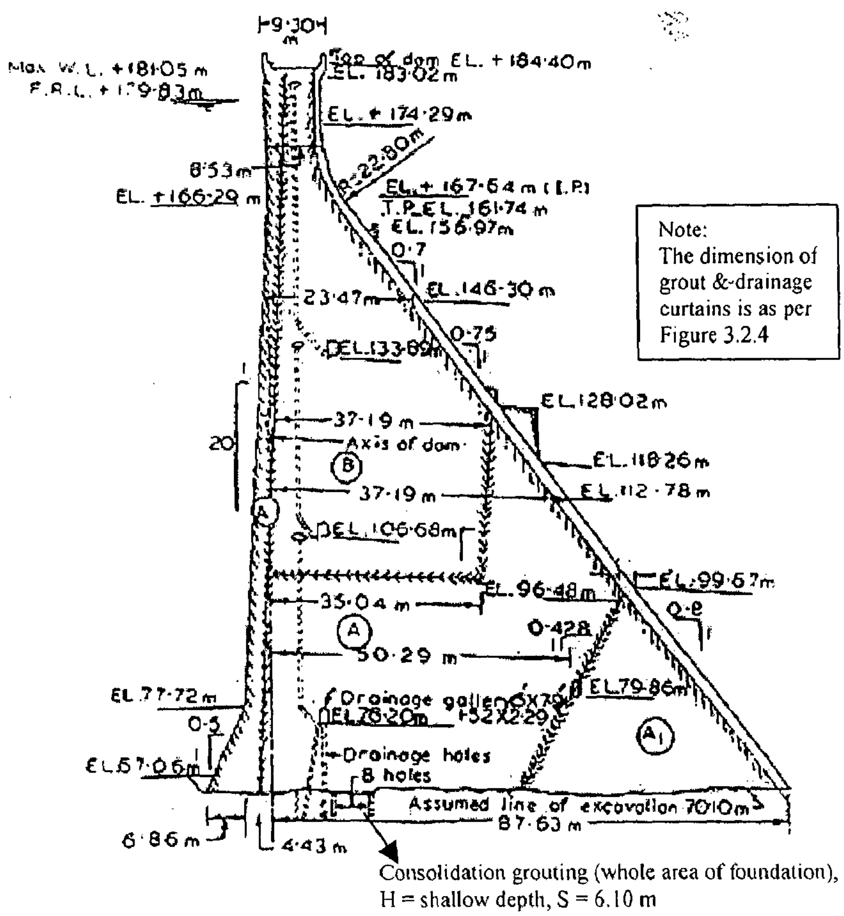


Fig. 3.2.5: Nagarjunasagar Dam. Treatment of Foundation Resting on Quartzite rock

3.2.3. Keban dam

a. Project feature

Keban dam was constructed on the river Euphrates which collects a large proportion of the Eastern Anatolian water, passes through the plains of Mesopotamia and after joining Tigress, the river discharges into the Persian gulf. The dam is a combination of a clay cored rock fill embankment and a concrete gravity dam, located on the river Firat (Euphrates), approximately 45 km northwest of Elazig, Turkey.

The rock fill dam section extends from the rock hillside on the right abutment a distance of 608 m, where it joints the concrete gravity dam which is 547 m long at crest. The maximum height of dam above the deepest excavation level is 211 m with the total crest length of 601 m and crest width of 11 m. Concrete gravity dam section consists of north and south gravity dams, intake and spillway structures.

b. Foundation conditions

Keban dam and the environs lies in between the two important tectonic lines, namely the North Anatolian fault line and the east Anatolian fault line. Hence Keban area has been subjected to large and small faulting, folding and displacements and large destructive earthquakes have been recorded the past 175 years.

Keban dam foundation consists of deeply kartstified metamorphic limestone and underlying schists both belonging to the Paleazoic age. The valley at the dam site is rather steep and contains an alluvium of 45 m thick at the bottom. Calcschist and black dolomitic limestone layers underlie the white and pink colored, karstified, cavernous limestone and marble, which outcrops at the top. All these formations are broken by numerous joint and fault systems of varying size and texture several orogenic movements. The original bedding planes generally are not visible although they are still visible in some locations. Karstic cavities developed mostly along the main faults and intersecting joints became waterways to hydrothermal ground water circulations. The karst structure is widely observed in the easily soluble limestone-marble mass.

One main fault, which deeply affects the far end of the right abutment and the access of the valve chamber tunnel, has N 5° - 10° W strike and 80° - 90° NE dip. Large clay filled cavities have developed on this fault. In most cases, these cavities are connected to each other more or less parallel with this fault.

Another fault zone cuts the block 5 and 6 in the north gravity dam and goes down till the schists, which are located between the two diversion tunnels. It has N 50° E strike and 70° - 80° NW dip. The large clay filled cavities, which have developed along this fault affect a large are under the north gravity dam and the plug area of the diversion tunnels.

Under the south gravity dam, the limestone-marble mass is heavily broken through joints and fractures of random direction, forming solution voids, chimneys, and cavities developed along the weak zone. The remaining rock is heavily weathered and brecciated in several locations. At the left end of the dam above elevation 835 there is a large cavity zone which is filled with plastic clay and conglomerates containing rounded gravels of limestone which are similar to those that can be found in actual river deposits. This fact proves that the river plays a big role in filling karstic cavities.

Both abutments were very steep till el. 685.00 m and were cut by vertical, oval shaped pinnacles. The rock at the left abutment consisted of very endurated schist within the 60 m wide core trench area. On the right abutment there was schist and limestone due to a big fault, which ran immediately upstream of the dam axis with 30 m of displacement in vertical direction. In spite of its big thrust, this fault was quite tight till el. 685.00 m.

c. *Foundation treatment (surface treatment)*

Due to the much-karstified nature of the dam foundation, a large-scale underground work and foundation treatment has become inevitable for the prevention of probable leakage and for the improvement of foundation strength. The surface zone under concrete gravity dams (north gravity, intake, spillway and south gravity) was treated in various ways in order to eliminate the possibility of uneven settlement as well as percolations through faults, fissures and cavities.

All the heavily weathered, broken and brecciated materials were excavated especially in core trench area of left abutment and under south gravity dam. All clay cavities were cleaned and backfilled with class D concrete. The weak zones in the grouting adits (fault, shear and brecciated zones) were excavated either by blasting or by hand working up to 7 m upstream and 2 m downstream from the dam axis. Steeply sloping or overhanging parts of the abutments within the core trench area were leveled of either by blasting or concreting to give a finished slope of 1:1 at maximum.

For consolidation grouting, 3 m by 3 m square pattern was normally used, but on deeply karstified and faulted swiss-cheese like foundation, the holes were spaced 1.5 m to 2 m as required. In order to avoid unremedial surface leaks, the first lift of concrete has been poured before starting to grout. The same concrete slab for the entire surface of core trench area above the el. 775.00 m was proposed.

The entire surface under each block of the concrete gravity dam and the impervious core and filter area of the rock fill dam was grouted through 10 m and 15 m deep holes using 3 m square pattern for the purpose of consolidating the foundation. The holes were drilled to full depth, washed, water tested and grouted under 2.11 kg/cm² (30 psi) using a surface packer. If the take in cement grout exceeded 1 sack (50 kg) per linear meter of one hole, check holes were drilled and grouted on the centers of the 3 m square pattern.

On account of heavily faulting and karstification of the foundation, it often happened that several holes were interconnected. Those holes were carefully washed and grouted simultaneously. Grout take in the holes which encountered a karstic cavity underneath was very high till 107 kg cement and 17 kg sand per linear meter of hole.

On all slopes steeper than 40° from the horizontal under concrete blocks, contact grouting system will be installed and grouted later with a pressure of 3.60 kg/cm² (50 psi).

d. Curtain grouting

The nature of the rock at Keban dam makes necessary to use multiple rows of grout curtain, which allows it to be relatively wide. A grout curtain system for prevention of excessive leakage has been constructed. The geometry of the curtains has been monitored with the freshly available information obtained during the construction period. The curtain system consists of three units, which are interconnected to each other i.e. left reservoir rim curtain, main dam foundation curtain and the right reservoir rim curtain. The typical cross section of grout curtain is shown in Figure 3.2.6.

The left reservoir rim curtain extends from the hanging type main curtain in limestone to the upper schists in the reservoir area. The right reservoir rim curtain again extends from the main curtain hanging in the upper limestone and dolomites of the Keban formation to the calcschists on the right bank and is intended to intercept any lateral leakage only in the upper limestone.

It was proposed to grout first downstream row with primary and secondary 3 m spaced holes to avoid any further travel of grout toward downstream, while the upstream main curtain row is grouted. The middle row will contain only check holes which are to be located where necessary after having completed the downstream and upstream rows. In order to avoid heavy leakage in lower adit, two 5 m upward holes and one 7 m upstream side horizontal hole for connection of two curtains were to be grouted first with a pressure of 3.60 kg/cm², before doing any grouting operation in the above next adit.

e. *Treatment by concrete cut-off wall*

The brecciated rock in the left abutment required the construction of a concrete cut-off wall under the rockfill dam, extending from el. 730.00 m to a vertical line at sta. 0+610, and in the south gravity dam from under block 18 to a shaft in left bank at el. 703.00 m.

In this area, the limestone is heavily broken and appears in blocks surrounded with red clay. In such an area performance of the impervious curtain by means of grouting was impossible.

The concrete cut-off wall was constructed using the existing grouting galleries and shafts, with a minimum width of 1.50 m upward from the gallery inverts. Excavation for the cut-off wall was performed with sloping method in stages having slope of 45°, and the concrete was placed in lifts of the same slope (Figure 3.2.7).

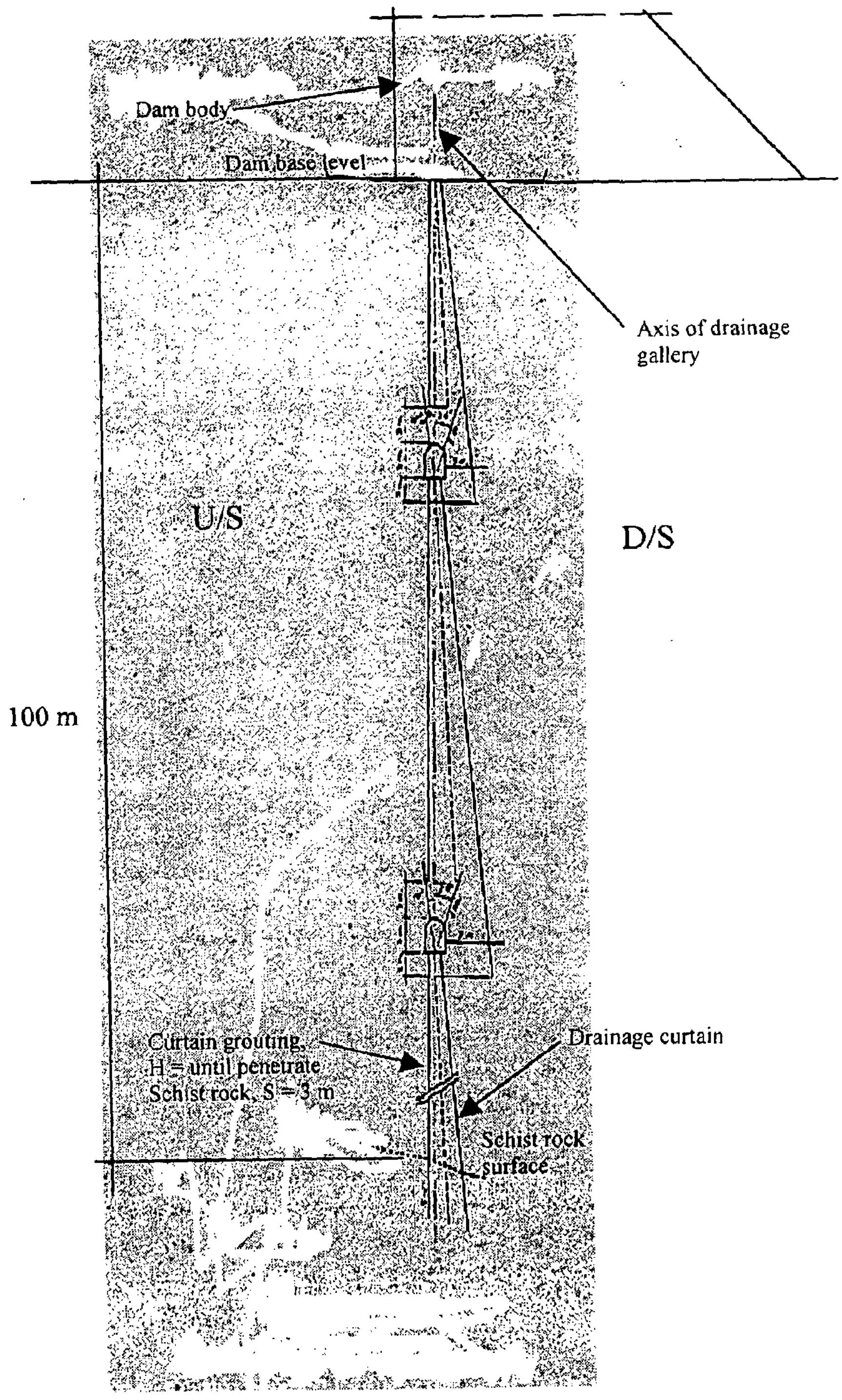


Fig. 3.2.6: Keban Dam.
Curtain grouting

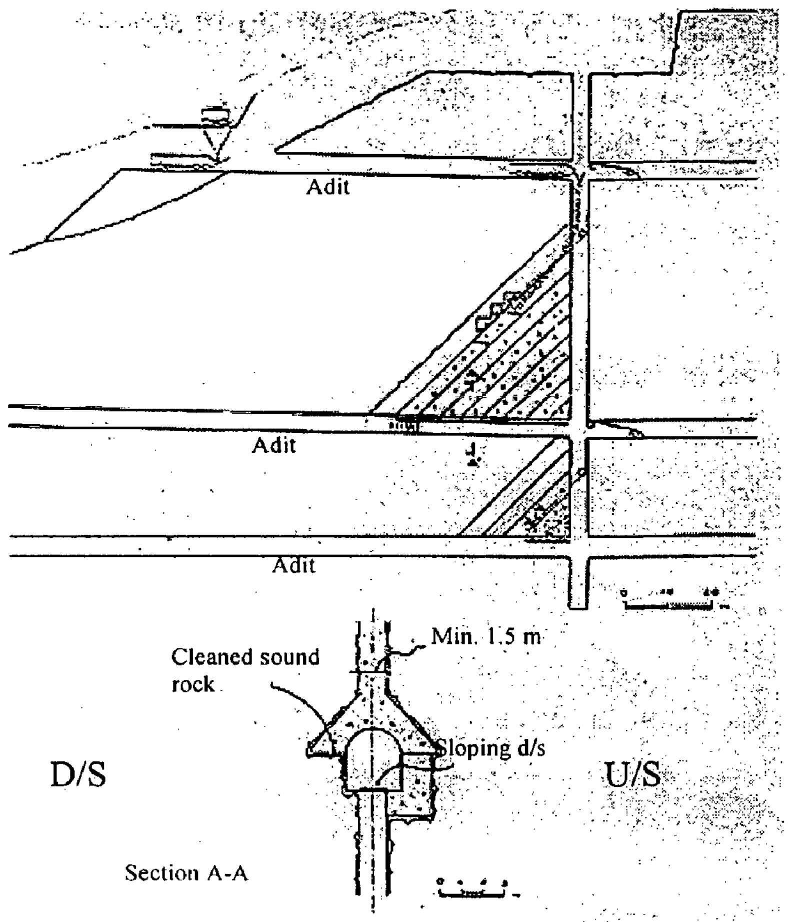


Fig. 3.2.7: Keban Dam.
Concrete Cut-off Wall

3.2.4. El Cajon dam

a. Project feature

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 main structures are a 238 m high arch dam, an underground power station with 4 units of 75 MW each and a grout curtain.

The dam site is located in an 800 m long, narrow canyon. The distance from San Pedro Sula, the economic center of Honduras, is 80 km. It was cut by the river Humuya through the northern limb of an anticline in the Atima limestone about 2 km downstream of the mouth of river Sulaco.

The curtain's crucial test was started on June 15, 1984, when first reservoir impounding raised the water level by 70 m in only two weeks without causing any major problems as to the curtain's behavior.

b. Foundation conditions

The geologic situation of the dam site is quite favorable. The surrounding landscape is entirely covered with volcanic rocks. The exception is the canyon of the river Humuya, which is approximately 800 m long and in which a window has been carved into this volcanic cover exposing an underlying limestone formation. This limestone is more than 700 m thick and subdivided into 4 different lithological units. Their base was not reached by any drilling. Neither older nor younger formations are exposed at the dam site.

The four lithological units of Atima limestone are as the follows:

i. A sublithographic limestone, which is massive and thickbedded. In the dam axis, it reaches up to 50 m below the crest level, and its base was not reached by subsurface investigation.

ii. A limestone, medium bedded with chart nodules, which is 50 m thick. It forms the upper part of both abutments.

iii. A thinbedded limestone, with a thickness of about 100 m to 120 m and layers generally ranging from 0.20 m to 2 m. It forms the canyon walls above crest level of the dam.

iv. A dolomitic limestone, about 140 m thick, which is only present at the downstream end of the canyon.

The direction of the dip is approximately 15° to 20° downstream in the axis of the dam and increases up to 35° near the end of the canyon. The volcanic rocks, which cover the limestone above el. 350 m at the site, are effusive rocks and very heterogeneous in the horizontal direction. The alluvium thickness in the riverbed is approximately 20 m.

The entire limestone formation is cut by 3 almost vertical fault zones, which run more or less parallel to the river. These predominant zones vary in thickness from a few centimeters to a maximum of 15 m and had a decisive influence on the layout of the dam and appurtenant works. As compared with the large scale faulting system of the Honduras depression, they are insignificant in size. These faults are inactive.

Karstic phenomena in form of tubes are frequent, even in the limestone unit above the ground water level. The dimensions of these tubes are small. Occasional channels of 1 m in diameter represent the rather extreme near-surface cases. Generally the tubes follow the discontinuities and are particularly pronounced at their intersections. As compared with the total excavation volume of the exploratory galleries, the degree of karstification (ratio of karstic to excavated volume) has been estimated at only about 1 percent.

Owing to karstic phenomena, the limestone constitutes a true aquifer. The ground water level perpendicular to the canyon has been determined by drilling at different distance of 300 m from the canyon wall. The ground water level fluctuates with only a small delay as compared with the river level.

The geomechanical properties of bedrock as the main result of the foundation investigation are that the static modulus of elasticity is 700 t/cm^2, unit weight is 2.70 t/m^3 and the compressive strength of limestone and volcanic rocks is 934 kp/cm^2 and 120 kp/cm^2 to 1,350 kp/cm^2 respectively.

c. *Curtain grouting*

Owing the karstic phenomena of the surrounding limestone, the treatment will be quite extensive. Moreover, extensive additional geological information gained during early construction lead to believe that the probability of a deep karst system was high. The curtain's geometry was therefore changed.

Three rows curtain grouting were applied, parallel to each other with additional intermediate holes at locations where the cement take in any 5 m stage of the obligatory ones exceeded 200 kg/m. The three rows are primary, secondary and tertiary holes, and the additional rows are quaternary and quandary holes. The spaces between the rows are 10 m, 10 m 5 m, 2.50 m and 1.25 m respectively, and the ones of holes are 10 m, 5 m, 2.50 m, 1.25 m and 0.625 m respectively.

All drilling was with AX-diameter and both rotary as well as percussion drillings were used. However, due to the importance of geometrical precision for the intersection curtain planes, rotary drilling was preferred occasionally.

Grouting was done upstage except in very weathered or karstified rock where downstage grouting had to be used. The grouting pressure varied between 10 bar to 50 bar, and the grout mix was expressed in W/C ration of 0.70 by weight.

A section of the curtain was considered completed if both of the following conditions were met in the 5 m-stages of check hole intersecting obliquely the curtain drillings:

i. Absorption in water pressure tests below 2 Lu (l/m.min at 10 bar)

ii. Cement absorption below 50 kg/m.

Finally, the total length of drilling was about 485,000 m with the vertical area of about 530,000 sq.m and the deepest curtain hole extended to a depth of 180 m below the dam base. The average cement take was 210 kg/m, and 100,000 tone of cement were injected.

d. Treatment of karst and gouge material of fault

Whenever karst phenomena were encountered during excavation, it was treated by removing the weak material then backfilled with concrete as well as grouting the contact zones subsequently (Figure 3.2.8). The largest karst hole was encountered in the contact zone between limestone and volcanites. It was filled with concrete. In addition, the gouge material of faults no I, II and III were replaced by concrete down to a depth where the forces from the dam are small.

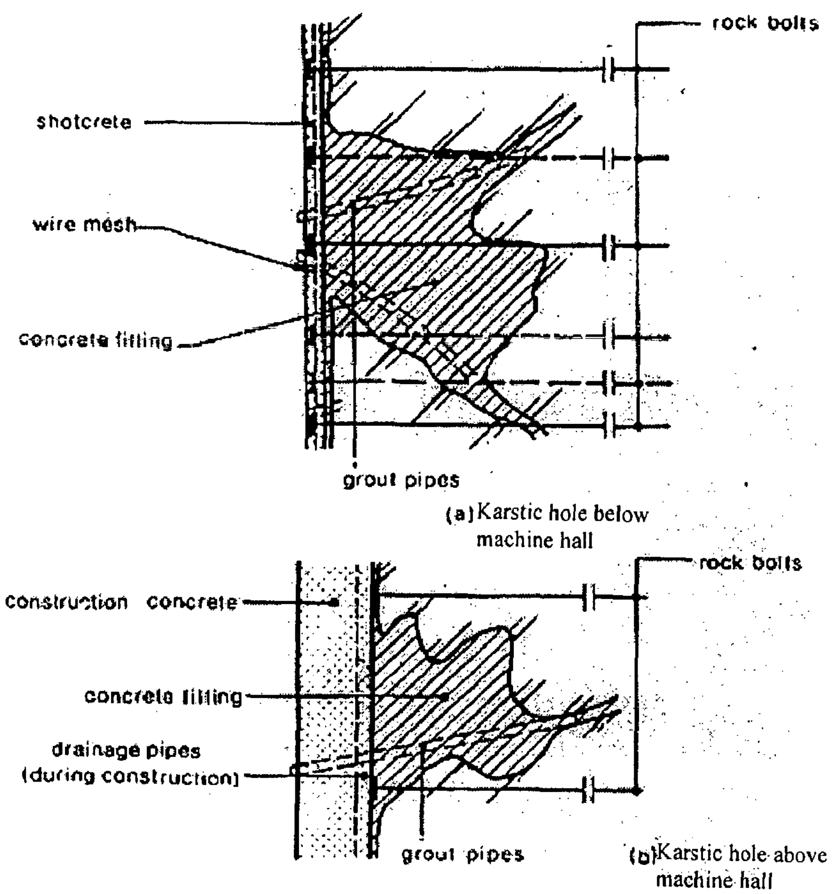


Fig. 3.2.8: El Cajon Dam.
Treatment of Karstic Holes

3.2.5. Liujiaxia dam

a. Project feature

The Liujiaxia hydropower project is located on the river Huang He (Yellow) in Yongjing County, 100 km upstream of Lanzhou, Gansu province. This project was completed at the end of 1974. The project includes a dam, a powerhouse, several outlet structures and other auxiliary structures.

The Liujiaxia dam is a concrete gravity dam composed of five parts namely a high gravity section, two low gravity wings, a spillway section and an embankment of loess. The high concrete gravity section has a maximum height above the deepest foundation level of 147 m, a crest length of 204 m, 16 m of crest width and 760,000 cu.m volume of concrete. The loess embankment has a maximum height of 45 m and a crest length of 216 m, with its right wing inserting into the plateau of gravel and loess.

b. Foundation conditions

The main dam is situated in a narrow gorge with steep bank slopes. The bedrock at the dam site consists of primarily of micaceous quartz schist of presinian system with some layers of hornblende schist. Part of the rock mass is interpenetrated with a small amount granite vein and lamprophyre dyke. In general, the rock is hard and of high compressive strength, providing a good site for high concrete dam.

As the rock formation intersects obliquely with the river, and the distribution of fissures and fractures varies from place to place, seepage through various parts of the foundation is quite different. In what follows (layers) the foundation of the main dam will be divided arbitrarily into three parts, namely the riverbed, the left and the right abutments with the seepage characteristics for each part described correspondently.

i. The permeability of the riverbed is rather small. The coefficient of permeability varies from 6 Lu to 0.30 Lu. On the left side of the riverbed, a group of tension faults along the river was found. The fault not only dips steeply, but also extends quite a distance and covers a wide area. The major fault, F_{69}, running across the dam axis, contains a crushed zone, usually more than 2 m in width, consisting primarily of tecnoclastic fragment, breccia and a small fraction of clay. The fault zone, composed of tectonite, is about 3 m to 4 m in width. When

F_{69} and its neighboring tension fault such as F_{154}, F_{155}, F_{175} and F_{176} merge together, however, the zone affected by the faults becomes as much as 10 m wide. Data from chemical analysis revealed that the ground water does not have any leaching effect on either the foundation rock, or the cement used in grouting, and the rock itself contains very little soluble constituents as well.

ii. The rock mass at the left abutment was moderately or highly weathered. It contains a comparatively well-developed tectonic crushed zone, in which 18 major strips of crushed material may be marked between el. 1,610.00 m and 1,720.00 m. The width of these strips varied from 2 cm to 15 cm in general, the widest being 40 cm. The pressure tests indicated that the permeability of the tectonic crushed zone is rather small, and not a single case of mechanical piping had occurred in 13 tests under low pressure (2 kg/cm^2) and 7 tests under high pressure (23 kg/cm^2). The occurrence of the crushed zone, however, conforms to that of the rock bedding, both dipping 20° to 40° downstream toward the riverbed. Further the downstream it is, the greater the dip will be. The joints and the fissures in the neighboring areas were well developed, the fraction of fissures being 0.50% to 1% and reaching 4% near the bank slope surface. A number of major fissures intersected with the rock mass of the abutment into trigonal pyramids of different sizes.

iii. The rock mass in the right abutment was only mildly weathered, the distribution of fissures and fractures was far from being well developed, and the occurrence of rock bedding dipped towards the bank. Hence the stability of the abutment will not be endangered.

c. *Foundation excavation*

In order to improve the condition of stability and the regime of seepage flow, the foundation of all dam blocks was excavated to fresh rock bedding. Endeavoring to provide an appropriate space for flow in front of the tunnel intakes to turbines no. 1 and no. 2 as well as to facilitate the treatment of fault F_{92}, the bedrock under the dam blocks I, II, III at the right abutment was further excavated a considerable depth to el. 1,673.00 m. Accordingly, with an aim to cut off some major fissures, and to remove the tectonic crushed zone near the ground surface, bedrock at the left abutment under dam blocks IX and X was cut back 20 m to 25 m.

d. Treatment of fault zones

As regards the treatment of tension fault zones along the river, such as F_{69}, a vertical shaft 3 m x 4 m in cross section and 15 m deep was excavated on the line of the deep grout curtain and backfilled with concrete, to serve as a cut off wall, under which a deep grout curtain 36 m in deep was set up. A trench was excavated in the fault zone, downstream of the cut off wall, also backfilled with concrete to form a plug. The depth of excavation is equal to the width of the fault. Cement grouting for consolidation to a depth of 15 m was also carried out on both sides of the fault including the fault affected zone. Filters were provided in the drainage wells to prevent mechanical piping.

e. Curtain grouting

One row of deep grout curtain of cement line (line B), shown in Figure 3.2.20, was provided for dam blocks II to X, parallel to the dam axis. In the riverbed, grout holes 50 m deep are spaced 2 m center to center. At the left bank, another row of grout curtain with depth varying from 80 m to 120 m was placed to minimize the influence of seepage flow around the dam. The grout curtain started from block IX, passed through the foundation of the concrete gravity wing and crossed fault F_{80} and F_{152}.

f. Consolidation grouting

In any parts of the foundation where fissures and fractures extensively distributed and the rocks moderately weathered, consolidation grouting was performed. The grout holes, 7 m to 10 m in depth, and 3 m to 4 m in spacing, were staggerly drilled with a row spacing of 1.50 m to 2.40 m on centers.

g. Drainage system

In the riverbed, downstream of the grout curtain, 3 rows of longitudinal drainage wells (line C, D and E) and 2 rows of transverse drainage wells (line G and H), all in 13 cm in diameter and spaced at 2 m, were provided. Line C, the primary drainage curtain, had a depth of 25 m, whereas line D had a depth of 15 m, and others 10 m. Figure 3.2.9 shows the general layout of the drainage system and grout curtain. It shows that the first drainage gallery links with both the drainage galleries in the left abutment at el. 1,631.00 m. The seepage discharge through the foundation was collected into water sumps, installed in dam blocks V and VI and was to be pumped into the river downstream.

The drainage system in the left abutment (left of dam block VIII) consisted of a curtain of drainage wells, a drainage gallery and a diversion tunnel. Two rows of drainage wells, 14 m in depth and 2.50 m in spacing, were set up in the horizontal gallery at el. 1,631.00 m. The exploration adits formerly driven for geological investigations at el. 1,631.00 m, el. 1,660.00 m and el. 1,690.00 m, were reconstructed also as horizontal drainage galleries. They extended from the riverbed right into the left bank until reaching the top of the diversion tunnel, its elevation being 1,625.00 m. Drains were drilled, fanning out from the top of the tunnel, and connected to drainage pipes to collect any seepage flow through the left bank, and diverted downstream through the diversion tunnel.

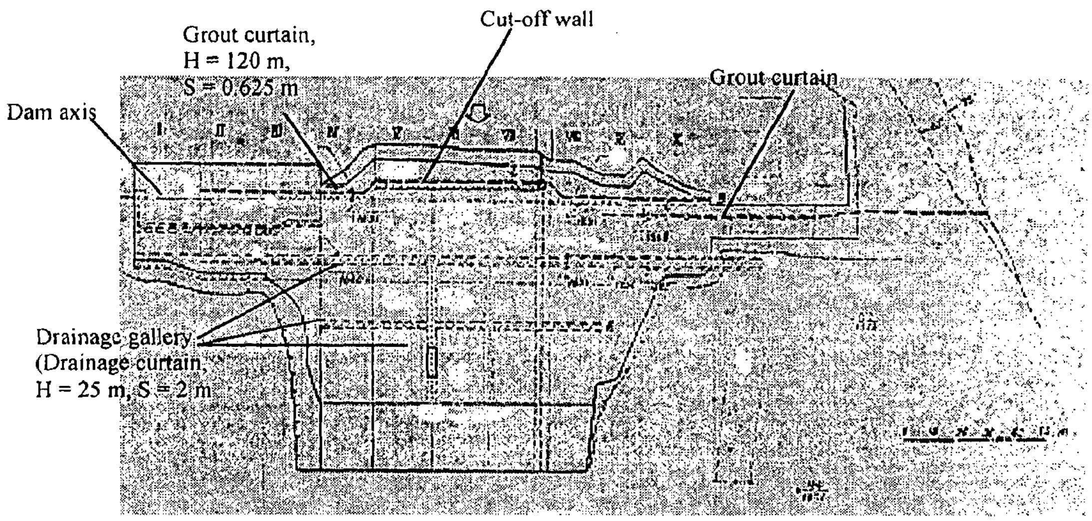


Fig. 3.2.9: Liujiaxia Dam.
Grout and Drainage Curtains

3.2.6. El Atazar dam

a. Project feature

The El Atazar project is located in the province of Madrid, 3 km downstream from the mouth of the river Riato and within the boundaries of the municipalities of Patones and Atazar, Spain.

The project includes a thick arch gravity dam completed with spillway and dewatering outlets. The dam has a maximum height above the deepest foundation level of 133.40 m, a crest length of 204 m, 6 m of crest thickness of the upper arch (el.867.00 m), 190 m of the radius of the reference surface (central zone), 52° angle in the center and 900,000 cu.m volume of concrete.

The main centerline of the dam is notably in a W-E direction. The sides of the valley are practically symmetrical, with slopes of approximately 40°. The bottom of the bed is at el. 747.00 m, the excavation of the central blocks at el. 740.00 m, and the crest of the dam is at el. 873.40 m.

b. Foundation conditions

The topography of the gorge is favorable for receiving the work envisaged as the contour lines of the ground have, in the main, a notable trend normal to the arches with the exception of the of the lower part of the left bank where that trend departs slightly from the normal due to the river in that zone, imposing a greater volume of excavation in the lower part of this bank.

The strata comprising the zone of the location of the El Atazar dam are Silurian, Pliocene and Holocene formations. Granite and crystal aureoles, extending to the west, have to be added to the former when referring to the surroundings of the dam.

The Silurian strata is represented by the two Ordovician and Gothlandian layers with respective predominance of quartzites and slates, traces having been found in the former of "crucianas" and other uncertain filiations. The lower quartzites only appear along the borders of the stain in contact with mica schist and gneiss of the substrata. The slate, blackish gray when cut and darkish on the outside, only seldom shows some amphelithic variations in contrast with zones of certain resistance.

The rock is fairly folded and broken, and small white veins of quartzite have infiltrated in localized zones, which restore the mechanical deficiencies of the rock in certain parts.

The local Cretaceous has the classical faces of the center of Spain, now very reduced in thickness, and it is composed in an upward direction of Albian sandstone, Cenomaniac-Turonian marls with the Suronian limestones above which here serve as a protective flank to the whole base of the Sierra. These limestones are used for the concrete and are located some 4 km to 25 km to the south of the work site. They enclose the marvels and curiosities of the famous Reguerillo Cave.

The folding and refolding of Silurian slates are many and often, but their general direction in the zone of the work remains at SW-NE with a dip to the S of the order of some 60°. This dip changes downstream, under the form of syncline. The direction of the layers is oblique to the river. Some two hundred meters downstream, they run side to side due to the winding of the Lozoya.

As important as the stratification joints are the diaclases, which, due to old orogenic pressures, divided the rock into parallel bands, difficult to distinguish from the sedimentary layers in many cases. There are several important directions of fractures, which subdivide the "packet" of strata into parallelepiped and wedges of different sizes. The orientation of greatest interest is parallel to the right bank although there also exists another in a direction parallel to the left bank. A third is practically vertical and perpendicular to the former two. Lastly, there exists a fourth with a practically horizontal orientation. These fractures must have occurred, at the latest, in Oligocene times during the Alpine folding, although they may have undergone some local depressive reactivation with the Quaternary deepening of the valley.

Of the families mentioned, the two which establish the most important breaks have a direction parallel to the river, one dipping to the North and the other to the South, and they are considered as being the most dangerous both because of their orientation, and because of the fill of a clayey nature or highly disturbed slate, with greatly precarious mechanical characteristics.

c. *Left bank foundation treatment*

To avoid sliding during the excavation of left bank foundation, the sustaining of the slope was proceeded by constructing a grid of reinforced concrete beams on the slope, deeply anchored into the ground. The ground water was evacuated by constructing a drainage system formed by holes and galleries at different levels.

The taluses were also protected in the upper zones of the excavation with simple rock bolts and guniting the surfaces to prevent the superficial deterioration of the rock.

d. Right bank foundation treatment

Work on this bank was commenced with the reinforcement of an entire zone of flat faces surrounding the void of the excavation upstream and below all the concreting installations of the dam, and which represented a danger to their support. This reinforcement consisted of the construction of a system of concrete buttress strongly anchored to the ground with 140-ton cables and intermediate rock bolting on the flat slabs themselves with 20-ton rock bolts.

e. Treatment of diaclases (highly fractured zone)

Those of particular interest are the diaclases of more than 0.50 m thickness, which are under the foundation of the dam. The treatment was carried out by means of a double emptying, with concreting afterward. The first was carried out from the surface and in alternate trenches of a meter width. The second emptying was carried out underground, in an upward direction, also in alternate trenches and at depths of 20 m to 30 m. In this manner, the rock was decompressed as little as possible, as the first series of trenches was concreted before the next voids were excavated. This treatment of replacement was employed also in five diaclases of the left and right banks.

f. Consolidation grouting

The consolidation grouting was performed with 30 m in depth, and 2.50 m center-to-center spacing. It was done by circulating grout throughout the length of the hole in 5 m stages and a downstream direction. The pressures employed were 4 kg/cm^2, 6 kg/cm^2, 7 kg/cm^2, 9 kg/cm^{2}and 12 kg/cm^2 at depths of 0-5 m, 5 m-10 m, 15 m-20 m, and 20 m-30 m respectively.

g. Curtain grouting

The curtain grouting was performed with 45 m in depth, and 4 m center-to-center spacing. It was done by circulating grout throughout the length of the hole in 5 m stages and a downstream direction. The pressures employed were 4 kg/cm^2, 6 kg/cm^2, 7 kg/cm^2, 9 kg/cm^2, 12 kg/cm^2, 16 kg/cm^2, 18 kg/cm^2 and 20 kg/cm^2 at depths of 0-5 m, 5 m-10 m, 15 m-20 m, 20 m-30 m, 30 m-35 m, 35 m-40 m, and 40 m-45 m respectively. Figure 3.2.10 shows the profile of the consolidation grouting, curtain grouting and drainage curtain.

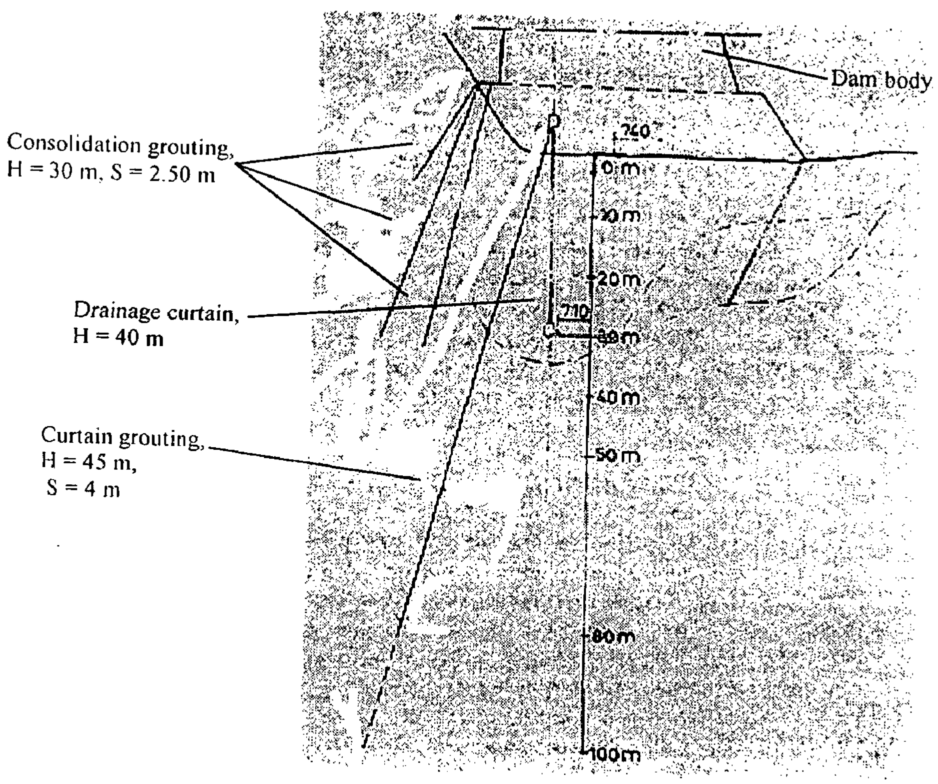


Fig.3.2.10: El Atazar Dam.
Impermeable Treatment

3.2.7. Wujiangdu dam

a. Project feature

The Wujiangdu hydropower project is located on the river Wujiang in Zunyi County, Guizhou province, southwest China. The construction of the main structure was started in 1976 and this project was completed in December 1982. The project includes a dam, a powerhouse, several outlet structures and other auxiliary structures. The Wujiangdu dam is a concrete arch gravity dam. The outlet structures composed of surface overflow spillway, spillway tunnels and spillway outlets.

The high concrete arch gravity section has a maximum height above the deepest foundation level of 165 m, a crest curved length of 396 m, 45 m of crest width, maximum central angle of 26° and 1,930,000 cu.m volume of concrete. The layout adopted is a superposed type, that is, the powerhouse, spillway and switchyard are all located in the valley but at different elevations, one on top of the other.

b. Foundation conditions

As in many other dam sites in China, the Wujiangdu site lacked sufficient space to accommodate the spillway and the powerhouse in the narrow valley. The width of the river channel in only 70 m during dry season, and the valley width at full reservoir level is only about 350 m

The main dam is founded on Triassic limestone formations with a stratigraphic thickness of 230 m. Karstic solution cavities are extensively developed in the limestone strata. Horizontal as well as vertical solution channels were encountered by investigation borings in both abutments. Deep cavities of considerable size were found down to 260 m below the river water level.

There is a shale stratum below the limestone strata, which was initially considered as an impervious layer capable of ensuring the water tightness of the dam foundation. Later investigation revealed that the shale layer had been broken by major fault.

The problem of treatment of karst foundation in dam construction becomes very important in karst regions. First of all, the location, thickness, size and depth of karst caverns should be identified through investigation. The total volume of karst caverns on the abutments is 800,000 cu.m. On the left abutment, the caverns extend along the fault F_{20} to 200 m deep under the riverbed. The maximum height of a karst caverns reaches 50 m. Most of the karst caverns, channels and fissures are filled with mud, but few of them filled with sand and gravel.

c. Curtain grouting

Providing deep grout curtain was the main measure selected for this project among a number alternative measures. The curtain measuring 1,020 m long runs along the dam axis and then turns upstream to connect with the impervious lower wall of Leping coal seam on the right bank and with the lower wall of Shabaowan shale on left bank.

To resist the maximum water head of 145 m, the grout curtain was formed by three rows of grout holes in the riverbed portion and in both abutments, two rows in the banks and one row in localized areas. The grout holes were spaced at 2 m, and the space between two rows was 1.20 m to 1.50 m. The curtain extends 80 m deep in the bedrock. On the left bank, the deepest grout holes reached 200 m below the riverbed (Figure 3.2.11). The grouting pressure was 20 kg/cm² to 40 kg/cm² in shale areas, 30 kg/cm² to 60 kg/cm² in limestone areas.

The results of the examination were found to be satisfactory. The effectiveness of the grout curtain in treatment of foundation of karstic formation in limestone areas was well assured.

d. Treatment by concrete diaphragm wall

Concrete diaphragm wall for seepage control are commonly adopted for thick pervious foundations beneath dams or other river structures. In this project, on the right abutment a 2 m thick concrete cut off wall was constructed from el. 640.00 to el. 680.00 m. On both abutments, all the karst caverns adjacent to grouting galleries were excavated and backfilled with concrete.

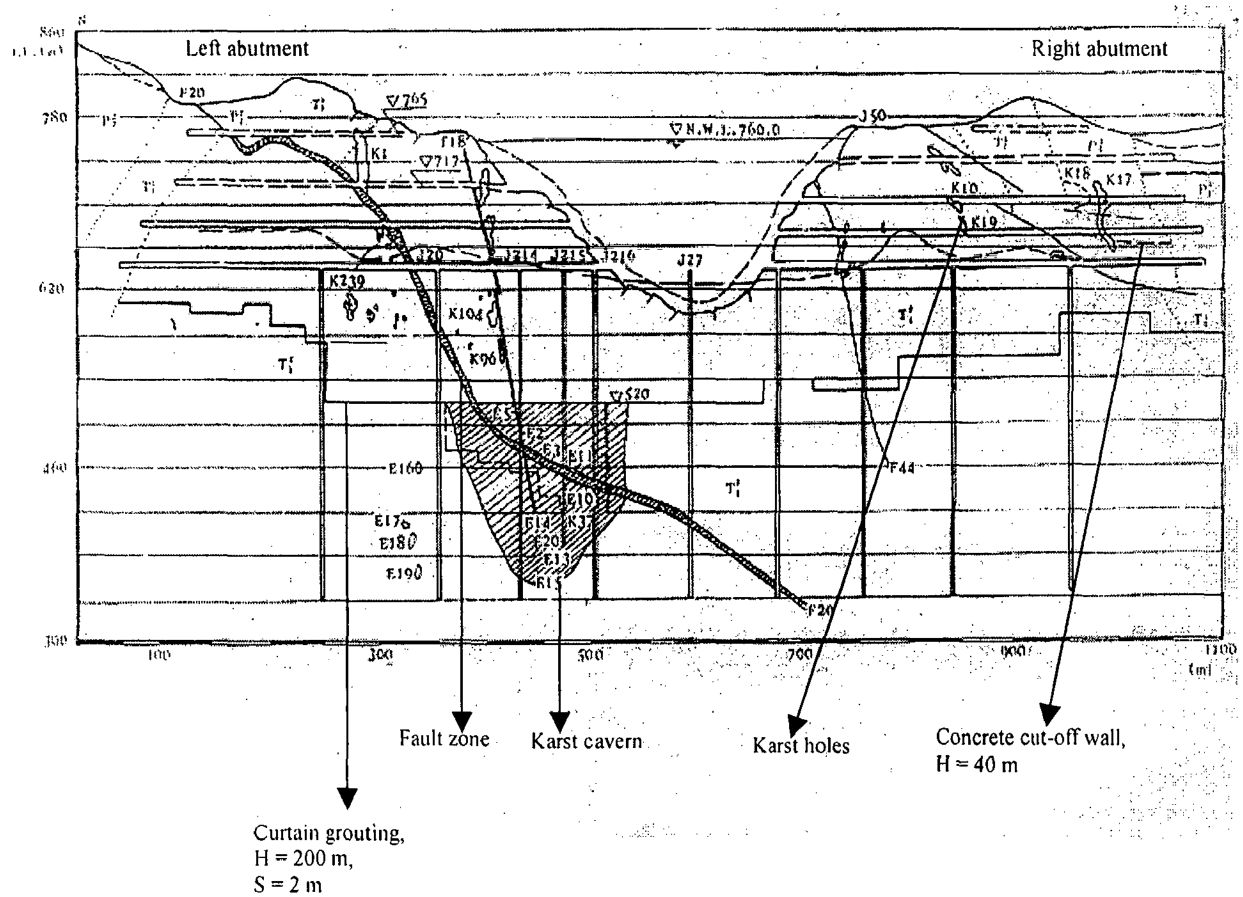


Fig.3.2.11: Wujiangdu Dam.
Curtain Grouting and Concrete Cut-off Wall

3.2.8. Srisailam dam

a. Project feature

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 downstream of the origin of the river Krishna. It is about 0.80 km downstream of Pathalaganga batching ghat near the famous shrine of Srisailam.

The project involves the construction of a masonry gravity dam of straight gravity type with an overall length of 512 m at road level of el. 274.32 m and a maximum height of about 144 m from the deepest foundation level of el. 131.06 m. The reservoir formed behind the dam has a storage capacity of 8,720 million cu.m at F.R.L of 269.75 m. The spillway portion is 266.40 m long having 12 bays of 18.30 m clear span each. The discharging capacity of the spillway is 37,380 cumec. Non-overflow of the dam is on either side of spillway portion.

b. Foundation conditions

The course of river Krishna near Srisailam dam site runs in a deep gorge, which lies in the northern extension of the Nallamalai Mountains. The width of the river at the bottom of the gorge is about 183 m of which the deep course of the river is towards the left margin over a width of 76 m. The bed of the river is at el. 152.40 m.

Quartzites and shales form the main foundation beds in the region which are near horizontal dipping at about 5° to 8° towards the right bank. At the dam site, the abutments of the river gorge comprise quartzite rock with intercalations of red ferruginous shales. Boulders to a depth of about 20 m are found in the deep river course below which the foundation consists of alternate sequence of quartzites and shales.

The following weaknesses were revealed during excavation of the foundation in the deep river portion, necessitated a review of the dam section.

i. Occurrence of a patch of soft and broken rock at the heel, necessitating deeper excavation and removal of the unsound material.

ii. A shale seam underlying the quartzitic formation and dipping towards the upstream at an angle of 3° to 5° with the horizontal.

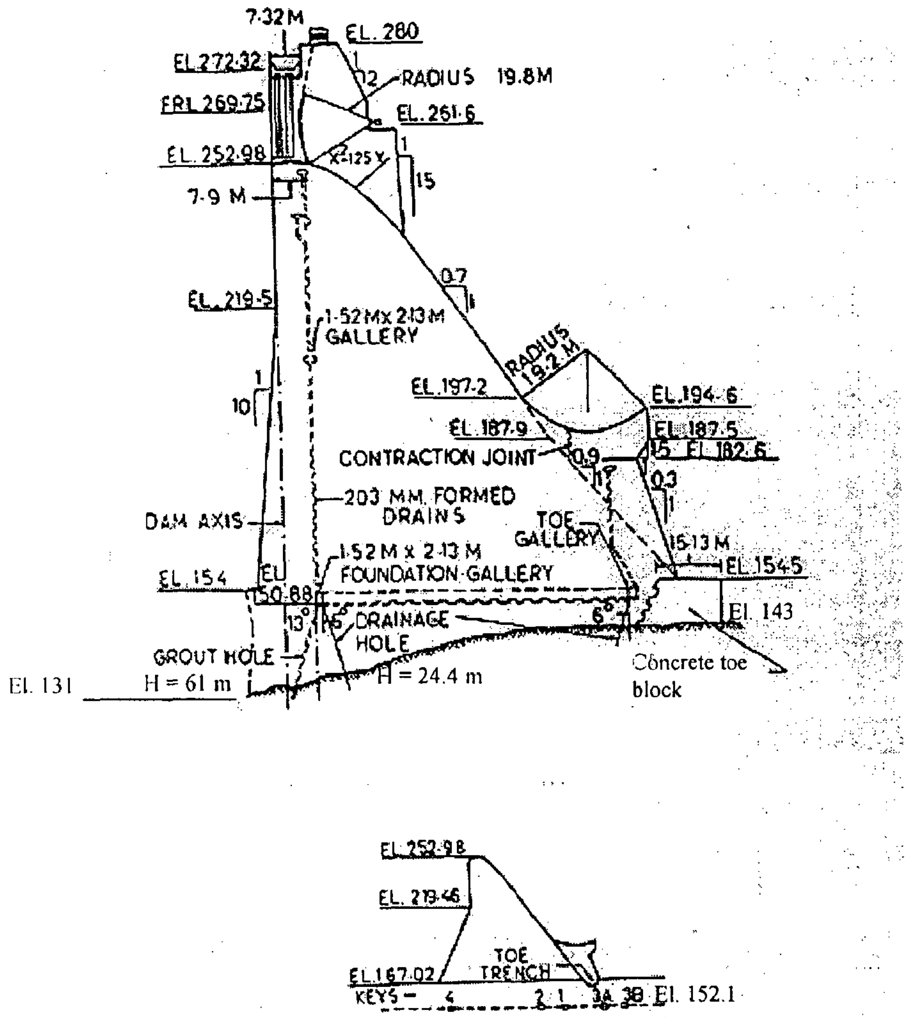


Fig.3.2.12: Srisailam Dam.
Curtain Grouting, Drainage Curtain
and Concrete Toe Block

3.2.9. Supa dam

a. Project feature

Supa dam is located across river Kali, 2 km downstream of Supa town in Karnataka, India for Hydropower generation by Karnataka Power Corporation Limited. It is a concrete gravity dam of 101 m high and the length of the crest is 350 m. The gross storage capacity at full reservoir level of 564 m is 4,354 hm^3.

The spillway consists of 3 spans of 15 m each, fitted with 10 m high radial type crest gates. The powerhouse is located on the right side and has an installed capacity of 100 MW (2 units of 50 MW each).

b. Foundation conditions

The foundation rock at the dam site consists of predominantly of banded calcareous magnetic quartzite. The river gorge is cut through a strike ridge of calcareous magnetic quartzite and hematic chert/quartzite. Calcareous magnetic quartzite and hematic quartzite interlayered with phyllite bands and intruded by 3 m – 5 m thick longitudinal and 15 m thick transverse dolerite dykes constitute the foundations. The foliation/bedding strikes in NNW-SSE direction and dips at moderate angles in either direction due to intricate folding. The rock mass exposed in river bed is fresh and suitable as compared to that in abutments, which is highly jointed and weathered at places.

The significant feature is that the steeply dipping dykes, which are found to be fresh in the river bed, are highly weathered and are reduced to pulpy clay at higher elevations in flanks. In addition, there are numerous zones of weathered and weak pockets/zones of varying dimensions at heel and toe of dam blocks, especially in the left flank blocks.

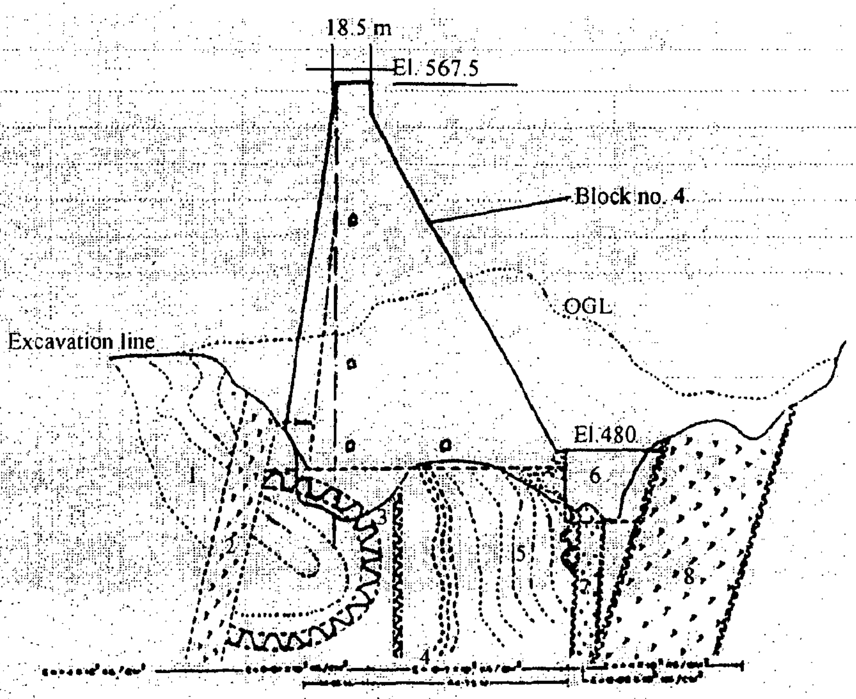
c. Treatment of weak zones

The heel portion of block 7 and 8 consists of weathered and soft bands of manganese. The foundation of block 4 consists of 3 m to 5 m wide shear zones both at heel and dam toe. The modulus of deformation of these portions is low. It was apprehended that the presence of the weak zones might result in undesirable differential settlements and possible formation of vertical crack in the dam blocks.

The treatment of the weak zones in block 7 and 8 was by transferring the load to shoulder rock by providing a concrete strut. The strut had to be founded against competent material met with at a large distance both in the upstream as well as downstream. The dimension of the strut is 9 m thickness and 20 m long.

Under reservoir full condition, the strut would be subjected to tensile stresses, which might cause cracking of the strut. To avoid such cracking, a joint was provided between the body of the dam and the strut.

In case of block 4, the block was designed with an upstream overhang to restrict the base width to the central good rock portion of 54.50 m wide against the normal required of 80 m. In order to improve the resistance against sliding, a concrete strut was provided on the down stream between the toe and the hard dolerite dyke about 25 m – 30 m away from the toe of the dam as shown in Figure 3.2.13. It was constructed by excavating/drifting and backfilling with concrete. The face area of the strut was 20 m wide and 10 m deep.



Legend:
1. Weathered magnetic-quartzite
2. Weathered dolerite dyke
3. Shear zone
4. Phyllite band
5. Calcareous magnetic-quartzite
6. Concrete strut
7. Offshoot dyke
8. Hard dolerite dyke

Figure 3.2.13: Supa Dam.
Concrete Toe Strut

3.2.10. Kadana dam

a. Project feature

Kadana dam across river Mahi in Gujarat, is a composite dam with a maximum height of 66 m for the masonry (spillway) portion.

b. Foundation condition

The foundation rock comprises interbanded quartzites, quartz-mica schist and phyllites with thickness varying from 0.50 m to 3 m. The rocks are highly jointed. Shearing along the bedding plane is also recorded at many places. Minor reverse faults intersect the rock formations accompanied by brecciation, veining and some small displacements. In the foundation under spillway block 2 to 15, quartzite is predominant while quartose and schistose phyllites occupy the rest of the masonry dam foundation including that of the powerhouse.

c. Foundation treatment

Dental treatment was carried out including removal of the fault gouge and fractured rock from the hanging wall above the fault trace up to an average depth of 1.20 m and backfilling with concrete or rich masonry. Further, the fault zones were treated by intense washing followed by grouting through additional rows of 15 m deep holes.

In power dam block 2 to 5, fault with gentle upstream dip got exposed on the down stream beyond the toe. The required factor of safety was achieved by improving the shear resistance through provision of shear keys by driving drifts below the dam as shown in Figure 3.2.14.

In the spillway block 8 and 9, two faults dipping 25° to 30° towards downstream and making angles of 70° to 75° with the dam axis were detected during excavation of the foundation below blocks 8 and 9.

The treatments for improving sliding stability comprised:

i. Removing hanging wall of the fault above its footwall in an average width of 10 m from the bucket area in a length of about 24 m and backfilling with concrete.
ii. Providing a key at the downstream edge of the bucket down to the foot wall of the fault.

iii. Combining blocks 7, 8, 9 and 10 to strengthen the weaker blocks 8 and 9 as blocks 7 and 10 rest on comparatively strong rocks.

iv. Consolidation grouting beyond the bucket and sill over a distance of 30 m in blocks 7 to 10 to improve the scouring resistance and ensure cover over the low angle faults.

v. Provision of concrete key through drifts below the foundation along the fault planes to improve shear resistance against sliding.

vi. Provision of relief wells on the downstream of the end sill to drain the seepage water.

In spillway blocks 15 to 19, the low angles fault traverses the foundation from the upstream to the downstream and changes its dip direction near the toe of the dam. The treatment of this fault comprised:

i. Excavation of two drifts of 4.50 m – 6 m wide running parallel to the axis of the dam along the fault near the toe of the dam in blocks 16 to 19 and backfilling with concrete to improve sliding resistance.

ii. Provision of drainage along the fault by drilling drainage holes and connecting them to the longitudinal gallery near the toe of the dam.

iii. Covering the low angle fault on the upstream by an impervious blanket in a width of 60 m.

iv. Provision of a 15 m wide and 1 m thick concrete apron beyond the end sill to check retrogression of the downstream trace of the low angle fault.

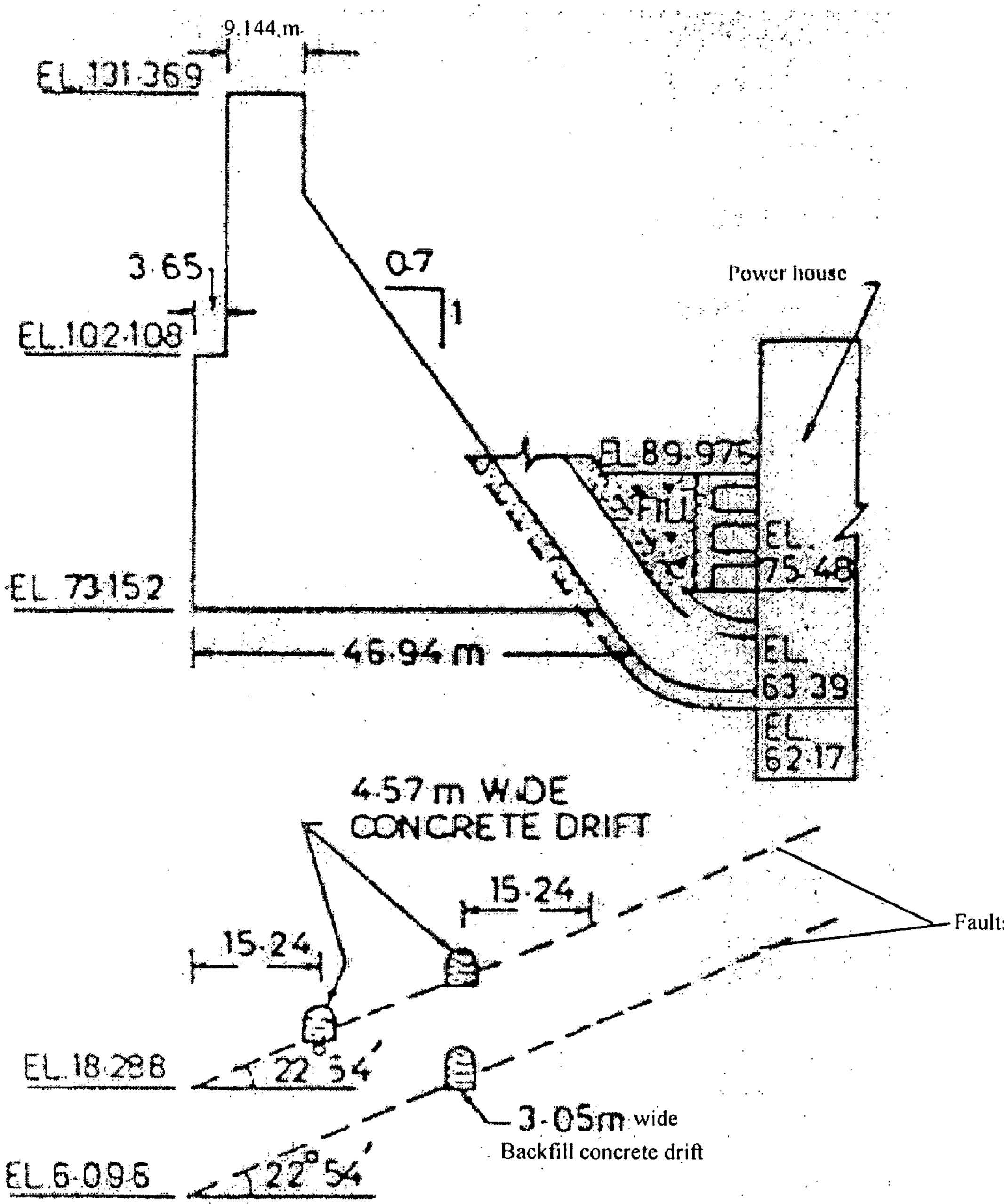


Figure 3.2.14: Kadana Dam.
Treatment of Faults by Shear Keys

3.3. Special Foundation Treatment

3.3.1. Ravasanella dam

a. Project feature

The Ravasanella dam has been constructed across the river Ravasanella in proximity of the town of Castelletto villa in the province of Vercelli – Piedmont, Italy. The reservoir has a storage capacity of 5 million cu.m is to be used primarily for irrigation purposes and to assure rice cultivation over a wide area of Piedmontese plain on the left bank of the river Sesia.

The basic design of the dam was subjected to a major change brought about by unforeseen geomechanic circumstances, which occurred during excavation operations. The first project envisaged a structure of a concrete gravity dam of 49 m high and 202 m crest length consisted of 14 blocks. The explorations, which were carried out during the excavation works, revealed that the geomechanic characteristics of the rock, especially on the right abutment, were much poorer than what was supposed from the preliminary results. A dam of mixed type, which best suited the different characteristics of the foundation rock on the two banks, was finally developed through detail comparative studies.

The mixed type dam has a crest length of 226 m comprising a 97 m long non overflow gravity dam on the left bank, a 36 m long overflow gravity dam in the central part and a 93 m long dam in concrete block type on the right bank. The gravity dam extends to a maximum height of 52.70 m, while the concrete block dam reaches a height of 49.40 m. The layout of the dam is shown in Figure 3.3.1, and the cross section of each type is shown in Figure 3.3.2.

b. Foundation conditions revealed from investigation before construction

The Ravasanella catchment area and the dam site are located in the volcanic formations of the quartz porphyry, in the Biella surroundings, belonging to the hercynic magma cycle (early Paleozoic). The volcanic structure near and around the dam site consists of a mass of non-uniform lithotypes, predominantly pyroclastic and subordinately lavic.

Particularly, from both a petrographic and geological-technical point of view, the various pyroclastic lithotypes fall under two groups, namely fine grain ash tuffs, and coarse grain rhyolitic tuffs. The lava lithotype, classified as dacite, only forms very small dykes, unimportant in technical terms.

The volcanic tuffs are affected by a mesh of fault and fractures, of varying orientation and spacing, connected with the alpine orogeny and subsequent postorogenic phenomena.

The subsurface explorations (soundings and trenches) carried out during the preliminary planning showed ash tuffs on the left bank and rhyolitic tuffs on the right one. Both lithotypes were randomly distributed in the bedrock of the valley bottom. Nonetheless, the rock appeared to be mostly fresh and moderately jointed.

Furthermore, the same investigation revealed, at the base and top of the right bank, the presence of several areas characterized by rock up to a cataclastic state and/or altered by the partial claying of components. The average geomechanic characteristics of the bedrock, however, appeared sufficient in relation to the stresses transmitted by a mass gravity concrete dam. The first project, therefore, envisaged structure of this type with the dimension mentioned above.

New explorations were carried out when construction work began. These ones included drillings, bore pressure tests, Lugeon permeability test, injection tests and laboratory tests on sample core. Excavations for the dam foundations were begun at the same time. It very quickly became evident that the geomechanic characteristics of the rock, especially on the right abutment, where much poorer than what was supposed from the preliminary results.

Along the excavated surfaces on the right abutment, the rock in fact appeared to be affected by very strong fragmentations of the order of centimeters or less, sometimes visible and sometimes hidden and only revealed by dynamic interventions, in addition to the numerous local tectonic discontinuities (small faults and fractures). Chemical-mineral alterations of components were widespread in this mass.

Moreover, both abutments showed major fault. The first one (F_0) was subvertical and run almost parallel to the river, at the foot of the right bank. The second one (F_{25}), not as significant as the first, ran alongside the left bank at the same dip (N-20°-W) at an inclination of 45°.

In order to clarify this situation further, new specific investigations were carried out including seismic exploration and an exploratory tunnel excavated in the right bank. Plate load tests were carried out to determine the modulus of deformation of the rock mass.

c. *Foundation conditions revealed from investigation during excavation*

The results of the various explorations during the foundation excavation works have made it possible to update the physical-mechanical characteristics of the bedrock. Schematically this rock mass can be considered as being divided by the two above mentioned major faults in the three sectors, roughly corresponding to each of the two abutments and valley bottom respectively.

The left abutment mainly comprises ash tuffs of stony consistency in stratified banks dipping opposite to the slope at an angle of 35° to 45°. The rock has a moderate amount of fractures, and the joints (facing various directions and generally closed) are spaced out at intervals of several decimeters. The alteration phenomena are few and, overall, unimportant. The ISRM geomechanical classification (Sharma, H.D., 1998) is RMR = 40 to 60. It means that the class of the rock mass is class no. III or fair rock.

In valley bottom, the bedrock is rather heterogeneous due both to a complex alteration and interaction between rhyolitic tuff and ash tuff banks and to the two major faults mentioned above. On the whole, the area bordering on the left abutment has very similar characteristics to the left bank itself. In the central area, the rock appears to have slightly higher degree of cracking, with areas of limited alterations. A RMR classification of around 40 was estimated.

The situation appears much worse in proximity of the main fault at the foot of the right abutment where for a width of 10 m to 20 m, the rock is crushed, laminated and altered as a result of claying phenomena.

In right abutment, the bedrock was found to comprise rhyolitic tuff in stratified banks dipping opposite to the slope, inclined at an angle of 30° to 65°. The deepening of the excavations, which was established to reach the rock mass at a depth at which the seismic wave velocity measured approximately 3.80 km/s, did not provide all the expected result.

In fact, while on the one hand the rock at this new foundation depth appeared to have a considerably smaller number of fault and fractures, areas presenting widespread alteration phenomena, with a particularly high degree of breakdown, still remain. After the excavation, that is to say following the absence of mechanical co-a action, slices of rock came free and soon formed an extensive detritus overburden.

The right abutment generally appears to comprise significant portions of sound rock and portions, which, on the contrary, have acquired the appearance and behavior of loose, weakly consistent material.

The high breakdown of the rock structure obviously means a reduced shear strength and a deformability of the rock mass at an average higher than the result obtained by the plate load tests, which turnout to affect one of the healthier portions.

d. Solution

The result of the investigations summarized in the previous paragraph confirmed that the geomechanical characteristics of the rock on the left bank and to a large extend that of the riverbed where such to constitute a suitable foundation for a mass gravity dam, but doubts arouse regarding the right abutment and area affected by the F_0 fault.

The problem derived, above all, from the presence of relatively significant and widespread rock alteration phenomena, which made reclamation and consolidation operations (essential to reduce the deformation characteristics and increase the shear strength of the rock mass), extremely expensive and unreliable.

An alternative solution was therefore considered, which apart from improving the foundation, would enable a structure to be built for this part of the dam ensuring :

i. Enhanced deformability.
ii. Improved distribution of the vertical stresses on the foundation.
iii. Reduced variation of the stresses transmitted to the foundation when varying loads acting on the dam (empty reservoir-full reservoir).
iv. Enhanced safety factor against sliding.

It was possible to demonstrate, by detailed comparative studies, that the concrete block dam was the most viable solution, from a technical and economical point of view. The following major advantages, with respect to an earthfill or rockfill dam, are offered by this kind of structure :

i. Fewer difficulties in connecting the gravity dam to the deformable one.

ii. The construction equipments are the same, thus resulting in reduced costs and organization difficulties.

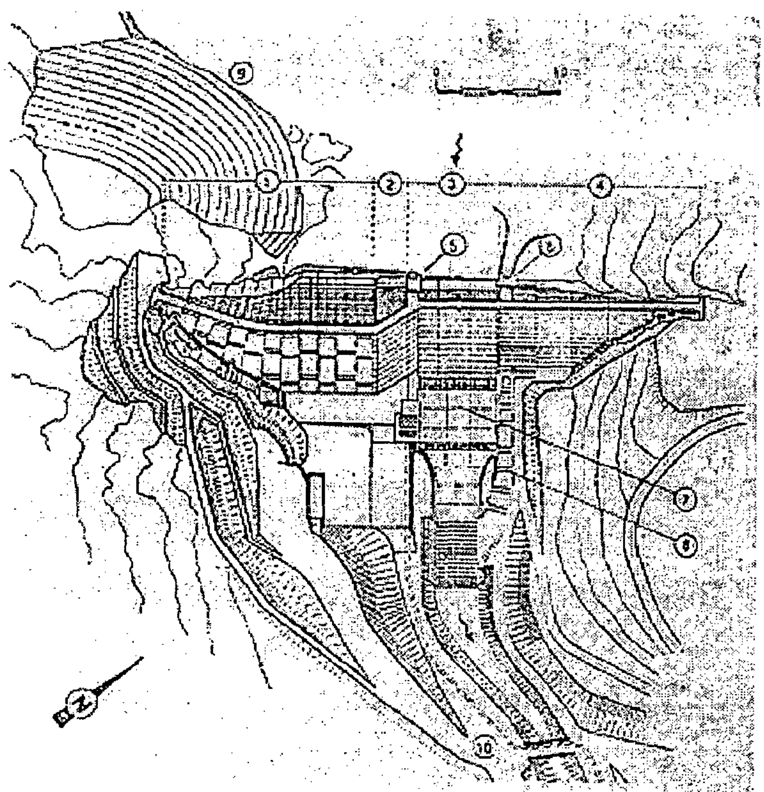
These advantages were particularly evident in the case of the Ravasanella dam situated in a narrow valley, where the connection between concrete structure and earth embankment was difficult to achieve. A dam of mixed type, which best suited the different characteristics of the foundation rock on the two banks, was therefore developed. The design entailed the construction of :

i. A gravity dam on the left bank and across a significant portion of the riverbed.

ii. A concrete block dam on the right bank and across the F_0 fault area.

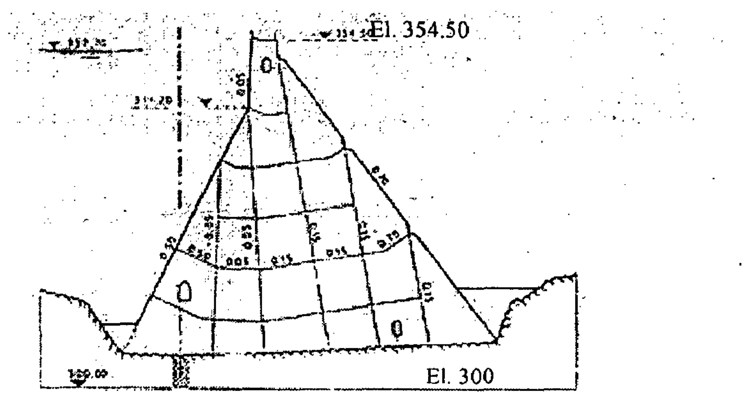
The two different structures ware connected by interposing a suitably shaped gravity transition element. The planimetric configuration of the concrete block dam was designed to use the grout curtain built during an earlier stage of the work. An original concave structure on the upstream side was the result. This situation is not detrimental to the conditions of stability of the dam, designed as a connection of independence buttresses.

Briefly, the structure of a concrete block dam comprises columns of concrete blocks. The columns are contact-poored so as to form the real buttress, which are made independent by lubricated joints (filled with gravel of a single grain size) and therefore, able to absorb any differential settlement of the foundation. A steel membrane on the upstream face ensures waterproofing of the dam.

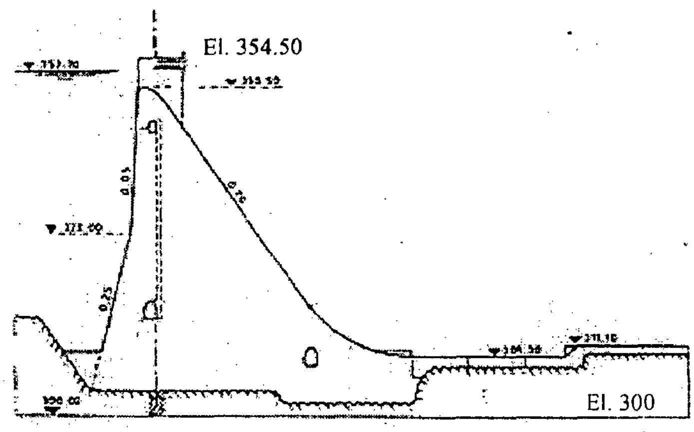


Legend:
1. Concrete block dam
2. Transition element
3. Overflow gravity dam
4. Non-overflow gravity dam
5. Draw-off tower
6. Bottom outlet
7. Stilling basin of spillway
8. Stilling basin of bottom outlet
9. Rockfill embankment (side protection)
10. Irrigation pipe

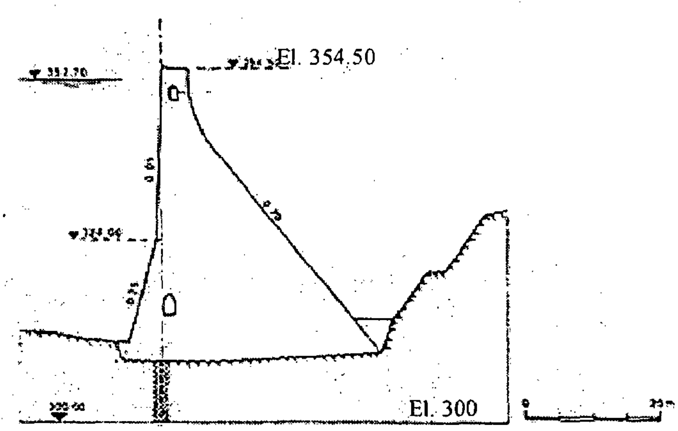
Fig. 3.3.1: Ravasanella Dam.
Layout of Composite Dam



Concrete block (NOF) section



Concrete gravity (OF) section



Concrete gravity (NOF) section

Fig. 3.3.2: Ravasanella Dam.
Cross section of Concrete Block and
Concrete Gravity Dams

3.3.2. San Giuliano dam

a. Project feature

The San Giuliano dam on the river Bradano is built across the river at about 30 km from its mouth and creates a reservoir with storage of 10,700 million cu.m, which is used solely for irrigation and flood control. The dam is a gravity dam of 44 m high from the foundation level and 79 m high from the sub foundation works.

b. Foundation conditions revealed from investigation before construction

The San Giuliano gorge is a deep ravine with steep sides, carved out by river Bradano running through Pliocene tufaceous limestone formations and, under these, the Cretaceous limestone. The very first check boreholes showed that the limestone, which is highly fractured and karstic in some places, is permeable. Thus as early as the design phase, intensive works for consolidating and sealing these formations were foreseen.

In final phase, these works proved to be much more complex than expected, requiring cement grouting with about 3 q per meter of hole, which is the highest rate of absorption ever recorded in Italy at that time. At the end of the treatment, the permeability of the check boreholes was found to be practically nil.

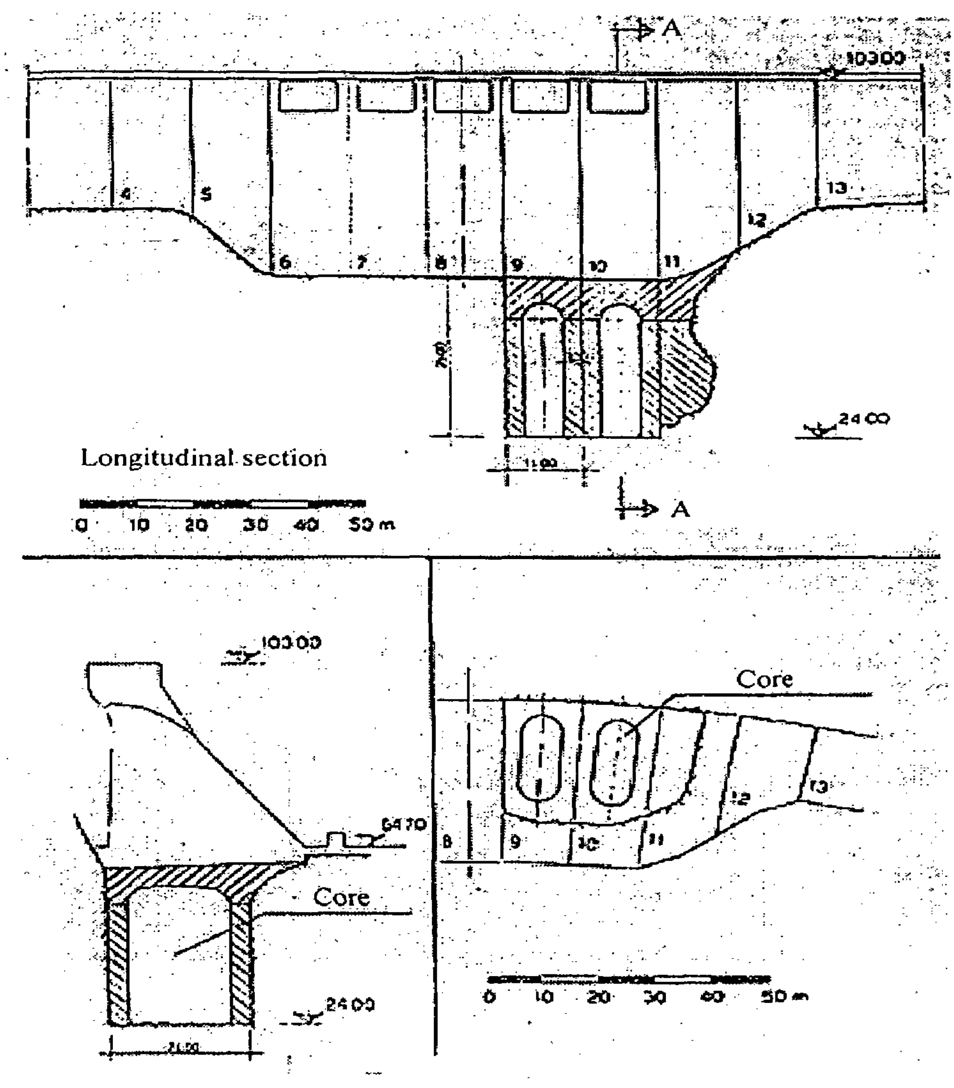
c. Foundation conditions revealed from investigation during excavation

The worst surprise came after the excavations were almost finished. A huge cavity was found full of cataclastic, mottled materials, highly clogged with clay. This cavity, which the boreholes had not sufficiently revealed, is part of the general panorama of natural limestones that are highly disturbed. It can be considered as an ancient karstic phenomenon of the limestone that very probably was established where the two fractures cross, before the Pliocene tuffs were deposited. The cavity was filled with a calcareous mass coming either from the sides or from the ancient roof of the cavity when it partly collapsed.

d. Solution

Because of the large quantity of clayey materials present, it was impossible to try and consolidate the cavity by cement grouting, similarly the size of the cavity meant that it could not be completely emptied. Thus it was decided to construct a sub foundation, as shown in Figure 3.3.3.

The measures adopted (sealing and sub foundations) have proved totally successful. The dam has been operating since 1957 and neither the stability of the structure nor the sealing of the dam site have given any cause for concern. Moreover, the Reclamation Authority of Bradano and Metaponto, the concessionary of the works, has judged the operation of the dam to be excellent. Recently, the idea of heightening the dam by 3 m to 5 m has been considered. However, the project is currently at a standstill because of a reluctance to submerge farmland and houses as part of the enlargement of the reservoir.



Section A-A

Plan

Fig. 3.3.3: San Giuliano Dam.
Sections of Masonry Subfoundation

3.3.3. Bhakra dam

a. Project feature

A mammoth multipurpose project, called Bhakra Nangal project has been constructed on the Sutlej in northwest of India for Irrigation and power generation. The project comprises a 225.60 m high concrete gravity dam with two powerhouses having an aggregate installed capacity of 1,050 MW, a 27.40 m high dam for after-bay regulation and diversion, a 64 km long power canal with two powerhouses of aggregate installed capacity of 154 MW and a network of irrigation canals which command an area of 2.63 million ha in the state of Punjab, Haryana and Rajasthan in addition to improving irrigation on the existing Sirhind canal system.

The dam is a straight gravity concrete structure, utilizes fully the power potential and water supplies available. The full reservoir level was kept at el. 512.07 m and maximum reservoir level at el. 513.59 m to start with. During construction, it was decided to raise the maximum to el. 514.50 m successfully since then. The lake created, extends about 97 km upstream of the dam and covers an area of 168 sq.km. The full reservoir capacity amounts to 986,400 ha.m.

b. Foundations conditions

Bhakra dam is located in the outer foothills of Himalayas, called Shiwalik hills. The dam site and the river canyon are underlain by rocks, chiefly the Shiwalik formation (Miocene to L. Pleistocene) of sandstone and clay beds with thick conglomerate forming upper portion of the stratographic column. The area gives evidence of great deformation, faulting and folding of tertiary strata.

Bhakra canyon has been eroded through an up thrust and tilted series of Shiwalik beds which dip 65° to 74° downstream and trend roughly at right angles to the river. The dam foundation area is made up predominantly of sandstone sandwiched within three large clay-stone bands.

The rock, although comprising mainly of these well-defined sandstone and clay features is intervened by erratic formations of silt zones, shear zones and cross shear zones, which have rendered very complex foundation treatment.

Three clay bands, i.e. the heel clay stone band, the middle clay stone band and the downstream clay stone band traverse the foundation area. The downstream clay band does not affect the stability of the dam in any way. However, the left power plat is founded on it after providing suitable raft foundation. These three clay bands though capable of carrying great loads in a confined and unweathered stage become plastic when submerged and disintegrate under conditions of alternate drying and wetting. As such suitable steps were taken to treat the heel and middle clay bands as well as the shear zones as described hereafter.

c. Treatment of heel clay band

The heel clay band is 30 m to 45 m wide and dips at an angle of 70° downstream with the nearest point 22.50 m upstream of the heel of the dam. It recedes away from the dam at higher elevation. It was considered that on account of continued submergence under great depth of water, this band might become plastic and give way under the load of the dam. It was, therefore, decided to excavate the clay band in the river section to a suitable depth and backfill with concrete. Furthermore, a concrete strut of adequate thickness, spanning the clay band was constructed for transferring the loads to the upstream sandstone called footwall (Figure 3.3.5).

Photoelastic experiments further indicated that if the bottom of the plug was sloped parallel to the strut, tensile stress in it were eliminated. This fact caused the upstream end of footwall contact to be depressed to el. 292.60 m (22.50 m deep).

Two longitudinal galleries, which were connected to the dam by means of three transverse galleries) were provided within the plug following the upstream and downstream clay sandstone junction for grouting the contact and consolidating disturbed sandstone beds to a depth of about 15.20 m. Water testing in these zones in 1964 showed good water tightness. For side slopes above the riverbed, an adequate concrete cover was provided to confine the clay stone members.

d. Treatment of middle clay band

The middle clay stone band varies in width from 6 m to 9 m and is completely confined by the dam except where it passes out from under the dam in the left abutment. It is treated by excavating the same and backfilling with specially precooled concrete with placing temperature of 10° C to 12.80° C. The trenching depth was based on formula determined by USBR for Sastha dam as discussed in Chapter II.

The sandstone contacts were specially grouted. A special feature for the middle clay stone is that a supplementary longitudinal drainage, which runs just upstream of the middle clay, has been provided with a view to releasing any water pressure, which may pile up against the almost impervious clay band.

e. *Treatment of axis shear zone*

This shear zone lies mostly in the body of the dam within 30 m of the axis from station 3.70 m to station 5.20 m. On right abutment, it deviates upstream to merge into the thrust zone at surface elevation of 472.00 m. In width, this shear zone bands varied from 3 m to 6 m. It was necessary to treat this zone to prevent further disturbance of abutment rock in proximity or under the dam (see Figure 3.3.5)

The treatment consisted of:

i. Cold concrete cap with the to depth formula of 1.50 to 2 times the width, subject to a minimum of 3 m up to el. 518.20 m (top of dam).

ii. Consolidation grouting of adjoining rock on both sides of the zones and provision of adequate subsurface drains where cap is thin to prevent uplift pressure as a consequence of rapid draw down.

f. *Treatment of toe shear zone*

This zone runs along the toe of the dam on the left abutment and varies in width from 6 m to 18 m. The treatment of this bed rock feature in the area where substantial toe thrust is imposed by the loaded dam, consisted of plugging with concrete as per standard formula, supplemented by a fillet to rock on the downstream side of the zone when practicable.

g. *Treatment of thrust zone*

A thrust zone consisting of gouge and sheeted rock of width varying from 0.60 m to 4.30 m crosses the right abutment obliquely from el. 408.40 m on the downstream side to el. 457.20 m at the axis of the dam. On the downstream side of the abutment area but within the section of dam, the thrust zone joints into a clay stone band.

The defective thrust zone is important as there was the chance of settlement of overlying rock and section of dam above, due to consolidation and squeezing under combined load of dam and reservoir water. Treatment necessary to prevent undesired settlement either as result of load, or in combination with effects of piping through seepage was as follows:

i. Percolation of reservoir water along the thrust zone was stopped by a tunnel of 30.50 m driven in line of the principal grout curtain into the abutment, backfilled with concrete and contacts grouted.

ii. Prevention of squeezing out of disturbed material was achieved by two plug tunnels, one 15.2 m deep along the zone under the heel of the dam and the second tunnel of 12.20 m deep along the zone at the toe of the dam.

h. Consolidation (blanket) grouting

Besides the special treatment of weak zones mentioned above, consolidation grouting of the entire foundation area (including area of 15.20 m upstream of the heel and 18.30 m downstream of the toe) has been thoroughly done to a normal depth varying from 9.10 m near top of the dam to 15.20 m below el. 426.70 m.

The consolidation grouting was done by pressure grouting. The criteria of completion was to achieve water loss of less than 1.49 liter per minute per meter length of the hole at the specified grout pressures in a diagonally drilled test hole. The maximum pressure applied did not exceed 7.03 kg/cm^2.

i. Curtain grouting

The curtain grouting was finally completed by holes at 1.5 m centers, initially, as a first sequence, the holes space was 3 m centers and grouted (Figure 3.3.4). With a view to avoid cracking of the concrete, pressure for the first stage of 15.20 m was limited to 24.60 kg/sq.cm.

j. Foundation drainage

The main drainage system provides relief through a series of vertical holes of 7.50 cm diameter just downstream of main grout curtain. These holes were drilled at 3.10 m centers in the dam area and 6.20 m apart in drainage tunnels in abutments where there is no deep grout curtain.

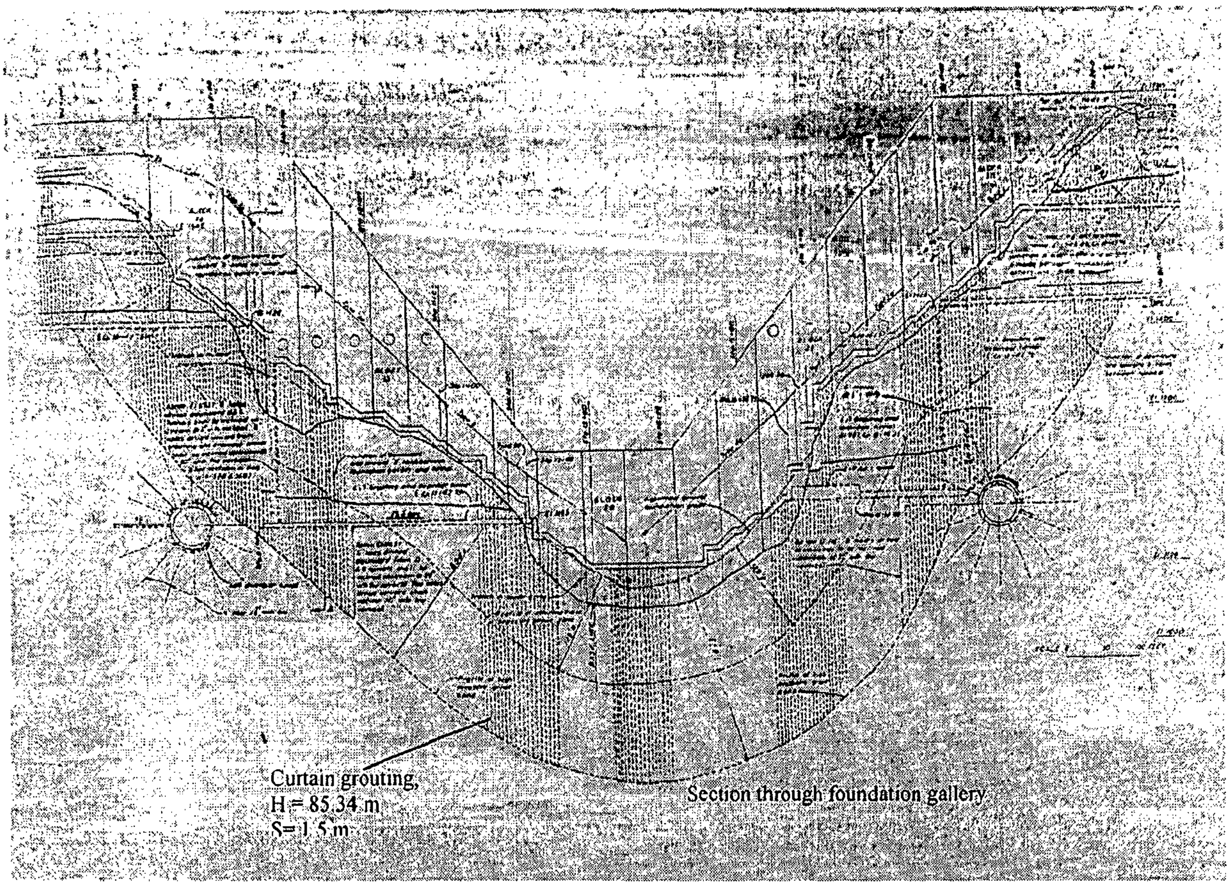


Figure 3.3.4: Bhakra Dam.
Curtain Grouting

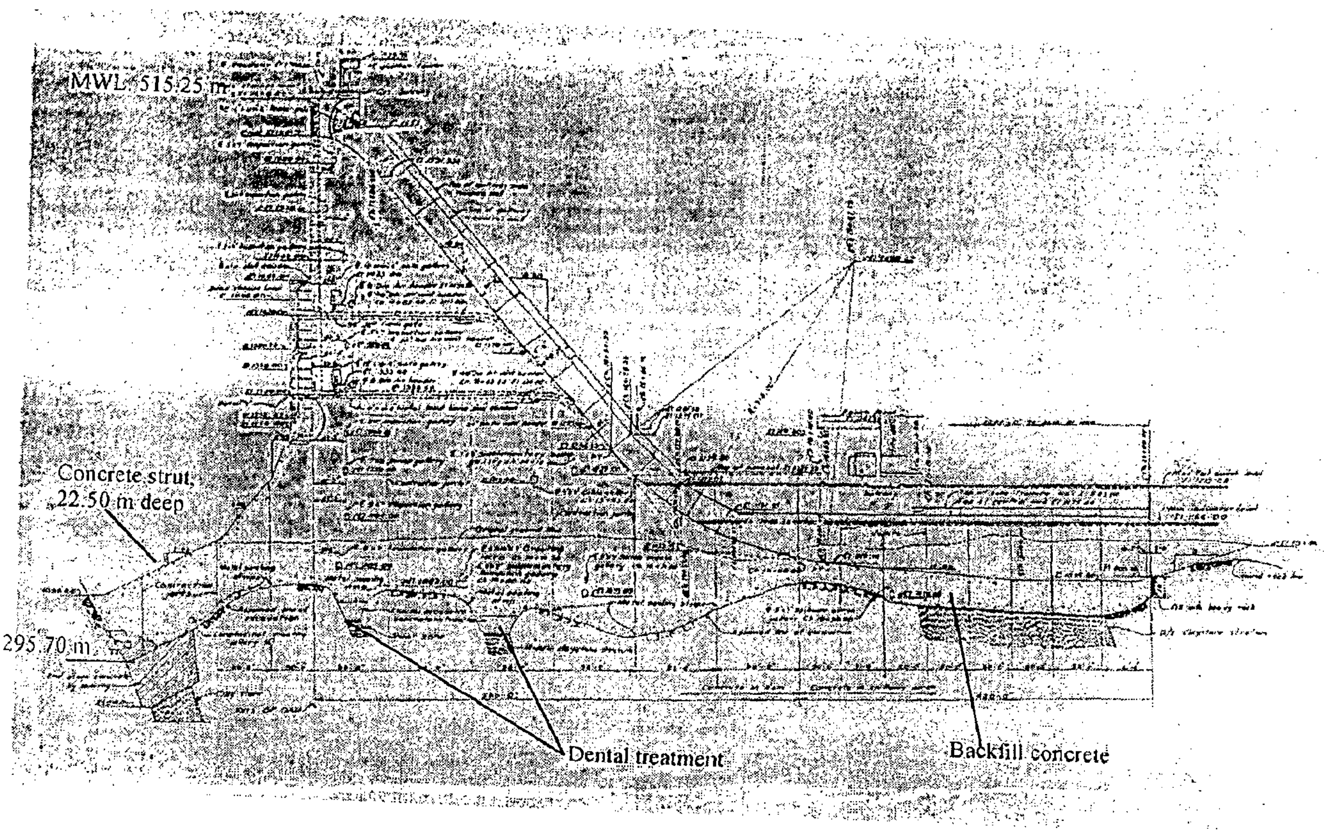


Figure 3.3.5: Bhakra Dam.
Treatment of Weak Zones

3.3.4. Itaipu dam

a. Project feature

The Itaipu project area is located in the Parana basin, between Brazil and Paraguay, at about 22° latitude south. The dam is located 14 km upstream of the international bridge connecting two towns of Iguacu in Brazil and President Stroessner in Paraguay. The reservoir extends northward about 150 km upstream up to Guaira (Brazil) and Salto del Guaira (Paraguay), with normal water level at el. 220.00, water surface area of 1,350 km^2 and a gross storage volume of 29 km^3.

The project consists of a dam of 7,772 m long crossing the river with the crest at el. 225.00 m and maximum height of 196 m includes several types of dam, namely:

i. Concrete hollow gravity dam of 612 m long.
ii. Concrete buttress dam of 1,450 m long.
iii. Concrete gravity dam of 532 m long (main dam).
iv. Rock fill dam of 1984 m long.
v. Earth fill dam of 3194 m long.

The spillway with a 61,000 cumec spilling capacity for reservoir levels at el. 223.00 m is located on the right bank at about 1 km downstream of the powerhouse.

b. Foundation conditions

Itaipu dam is located in the central part of the Parana sedimentary basin. Successive basalt flows from the Sao bento group (belonging to the Cretaceous period) constitute the rock foundation of the dam.

There are five basaltic flows in the area directly related to the project. They were denominated in an increasing order A, B, C, D and E (A is the lower most) with variable thickness of 30 m to 70 m (see Figure 3.1.5). The mass rock of the river channel also has shearing zones with a general direction parallel to the river axis dipping either east or west, which seems to be caused by the action of horizontal compressive forces in the east-west direction. The basalt flows present three distinct types of basaltic rocks such as:

i. Dense basalt with high density of 2.95 approximately, deformability modulus of above 200,000 kg/cm^2 and highly fractured.

ii. Amygdaloidal vesicular basalt having a texture similar to the dense basalt with density of 2.6 to 2.7, deformability modulus between 100,000 kg/cm^2 and 150,000 kg/cm^2 and less fractured than dense basalt.

iii. Breccia and scoriaceous lava with the density from 2.1 to 2.4, deformability modulus is about 70,000 kg/cm^2, high porosity and high permeability of more than 0.001 cm/s.

The breccia layers between basalt flows has a thickness from 1 m to 30 m, always heterogeneous, usually weaker and more deformable than basalt flows. Sub horizontal discontinuities were also found, which have been formed by the erosion of the Parana River so that the unconfined rock mass had been slightly displaced toward the river channel under horizontal forces. The discontinuities that generally occur at the contact of the upper part of the breccia and the lower part of the dense basalt represent important horizons for the stability of the dam. These discontinuities usually are the weak zones and are the most pervious layers. The most significant of these features are:

i. A discontinuity in the basalt flow D at el. 125.00 m on the right bank intercepted under block F1/2 by a short part of exploratory and drainage tunnel. It is a joint with clay infilling, 10 cm to 30 cm thick with average $\emptyset = 30°$.

ii. A contact between lava flows C and D at el. 112.00 m on the right bank with a thickness of a few centimeters. It is filled with plastic clay with average $\emptyset = 25°$

iii. A discontinuity in the dense basalt flow B at el. 20.00 m in the river bed under the highest block of the dam with a thickness from 3 m to 5 m. It is presented as a bundle of oxidized or weathered fractures, with spacing of some centimeters or decimeters between fractures and with a deformability modulus lower than 70,000 kg/cm^2.

The most critical discontinuities from the deformability as well as from the resistance point of view are associated with the discontinuity at el. 20.00 m. In general, the contact between flow A and flow B is open and filled with weathered rock that varies from basalt to clay.

c. *Foundation treatment of main dam in the river channel*

The most extensive beneficial treatment was carried out under the foundations of the highest blocks of the main dam located in the main river channel. The treatment required the excavation of tunnels at el. 20.00 m, 15 m to 20 m below the dam foundation level, followed by backfilling with concrete of compressive strength of 280 kg/cm² at 360 days. With this treatment the sliding safety factor of the corresponding block was raised significantly counting with the shearing resistance of the concrete $\tau = 50$ kg/cm².

An additional purpose to this treatment is to prevent excessive differential settlement in areas of occurrence of thick deformable layers.

This treatment, as shown in Figure 3.3.6 and 3.3.7 consists of a grid of 8 shear keys parallel and 8 perpendicular to the axis of the dam in the foundation rock of four hollow gravity dam blocks. The size of the tunnel is 3.5 m wide and 2.5 m high.

The total length of the keys is 2,600 m and the total volume of concrete is 30,000 cum. The grid of shear keys is circumscribed by the perimeter drainage tunnel of 800 m long, excavated along contact A/B at about el. 20.00 m. It isolates the treated region so that uplift in the inner area would be reduced to a minimum.

e. *Grouting system*

The grouting system includes consolidation grouting, contact grouting and curtain grouting.

Consolidation grouting was executed where the rock appeared superficially shaken by blasting or where breccia occurred with characteristics of high deformability. Its application was determined by the field inspections, and could be carried out at any foundation area and until the placing of the first concrete layer. The mandatory treatment was restricted to the region below the blockheads, the holes deepening to 3 m into the rock, in a 6 m x 6 m square mesh.

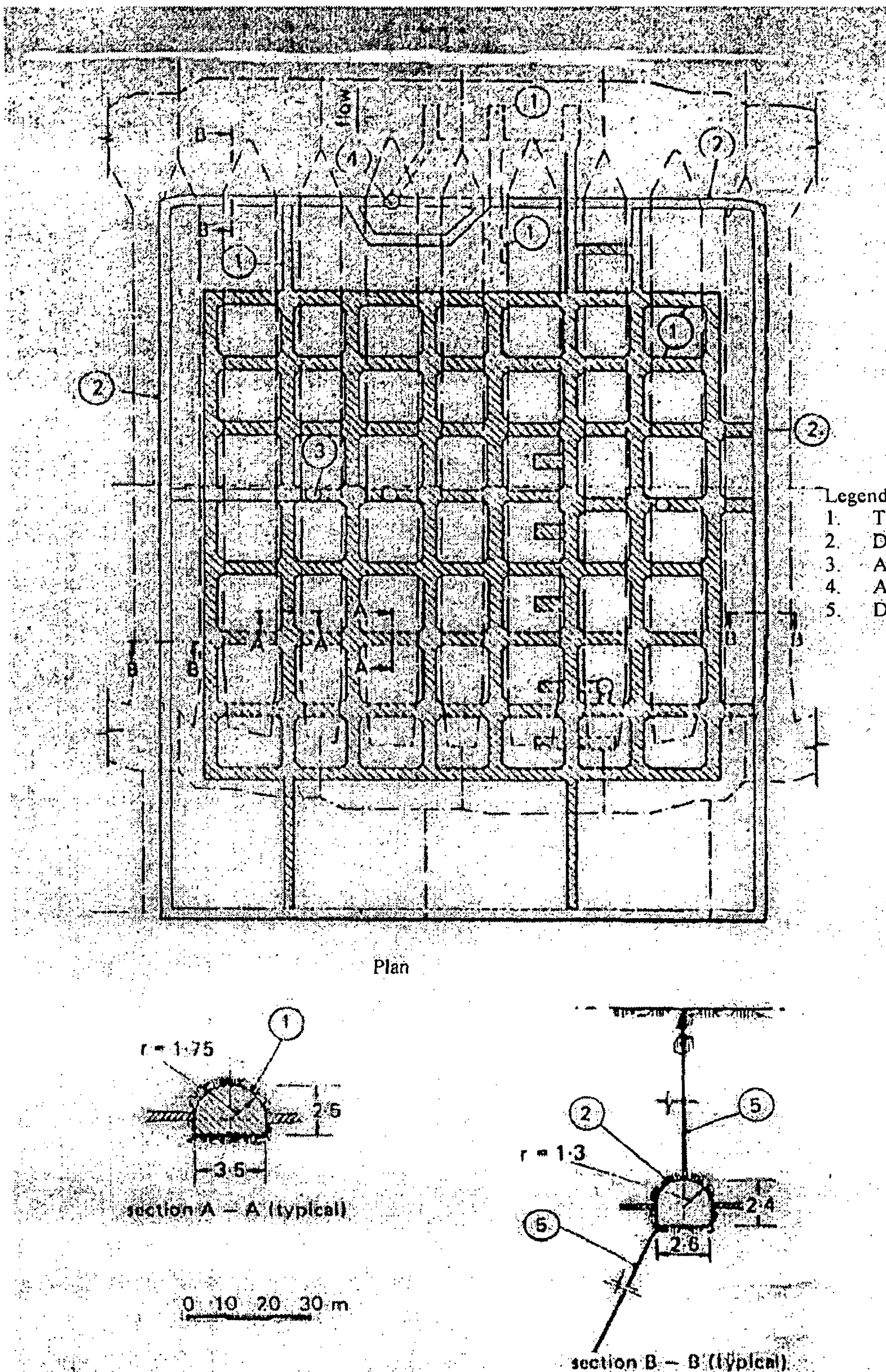
Upstream curtain was up to 120 m deep and basically composed of two lines with 3 m spaced holes. A third line (inside one), with holes provided for 1.5 m intervals, would be executed only a stretches where grout-take persistence existed above the limits previously established. In the main dam, the left transition stretch and the right wing dam, the curtain was located upstream the blocks, in the open air. However, at the area comprising the diversion structure, spillway and powerhouse, the work was executed from 3.8 m to 2.5 m galleries, which allowed the utilization and ease handling of large equipment.

Water-cement ratio of the grouting material is W : C = 1 : 1 (by weight) with 2% of bentonite regarding the cement weight. This grouting treatment resulted the great reductions of 10 times to 100 times of permeability of the foundation rocks.

f. Drainage system

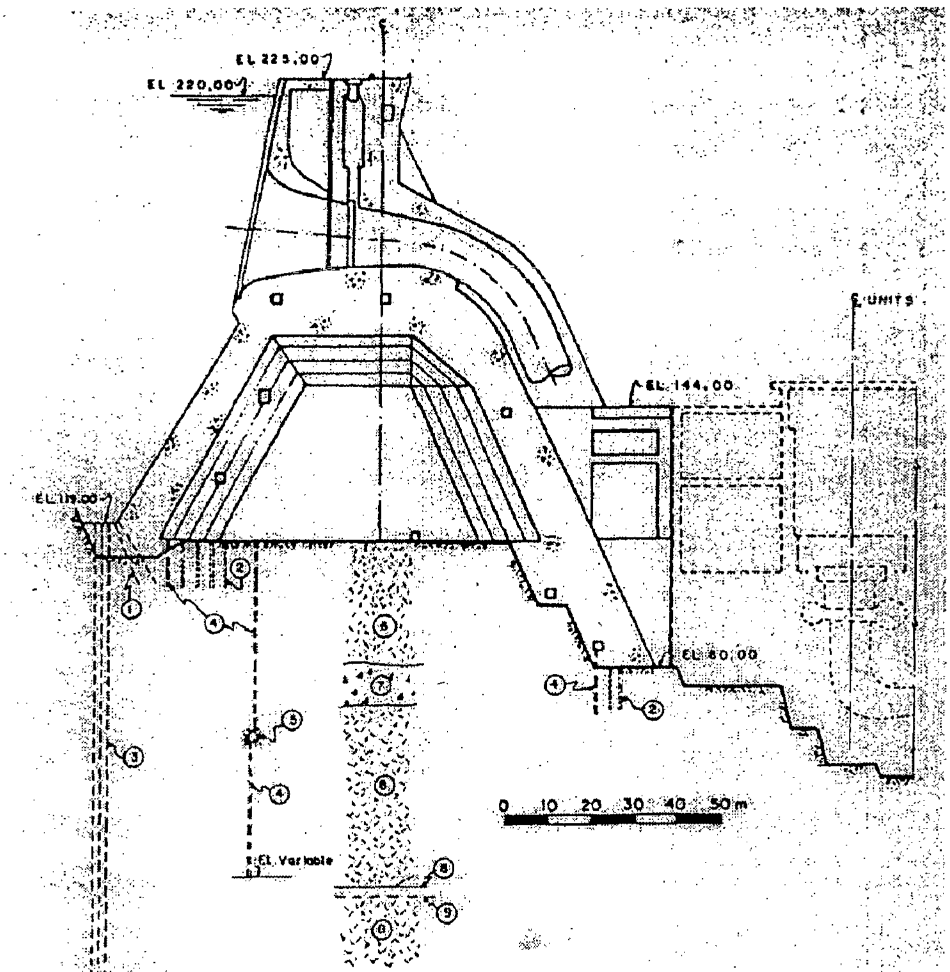
This system is of particular importance in the case of Itaipu, because of the sliding stability of the dam along the sub horizontal discontinuities in the foundation. Between the stems, next to the upstream head of the blocks, a line of fan-shaped vertical drains 4 ft in diameter spaced approximately 2 m apart connects the foundation surface with the drainage tunnels. This line of drains is extended from the drainage tunnels to the lower levels of the rock foundation according to the depth of the grout curtains.

In the river channel area, besides the upstream drainage tunnel at el. 20.00 m, a downstream tunnel under the powerhouse and two transverse lateral tunnels were excavated isolating an area of approximately 145 m x 185 m. These lateral tunnels are also connected to the surface through Ø 4 ft drainage holes spaced 6 m apart and bored from the tunnels about 20 m deep.



Legend:
1. Tunnel plugged with concrete
2. Drainage tunnel
3. Access shaft
4. Access shaft
5. Drilled drain

Figure 3.3.6: Itaipu Dam.
Grid of Shear Keys



Legend:
1. Contact grouting
2. Consolidation grouting, H = 3 m, S = 6 m
3. Curtain grouting, H = 120 m, S = 1.50 m
4. Drainage curtain, H = 20 m, S = 6 m
5. Basalt
6. Breccia
7. Contact A/B
8. Discontinuity A

Figure 3.3.7: Itaipu Dam.
Impermeable Treatment

3.4. Summary of The Review, and Study & The Inferences

3.4.1. Summary of the review

The review of foundation treatments of existing dams is summarized in Table 3.4.2, whereas Table 3.4.3, 3.4.4 and 3.4.5 give the summary of details of curtain grouting, consolidation grouting and drainage curtain respectively.

3.4.2. Study and the inferences

a. *Foundation weakness and the related problem*

From review, the foundation weakness could be classified as weakness zones, smooth surface rock and Karstic phenomena. The following is the study and the inferences of these weaknesses.

i. Weakness zones. It could be the occurrence of fractures, joints, fissures, weathered rocks, shear zones, dykes, broken rocks and soft rocks or other soft materials such like silt, clay or seam layers which occur in most of the foundation of the dams under review. The problems related to these weaknesses are uncontrolled seepage, potential sliding, different settlement and concentration of compressive stress. It can be inferred that these weakness zones take place at most types of rock.

ii. Smooth surface rock. It took place at the foundation of Torrejon and Keban dams where the type of the foundation rock is Schist. The problem related to this weakness is sliding and uncontrolled seepage. It can be inferred that in case foundation is Schist rock, particular treatment should be carried out to prevent sliding and uncontrolled seepage.

iii. Karstic phenomena. It occurred at Keban dam in Turkey, El Cajon dam in Honduras and Wujiangdu dam in China. Karstic phenomena is the phenomena of the occurrence of cavities or caverns or holes or tubes or openings, which are with or without soft soil filling. Another study on San Giuliano dam shows that karst phenomena occurred also at limestone rock. It can be inferred that karstic phenomena could be suspected where the foundation consists of limestone rock.

b. Foundation treatment

The foundation treatments carried out on the dams under review could be classified with respect to the related foundation problem as treatments against potential sliding and potential high or different settlements (treatment against weak zones) and seepage control (impermeabilization). The followings are the inferences of these treatments.

i. Treatment against weak zones. It could be made by (i) construction of shear key or shear plug or strut monolith with the dam structure penetrating the weak layer, or backfilling concrete. It is carried out in case the weak zone appears in form of relatively horizontal soft rock layers or in form of local pockets of soft or broken rocks at a shallow depth below or around the dam foundation (Nagarjunasagar dam, Keban dam, El Cajon dam, Liujiaxia dam, Bhakra dam, Longyangxia dam, Nagawado dam, Supa dam, Tanchiangkou dam, Uruga - i dam, Dudhganga dam, Ohkawase dam and Ranapratap Sagar dam), (ii) construction of a grid of shear keys by trenching on shear zones. It is carried out in case the weak zone appears in form of relatively horizontal shear zone layer located deep below the dam foundation level (Sardar Sarovar dam, Srisailam dam, Glen Canyon dam, Itaipu dam, Clyde dam, Kadana dam and Torrejon dam), (iii) construction of dental structure. It is carried out in case the weak zone appears in form of relatively vertical deep soft layer e.g. fault, shear zone and dyke (Bhakra dam, Clyde dam, Bansagar dam, Canyon Ferry dam and Kadana dam), (iv) construction of buttressing structure at the toe of the dam structure. It is carried out in case of the weak zone below the dam extends to downstream of the dam (Srisailam dam and Dowrshak dam), and (v) strengthening or roughing the surface of the rock foundation by anchoring (Dudhganga dam), prestressing and anchoring (Ranapratap Sagar dam), saw-tooth shaping (Barret chute dam).

ii. Seepage control. Table 3.4.2 indicates that from the dams under review, most of them are provided seepage control system. The system consists of : (i) curtain grouting or concrete diaphragm wall, (ii) consolidation grouting, and (iii) drainage system. Little number of literatures does not discuss these treatments completely.

iii. Curtain grouting or concrete diaphragm wall. There are 2 dams which are provided concrete diaphragm wall i.e. Keban dam and Wujiangdu dam which the foundation suffering from highly permeable and soft rocks, where in this case, the performance of curtain grouting is doubtful. At both dams, the foundation rock is limestone rock having karstic phenomena. It can be inferred that when the permeability of foundation is so high that providing a curtain grouting is doubtful, concrete diaphragm wall could be considered. There are 6 dams, that multi rows curtain grouting are provided, out of 9 dams, that part of the foundation rock is of limestone rock and the row number of the grout curtain is discussed in the literature (see Table 3.4.3). Out of these 6 dams, single grout curtain is provided. It can be inferred that when the type of foundation rock is limestone, providing multi rows curtain grouting is preferred. In case of the foundation rock is Schist (Keban dam), curtain grouting should be carried out to a depth at least until penetrates the rock Study on curtain grouting presented in Table 3.4.6 indicates that most of the dams under review having maximum depth of curtain grouting (primary hole) of greater than one third of the dam height and maximum equal to the dam height, this range covers most types of foundation rock. It can be inferred that maximum depth of curtain grouting (Hc) of $H/3 < Hc \leq H$ valid for the dams with heights equals to and grater than 41.80 m and valid for most types of foundation rock. Table 3.4.7 indicates that most of the dams under review have the minimum space of curtain grouting holes of greater than 1 m up to equal to 2 m ($1 \text{ m} < S \leq 2$ m) which valid for most dam heights and most types of foundation rock.

iv. Consolidation grouting. Study on consolidation grouting presented in Table 3.4.8 indicates that depths of consolidation grouting holes of greater than 5 m up to equal to 20 m ($5 \text{ m} < D \leq 20$ m) valid for most dam heights and all types of foundation rock. Table 3.4.9 indicates that the space of consolidation grouting holes of $1 \text{ m} < S \leq 3$ m valid most dam heights and for all types of foundation rock.

v. Drainage system. Table 3.4.2 indicates that most of the foundation drainage system is in form of drainage curtain. Table 3.4.10 indicates that if high of the dam equals to and greater than 76 m, number of drainage gallery is more than 1.

Table 3.4.11 indicates that for the dam height equal to and greater than 135 m, the depth of drainage holes is greater than 20 % up to equal to 40 % of the maximum depth of grouting curtain holes. For the dam height less than 135 m, the depth of drainage holes is greater than 40 % up to equal to 60 %.

Table 3.4.1: List of Dams Reviewed

No.	Name of Dam	Dam Type	Dam Height (m)	Country	Literature
1	Sardar Sarovar dam	Concrete gravity dam	163.00	India	CBIP No. 266/Jan.'98 & April'87
2	Nagarjunasagar dam	Masonry gravity dam	124.66	India	CBIP No.138/Oct.'79
3	Keban dam	Composite dam	209.20	Turkey	ICOLD,Q.37R.43/Montr.'70 & Q.45R.27,Mex.'76
4	El Cajon dam	Concrete arch dam	238.00	Honduras	ICOLD,Q.58R.58/Laus.'85,WP&DC/Febr.'81
5	Liujiaxia dam	Concrete gravity dam	147.00	China	ICOLD,Q.45R.67/Mex.'76
6	El Atazar dam	Concrete arch dam	133.00	Spain	ICOLD,Q.37R.59/Mont.'70
7	Wujiangdu dam	Concrete gravity dam	165.00	China	Large dams in China/'87
8	Srisailam dam	Masonry -conc. gravity dam	144.00	India	CBIP No. 266/Jan.'98
9	Ravasanella dam	Concrete block-gravity dam	97.00	Italy	ICOLD,Q.66R.21/Vienne.'91
10	San Giuliano dam	Concrete gravity dam	79.00	Italy	ICOLD,Q.66R.45/Vienne.'91
11	Bhakra dam	Concrete gravity dam	225.60	India	CBIP No.138/Oct.'79
12	Dowrshak dam	Concrete gravity dam	218.50	USA	CBIP No. 266/Jan.'98
13	Glen Canyon dam	Concrete arch dam	216.40	USA	Glen C dam&res/USBR'70
14	Kolbrein dam	Concrete arch dam	200.00	Austria	ICOLD,Q.58R.81/Laus.'85
15	Itaipu dam	Composite dam	196.00	Brazil	ICOLD,Q.58R.8/Laus.'85, ICOLD,Q.53,R.10/Rio.'82, Q.53R.48/Rio'82 & WP&DC/May'82
16	Longyangxia dam	Arch gravity dam	178.00	China	ICOLD,Q.66R.90/Vienne.'91
17	Nagawado dam	Concrete arch dam	155.00	Japan	ICOLD,Q.37R.9/Montr.'70
18	Alcantara dam	Concrete buttress dam	135.00	Spain	Found. for dams/ASCE,'74
19	Pine Flat dam	Concrete gravity dam	130.80	USA	ICOLD,Q.45R.5/Mex.'76
20	Detroit dam	Concrete gravity dam	125.90	USA	ICOLD,Q.45R.5/Mex.'76
21	Cortes dam	Concr.arch gravity dam	116.00	Spain	ICOLD,Q.45R.5/Mex.'76
22	Fengtan dam	Hollow gravity arch dam	112.50	China	Large dams in China/'87
23	Libby dam	Concrete gravity dam	110.30	USA	ICOLD,Q.45R.5/Mex.'76
24	Clyde dam	Concrete gravity dam	102.00	New Z'land	ICOLD,Q.66R.10/Vienne.'91
25	Supa dam	Concrete gravity dam	101.00	India	CBIP No.266/Jan.'98
26	Green Peter dam	Concrete gravity dam	99.70	USA	CBIP No. 215/March '90
27	Tanchiangkou	Composite dam	97.00	China	ICOLD,Q.48R.59/Del.'79
28	Badush dam	Concr. hollow-buttress dam	90.00	Iraq	ICOLD,Q.66R.86/Vienne.'91
29	Bemposta dam	Concr.arch hollow grav. dam	87.00	Portugal	ICOLD,Q.58R.46/Laus.'85
30	Bull shoals	Concrete gravity dam	86.60	USA	ICOLD,Q.45R.5/Mex.'76
31	East canyon dam	Concrete arch dam	79.19	USA	East C dam/USBR
32	Table rock dam	Concrete gravity dam	76.80	USA	ICOLD,Q.45R.5/Mex.'76
33	Mt. Morris dam	Concrete gravity dam	76.20	USA	ICOLD,Q.45R.5/Mex.'76
34	Urugua-I RCC dam	Concrete gravity dam	76.00	Argentina	WP & DC/Dec.'89
35	Dudhganga dam	Masonry gravity dam	75.00	India	ICOLD,Q.53R.40/Rio.'82,ICOLD,Q.66R.52/Vienne'91
36	Hitokura dam	Concrete gravity dam	75.00	Japan	ICOLD,Q.58R.25/Laus.'85
37	Gezende dam	Concrete arch dam	75.00	Turkey	ICOLD,Q.66R.43/Vienne.'91
38	Greers Ferry dam	Concrete gravity dam	74.10	USA	ICOLD,Q.45R.5/Mex.'76
39	Chief Joseph dam	Concrete gravity dam	70.10	USA	ICOLD,Q.45R.5/Mex.'76
40	Bansagar dam	Masonry gravity dam	63.00	India	Workshop on .../Jul.'92
41	Canyon Ferry dam	Concrete gravity dam	68.50	USA	U S D I, '57
42	Kadana dam	Masonry -concr. grav.. dam	66.00	India	CBIP No. 266/Jan.'98
43	Stewartville dam	Concrete gravity dam	62.50	Canada	ICOLD,Q.65R.53/Vienne.'91
44	Clark hill dam	Concrete gravity dam	61.00	USA	ICOLD,Q.45R.5/Mex.'76
45	Ohkawase dam	Concrete gravity dam	50.40	Japan	ICOLD,Q.66R.27/Vienne.'91
46	Mountain chute dam	Concrete gravity dam	50.00	Canada	ICOLD,Q.65R.53/Vienne.'91
47	Ranapratap Sagar dam	Masonry gravity dam	42.67	India	CBIP No.138/June '83
48	Conemaugh dam	Concrete gravity dam	41.80	USA	ICOLD,Q.45R.5/Mex.'76
49	Torrejon dam	Concrete gravity dam	30.00	Spain	ICOLD,Q.66R.20/Vienne.'91
50	Barrett Chute dam	Concrete gravity dam	27.00	Canada	ICOLD,Q.65R.53/Vienne.'91

No.	Name of Dam	Dam Type	Dam Height (m)	Country	Foundat	Type of Foundation Treatment									
						Particular treatment in Foundation Shaping	Treatment of Fault and Weak Zones				Foundation Grouting		Foundation Drainage		Special Treatment
							Shear key/plug/strut	Grid of shear keys	Dental treatment	Buttressing structure	Curtain grouting	Consolidation grouting	Drainage curtain	Other type	
1	Sardar Sarovar dam	Concrete gravity dam	163.00	India	Basalt, argil sandstones,q limestone roc	-	-	✓	-	-	✓	✓	✓	-	-
2	Nagarjunasagar dam	Masonry gravity dam	124.66	India	Granite-gneiss	-	✓	-	-	-	✓	✓	✓	-	-
3	Keban dam	Composite dam	209.20	Turkey	Karstified met & schist rocks	-	✓	-	-	-	Curtain grouting & concrete diaphragm wall	✓	-	-	-
4	El Cajon dam	Concrete arch dam	238.00	Honduras	Karstified atima	-	✓	-	-	-	✓	-	-	-	-
5	Liujiaxia dam	Concrete gravity dam	147.00	China	Micaceous qua	-	✓	-	-	-	✓	✓	✓	-	-
6	El Atazar dam	Concrete arch dam	133.00	Spain	Quartzite & sla	Shotcrete & rockbolting	✓	-	-	-	✓	✓	✓	-	-
7	Wujiangdu dam	Concrete gravity dam	165.00	China	Karstified triasic li	-	-	-	-	-	Curtain grouting & concrete diaphragm wall	-	-	-	-
8	Srisailam dam	Mascury -conc. gravity dam	144.00	India	Quarzites & sha	-	-	✓	-	✓	✓	✓	✓	-	-
9	Ravasanella dam	Concrete block-gravity dam	97.00	Italy	Quartz porphyry r	-	-	-	-	-	-	-	-	-	Construction of concrete block-gravity dam
10	San Giuliano dam	Concrete gravity dam	79.00	Italy	Limestone rock &	-	-	-	-	-	✓	✓	-	-	Construction of sub-foundation
11	Bhakra dam	Concrete gravity dam	225.60	India	Sandstone, claybed conglomerate rocks	-	✓	-	✓	-	✓	✓	✓	-	Construction of deep strut
12	Dowrshak dam	Concrete gravity dam	218.50	USA	Granodiorite rock	-	-	-	-	✓	✓	-	✓	-	-
13	Glen Canyon dam	Concrete arch dam	216.40	USA	Sandstone, shale ro	Rockbolting	-	✓	-	-	✓	✓	✓	-	-
14	Kolbrein dam	Concrete arch dam	200.00	Austria	Granite gneiss rock	-	-	-	-	-	✓	✓		-	-
15	Itaipu dam	Composite dam	196.00	Brazil	Basalt & breccia rock	Rockbolting	-	✓	-	-	✓	✓	✓	-	Constr. Of grid of shear keys
16	Longyangxia dam	Arch gravity dam	178.00	China	Sound granodiorite ro	Abutment anchoring	✓	-	-	-	✓	✓	✓	-	-
17	Nagawado dam	Concrete arch dam	155.00	Japan	Biotite granite rock	-	✓	-	-	-	-	✓	-	-	-
18	Alcantara dam	Concrete buttress dam	135.00	Spain	Slaty schist rock	-	-	-	-	-	✓	✓	✓	-	-
19	Pine Flat dam	Concrete gravity dam	130.80	USA	Amphibolite rock	-	-	-	-	-	✓	-	✓	-	-
20	Detroit dam	Concrete gravity dam	125.90	USA	Andesite rock	-	-	-	-	-	✓	-	✓	-	-
21	Cortes dam	Concr. arch gravity dam	116.00	Spain	Granite gneiss rock	-	-	-	-	-	✓	✓	✓	-	-
22	Fengtan dam	Hollow gravity arch dam	112.50	China	Sandstone, slate rocks	-	-	-	-	-	✓	-	-	-	-
23	Libby dam	Concrete gravity dam	110.30	USA	Argillite rock	-	-	-	-	-	✓	-	✓	-	-
24	Clyde dam	Concrete gravity dam	102.00	New Zealand	Quartz, albite, muscovite micas rocks	-	-	✓	✓	-	✓	-	✓	-	-
25	Supa dam	Concrete gravity dam	101.00	India	Calcareous magnetite quartzi	-	✓	-	-	-	-	-	-	-	-

Table 3.4.2: Summary of Foundation Treatment

No.	Name of Dam	Dam Type	Dam Height (m)	Country	Foundation Material Type	Particular Foundation Weakness	Type of Foundation Treatment: Particular Treatment in Foundation Shaping	Treatment of Fault and Weak Zones: Shear key/plug/strut	Treatment of Fault and Weak Zones: Grid of shear keys	Treatment of Fault and Weak Zones: Dental treatment	Treatment of Fault and Weak Zones: Buttressing structure	Foundation Grouting: Curtain grouting	Foundation Grouting: Consolidation grouting	Foundation Drainage: Drainage curtain	Foundation Drainage: Other type	Special Treatment
1	Sardar Sarovar dam	Concrete gravity dam	163.00	India	Basalt, argillaceous sandstones,quartzitic sandstone & limestone rocks	Fault, fractures & dyke	-	-	✓	-	-	✓	✓	✓	-	-
2	Nagarjunasagar dam	Masonry gravity dam	124.66	India	Granite-gneiss & quartzite rocks	Weak seam, fault & fracture zones	-	✓	-	-	-	✓	✓	✓	-	-
3	Keban dam	Composite dam	209.20	Turkey	Karstified metamorphic limestone & schist rocks	Karstic phenomena, weak & fault zones	-	✓	-	-	-	Curtain grouting & concrete diaphragm wall	✓	-	-	-
4	El Cajon dam	Concrete arch dam	238.00	Honduras	Karstified alma limestone rock	Fault zone & karstic phenomena	-	✓	-	-	-	✓	-	-	-	-
5	Liujiaxia dam	Concrete gravity dam	147.00	China	Micaceous quartz schist rock	Weathered, joints & fault zones	-	✓	-	-	-	✓	✓	✓	-	-
6	El Atazar dam	Concrete arch dam	133.00	Spain	Quartzite & slate rocks	Broken rocks & fracture zones	Shotcrete & rockbolting	✓	-	-	-	✓	✓	✓	-	-
7	Wujiangdu dam	Concrete gravity dam	165.00	China	Karstified triasic limestone rock	Occurrence of Karstic cavities & fault zone	-	-	-	-	-	Curtain grouting & concrete diaphragm wall	-	-	-	-
8	Srisailam dam	Masonry-conc. gravity dam	144.00	India	Quarzites & shales rocks	Occurrence of soft & broken rocks	-	-	✓	-	✓	✓	✓	✓	-	-
9	Ravasanella dam	Concrete block-gravity dam	97.00	Italy	Quartz porphyry rock	Very strong fragmentation on right abutment, known during excavation	-	-	-	-	-	-	-	-	-	Construction of concrete block-gravity dam
10	San Giuliano dam	Concrete gravity dam	79.00	Italy	Limestone rock & clay soil	Existence of huge cavity filled with clay, known during excavation	-	-	-	-	-	✓	✓	-	-	Construction of sub-foundation
11	Bhakra dam	Concrete gravity dam	225.60	India	Sandstone, claybeds & conglomerate rocks	Silt & shear zones	-	✓	-	✓	-	✓	✓	✓	-	Construction of deep strut
12	Dowrshak dam	Concrete gravity dam	218.50	USA	Granodiorite rock	Joints & shear zones	-	-	-	-	✓	✓	-	✓	-	-
13	Glen Canyon dam	Concrete arch dam	216.40	USA	Sandstone, shale rocks	Seam layer & joints	Rockbolting	-	✓	-	-	✓	✓	✓	-	-
14	Kolbrein dam	Concrete arch dam	200.00	Austria	Granite gneiss rock	Joints	-	-	-	-	-	✓	✓		-	-
15	Itaipu dam	Composite dam	196.00	Brazil	Basalt & breccia rocks	Fracture & shear zones	Rockbolting	-	✓	-	-	✓	✓	✓	-	Constr. Of grid of shear keys
16	Longyangxia dam	Arch gravity dam	178.00	China	Sound granodiorite rock	Fault & shear zones	Abutment anchoring	✓	-	-	-	✓	✓	✓	-	-
17	Nagawado dam	Concrete arch dam	155.00	Japan	Biotite granite rock	Fault zone	-	✓	-	-	-	-	✓	-	-	-
18	Alcantara dam	Concrete buttress dam	135.00	Spain	Slaty schist rock	Joints & fracture zones	-	-	-	-	-	✓	✓	✓	-	-
19	Pine Flat dam	Concrete gravity dam	130.80	USA	Amphibolite rock	Joints	-	-	-	-	-	✓	-	✓	-	-
20	Detroit dam	Concrete gravity dam	125.90	USA	Andesite rock	Fracture zone	-	-	-	-	-	✓	-	✓	-	-
21	Cortes dam	Concr.arch gravity dam	116.00	Spain	Granite gneiss rock	Fault & fracture zones	-	-	-	-	-	✓	✓	✓	-	-
22	Fengtan dam	Hollow gravity arch dam	112.50	China	Sandstone, slate rocks	Fault & clay zones	-	-	-	-	-	✓	-	-	-	-
23	Libby dam	Concrete gravity dam	110.30	USA	Argillite rock	Joints	-	-	-	-	-	✓	-	✓	-	-
24	Clyde dam	Concrete gravity dam	102.00	New Zealand	Quartz, albite, muscovite & chlorite micas rocks	Weak seam (shear) & fault zones	-	-	✓	✓	-	✓	-	✓	-	-
25	Supa dam	Concrete gravity dam	101.00	India	Calcareous magnetite quartzite rock	Weathered rock, shear zones & dykes	-	✓	-	-	-	-	-	-	-	-

Table 3.4.2: Summary of Foundation Treatment

No.	Name of Dam	Dam Type	Dam Height (m)	Country	Foundation Material Type	Particular Foundation Weakness	Type of Foundation Treatment									
							Particular Treatment in Foundation Shaping	Treatment of Fault and Weak Zones				Foundation Grouting		Foundation Drainage		Special Treatment
								Shear key/plug/ strut	Grid of shear keys	Dental treatment	Buttressing structure	Curtain grouting	Consolidation grouting	Drainage curtain	Other type	
26	Green Peter dam	Concrete gravity dam	99.70	USA	Basalt rock	Layered rock	.	.	.	.	.	✓	.	✓	.	.
27	Tanchiangkou	Composite dam	97.00	China	Metamorphic igneous rock	Fault & Fracture zones	.	✓	.	.	.	✓	.	.	.	.
28	Badush dam	Concr. hollow-buttress dam	90.00	Iraq	Marls, sandstone, gypsum & limestone rocks	Highly permeable	.	.	.	.	.	✓	✓	.	.	.
29	Bemposta dam	Concr.arch hollow grav. dam	87.00	Portugal	Crystalline schist rock	Rock fissures	.	.	.	.	.	✓	.	✓	.	.
30	Bull shoals	Concrete gravity dam	86.60	USA	Dolomite rock	.	.	.	.	.	.	✓	.	✓	.	.
31	East canyon dam	Concrete arch dam	79.19	USA	Quartzite & limestone rocks	Joints	.	.	.	.	.		✓	✓	.	.
32	Table rock dam	Concrete gravity dam	76.80	USA	Dolomite & sandstone rocks	.	.	.	.	.	.	✓	.	✓	.	.
33	Mt. Morris dam	Concrete gravity dam	76.20	USA	Shale rock	Joints	.	.	.	.	.	✓	.	✓	.	.
34	Urugua-I RCC dam	Concrete gravity dam	76.00	Argentina	Basalt rock & lateritic soil	Joints & weathered rock seam	.	✓	.	.	.	✓	✓	✓	.	.
35	Dudhganga dam	Masonry gravity dam	75.00	India	Ortho-quartzites & shale rocks	Low shear parameters due to existence of shale band	D/s rock anchoring	✓	.	.	.	✓	.	✓	.	.
36	Hitokura dam	Concrete gravity dam	75.00	Japan	Slate & sandstone rocks	Fracture, foldings & fracture zones	.	.	.	.	.	✓	✓	✓	.	.
37	Gezende dam	Concrete arch dam	75.00	Turkey	Cretaceous limestone rock	Fracture, foldings & unstable abutment	Facing concrete	.	.	.	.	✓	.	.	.	.
38	Greers Ferry dam	Concrete gravity dam	74.10	USA	Sandstone & shale rocks	.	.	.	.	.	.	✓	.	✓	.	.
39	Chief Joseph dam	Concrete gravity dam	70.10	USA	Granodiorite rock	Fault zone	.	.	.	.	.	✓	.	✓	.	.
40	Bansagar dam	Masonry gravity dam	63.00	India	Qurtzo-feldspathic gneiss rock	Fault, fracture zones & master joint	.	.	.	✓	.	✓	.	.	.	.
41	Canyon Ferry dam	Concrete gravity dam	68.50	USA	Shale rock	Fault zone	.	.	.	✓	.	✓	✓	✓	.	.
42	Kadana dam	Masonry -concr. grav. dam	66.00	India	Quartzites, quartz-mica schist & phyllites rocks	Joints, shear zones, minor faults & small displacement	.	.	✓	✓	.	✓	✓	.	Drainage holes along faults	.
43	Stewartville dam	Concrete gravity dam	62.50	Canada	Crystalline limestone rock	Erodible micaceous material	.	.	.	.	.	✓	.	.	Box drain at concrete-rock contact	.
44	Clark hill dam	Concrete gravity dam	61.00	USA	Biotite gneiss rock	.	.	.	.	.	.	✓	.	✓	.	.
45	Ohkawase dam	Concrete gravity dam	50.40	Japan	Rhyolite rock	Shear zone of horizontal thin weak clay layer	.	✓	.	.	.	✓	✓	.	.	.
46	Mountain chute dam	Concrete gravity dam	50.00	Canada	Crystalline limestone rock	Zones of weakness which is like soil	.	.	.	.	.	✓	.	.	Pressure relief well	.
47	Ranapratap Sagar dam	Masonry gravity dam	42.67	India	Sandstone & shale rocks	Shear zones, weathered rocks & slumped shales	Prestressing & anchoring dam to rock	✓	.	.	.	✓	.	.	.	.
48	Conemaugh dam	Concrete gravity dam	41.80	USA	Sandstone rock	.	.	.	.	.	.	✓	.	✓	.	.
49	Torrejon dam	Concrete gravity dam	30.00	Spain	Schist, siliceous quartzites & diorite rocks	Possibility of sliding along the smooth surface of schist	.	.	✓	.	.	✓	.	✓	.	.
50	Barrett Chute dam	Concrete gravity dam	27.00	Canada	Biotite gneiss, limestone & granitic gneiss rocks	Shear zones & weak zones in the different interlayered bedrock	Surface roughening	.	.	.	.	✓	.	.	Box drain at concrete-rock contact	.

Note :

.	means : details not available

Table 3.4.3: Summary of Curtain Grouting

No.	Name of Dam	Dam Type	Dam Height (m)	Country	Foundation Material Type	Curtain Grouting							
						Max. depth applied(m)	Range of max. depth of USBR		Min. space of holes (m)	Diameter of holes (mm)	Single or multi row	Grouting criteria	Max. grouting press. (kg/cm^2)
							Max. range (m)	Min. range (m)					
1	Sardar Sarovar dam	Concrete gravity dam	163.00	India	Basalt, argillaceous sandstones, quartzitic sandstone & limestone rocks	40.75	79.33	62.33	3.00	35.00	Single	-	-
2	Nagarjunasagar dam	Masonry gravity dam	124.66	India	Granite-gneiss & quartzite rocks	45.72	66.55	49.55	3.00	76.20	Single	-	21.09
3	Keban dam	Composite dam	209.20	Turkey	Karstified metamorphic limestone & schist rocks	Penetrate sound rock	94.73	77.73	3.00	-	Multi (2)	-	-
4	El Cajon dam	Concrete arch dam	238.00	Honduras	Karstified atima limestone rock	180.00	104.33	87.33	0.625	-	Multi (3)	<2 Lu &<200 kg/m	50.00
5	Liujiaxia dam	Concrete gravity dam	147.00	China	Micaceous quartz schist rock	120.00	74.00	57.00	2.00	-	Single	-	-
6	El Atazar dam	Concrete arch dam	133.00	Spain	Quartzite & slate rocks	45.00	69.33	52.33	4.00	-	Single	-	20.00
7	Wujiangdu dam	Concrete gravity dam	165.00	China	Karstified triasic limestone rock	200.00	80.00	63.00	2.00	-	Multi (3)	-	60.00
8	Srisailam dam	Masonry -concr. Gr.dam	144.00	India	Quarzites & shales rocks	***61.00***	73.00	56.00	-	-	Single	-	-
9	Bhakra dam	Concrete gravity dam	225.60	India	Sandstone, claybeds & conglomerate rocks	***85.34***	100.20	83.20	1.50	-	Single	-	35.10
10	Dowrshak dam	Concrete gravity dam	218.50	USA	Granodiorite rock	76.00	97.83	80.83	1.50	-	-	-	-
11	Glen Canyon dam	Concrete arch dam	216.40	USA	Sandstone, shale rocks	76.20	97.13	80.13	3.00	-	Single	-	-
12	Kolbrein dam	Concrete arch dam	200.00	Austria	Granite gneiss rock	23.00	91.67	74.67	1.30	120.00	Single	-	-
13	Itaipu dam	Composite dam	196.00	Brazil	Basalt & breccia rocks	120.00	90.33	73.33	1.50	-	Multi (2)	<12 kg/m	81.00
14	Alcantara dam	Concrete buttress dam	135.00	Spain	Slaty schist rock	100.00	70.00	53.00	3.00	-	Single	-	-
15	Pine Flat dam	Concrete gravity dam	130.80	USA	Amphibolite rock	37.00	68.60	51.60	1.50	-	-	-	-
16	Detroit dam	Concrete gravity dam	125.90	USA	Andesite rock	76.00	66.97	49.97	1.50	-	-	-	-
17	Cortes dam	Concr.arch gravity dam	116.00	Spain	Granite gneiss rock	31.00	63.67	46.67	1.50	25.40	Single	-	-
18	Fengtan dam	Hollow gravity arch dam	112.50	China	Sandstone, slate rocks	96.50	62.50	45.50	1.50	-	Single	<21.3 kg/m	-
19	Libby dam	Concrete gravity dam	110.30	USA	Argillite rock	***49.00***	61.77	44.77	3.00	-	-	-	-
20	Green Peter dam	Concrete gravity dam	99.70	USA	Basalt rock	61.00	58.23	41.23	1.50	-	-	-	-

Table 3.4.3: Summary of Curtain Grouting

No.	Name of Dam	Dam Type	Dam Height (m)	Country	Foundation Material Type	Curtain Grouting							
						Max. depth applied(m)	Range of max.depth of USBR		Min. space of holes (m)	Dia. of holes (mm)	Single or multi row	Grouting criteria	Max. grouting press. (kg/cm^2)
							Max.range (m)	Min.range (m)					
21	Badush dam	Composite dam	90.00	Iraq	Marls, sandstone, gypsum & limestone rocks	*50.00*	55.00	38.00	1.50	-	Multi (3)	-	-
22	Bemposta dam	Concr.arch hollow grav. dam	87.00	Portugal	Crystalline schist rock	30.00	54.00	37.00	-	-	Single	-	-
23	Bull shoals	Concrete gravity dam	86.60	USA	Dolomite rock	30.00	53.87	36.87	1.80	-	-	-	-
24	East Canyon dam	Concrete arch dam	79.19	USA	Quartzite & limestone rocks	*45.72*	51.40	34.40	3.00	-	-	-	-
25	Table rock dam	Concrete gravity dam	76.80	USA	Dolomite & sandstone rocks	*43.00*	50.60	33.60	0.90	-	-	-	-
26	Mt. Morris dam	Concrete gravity dam	76.20	USA	Shale rock	15.00	50.40	33.40	1.50	-	-	-	-
27	Urugua-I RCC dam	Concrete gravity dam	76.00	Argentina	Basalt rock & lateritic soil	-			3.00	-	Single	-	-
28	Hitokura dam	Concrete gravity dam	75.00	Japan	Slate & sandstone rocks	99.00	50.00	33.00	-	-	Single	2 Lu	30.00
29	Gezende dam	Concrete arch dam	75.00	Turkey	Cretaceous limestone rock	*40.00*	50.00	33.00	2.00	-	Multi (2)	-	-
30	Greers Ferry dam	Concrete gravity dam	74.10	USA	Sandstone & shale rocks	*37.00*	49.70	32.70	0.90	-	-	-	-
31	Chief Joseph dam	Concrete gravity dam	70.10	USA	Granodiorite rock	23.00	48.37	31.37	1.50	-	-	-	-
32	Bansagar dam	Masonry gravity dam	63.00	India	Qurtzo-feldspathic gneiss rock	15.00	46.00	29.00	3.00	-	Single	-	-
33	Canyon Ferry dam	Concrete gravity dam	68.50	USA	Shale rock	30.50	47.83	30.83	12.70	63.50	Single	-	44.00
34	Kadana dam	Masonry -concr. grav.dam	66.00	India	Quarzites, quartz-mica schist & phyliites rocks	*36.60*	47.00	30.00	1.50	-	Single	1 Lu	30.00
35	Stewartville dam	Concrete gravity dam	62.50	Canada	Crystalline limestone rock	*30.00*	45.83	28.83	-	-	Single	-	-
36	Clark hill dam	Concrete gravity dam	61.00	USA	Biotite gneiss rock	53.00	45.33	28.33	2.40	-	-	-	-
37	Mountain chute dam	Concrete gravity dam	50.00	Canada	Crystalline limestone rock	115.00	41.67	24.67	3.00	75.00	Single	-	-
38	Conemaugh dam	Concrete gravity dam	41.80	USA	Sandstone rock	*37.00*	38.93	21.93	1.50	-	-	-	-
39	Barrett Chute dam	Concrete gravity dam	27.00	Canada	Biotite gneiss, limestone & granitic gneiss rocks	9.00	34.00	17.00	1.50	-	Multi (2)	-	-

Note :

USBR formula for depth of curtain grouting : $D = H/3 + C$, where $C = 8$ to 25

-	means : details not available

Table 3.4.4: Summary of Consolidation Grouting

No.	Name of dam	Dam type	Dam Height (m)	Country	Foundation Material Type	Consolidation Grouting				
						Depth (m)	Space of holes (m)	Diameter of holes (mm)	Grouting criteria	Max. grouting pressure (kg/cm^2)
1	Sardar Sarovar dam	Concrete gravity dam	163.00	India	Basalt, argillaceous sandstones,quartzitic sandstone & limestone rocks	30.00	3.00	35.00	-	-
2	Nagarjunasagar dam	Masonry gravity dam	124.66	India	Granite-gneiss & quartzite rocks	Shallow	7.62	63.50	-	7.03
3	Keban dam	Composite dam	209.20	Turkey	Karstified metamorphic limestone & schist rocks	15.00	1.50	-	<50kg/m	2.11
4	Liujiaxia dam	Concrete gravity dam	147.00	China	Micaceous quartz schist rock	10.00	1.50	-	-	-
5	El Atazar dam	Concrete arch dam	133.00	Spain	Quartzite & slate rocks	30.00	2.50	-	-	12.00
6	Bhakra dam	Concrete gravity dam	225.60	India	Sandstone, claybeds & conglomerate rocks	15.20	-	-	<1.49 l/min/m	7.03
7	Glen Canyon dam	Concrete arch dam	216.40	USA	Sandstone, shale rocks	7.62	6.00	-	-	-
8	Itaipu dam	Composite dam	196.00	Brazil	Basalt & breccia rocks	3.00	6.00	-	<12.5 kg/m	1.00
9	Hungary Horse dam	Concrete arch dam	171.91	USA	Limestone rock	-	3.00	35.00	-	-
10	Alcantara dam	Concrete buttress dam	135.00	Spain	Slaty schist rock	12.00	2.00	-	-	-
11	Cortes dam	Concr.arch gravity dam	116.00	Spain	Granite gneiss rock	9.14	-	25.40	-	-
12	Clyde dam	Concrete gravity dam	102.00	New Zealand	Quartz, albite, muscovite & chlorite micas rocks	10.00	-	-	-	-
13	Badush dam	Composite dam	90.00	Iraq	Marls, sandstone, gypsum & limestone rocks	10.00	-	-	-	-
14	East canyon dam	Concrete arch dam	79.19	USA	Quartzite & limestone rocks	9.14	3.00	-	-	Low pressure
15	Urugua-I RCC dam	Concrete gravity dam	76.00	Argentina	Basalt rock & lateritic soil	25.00	6.00	-	<50 kg/m	-
16	Hitokura dam	Concrete gravity dam	75.00	Japan	Slate & sandstone rocks	8.00	2.00	-	5 Lu	-
17	Canyon Ferry dam	Concrete gravity dam	68.50	USA	Shale rock	12.20	6.00	25.40	<0.03m^3/5 min	14.00
18	Kadana dam	Masonry -concr. grav. dam	66.00	India	Quarzites, quartz-mica schist & phyliites rocks	15.00	3.00	-	-	7.00

Note :

-	means : details not available

Table 3.4.8: Relation of Depth of Consolidation Grouting with Dam Height and Type of Foundation Rock

No	Depth of Consolidation Grouting (D)	Frequency	Percentage (%)	Dam Height (m)	Type of Foundation Rock
1	$D \leq 5$ m	1	6.25	196	Basalt and breccia
2	$5 \text{ m} < D \leq 20$ m	12	75.00	41.80 to 238	Schist, sandstone, shale, gneiss, quartzite, albite, muscovite, mica, marls, gypsum, limestone, slate, conglomerate & phyllite
3	$20 \text{ m} < D \leq 30$ m	3	18.75	76, 133, 163	Basalt, quartzite, slate, sandstone & limestone
4	$D > 30$ m	0	0.00	-	-
Total :		16	100		

Table 3.4.9: Relation of Space of Consolidation Grouting with Dam Height and Type of Foundation Rock

No	Space of Consolidation Grouting (S)	Frequency	Percentage (%)	Dam Height (m)	Type of Foundation Rock
1	$1 \text{ m} < S \leq 3$ m	9	64.29	209.20,171.91,163, 147, 135, 133, 79.19, 75, 66	Basalt, sandstone, limestone, quartzite, slate, phyllite, schist
2	$S = 6$ m	4	28.57	216.40, 196, 76, 68.50	Sandstone, shale, basalt & breccia
3	$S > 6$ m	1	7.14	124.66	Gneiss & quartzite
Total :		14	100		

Table 3.4.10: Relation of Number of Foundation Drainage Gallery with Height and Type of The Dam

No	Number of Drainage Gallery	Frequency	Percentage (%)	Dam Height (m)	Dam Type
1	1	10	66.67	61, 75	Gravity (4), arch (3), Buttress (1), Arch gravity (2)
2	More than 1	5	33.33	76, 144, 147, 163, 225	Gravity
Total :		15	100		

Table 3.4.11: Relation of Depth of Drainage Holes with Grout Curtain Holes

No	Depth of Drainage Curtain (%depth of grout curtain)	Frequency	Percentage (%)	Dam Height (m)
1	$D \leq 20$	3	14.29	61, 74.1, 75
2	$20 < D \leq 40$	5	23.81	147, 144, 225.6, 216.4, 135
3	$40 < D \leq 60$	6	28.57	124.66, 125.9, 116, 86.6, 70.1, 41.8
4	$60 < D \leq 80$	3	14.29	87, 99.7, 110.3,
5	$80 < D \leq 100$	4	19.05	163, 133, 130.8, 68.5
Total :		21	100	

Table 3.4.12: Relation of Space of Drainage Holes with Dam Height and Type of Foundation Rock

No	Space of Drainage Curtain (S)	Frequency	Percentage (%)	Dam Height (m)	Type of Foundation Rock
1	S = 2 m	1	4.55	147	Schist
2	2 m < S < 3 m	2	9.09	74.10, 76.80	Sandstone, shale & dolomite
3	S = 3 m	12	54.55	41.80 to 225.60	Basalt, sandstone, limestone, conglomerate, shale, amphibolite, andesite, gneiss, argillite, basalt, quartzite & granodiorite
4	3 m < S < 4 m	1	4.55	61	Gneiss
5	S = 4 m	2	9.09	76, 135	Basalt & schist
6	4 m < S < 5 m	1	4.55	86.60 m	Dolomite
7	S = 5 m	2	9.09	75, 87	Slate, sandstone & schist
8	5 m < S < 6 m	0	0.00	-	-
9	S = 6 m	1	4.55	124.66	Gneiss and quartzite
Total :		22	100		

Table 3.4.4: Summary of Consolidation Grouting

No.	Name of dam	Dam type	Dam Height (m)	Country	Foundation Material Type	Consolidation Grouting				
						Depth (m)	Space of holes (m)	Diameter of holes (mm)	Grouting criteria	Max. grouting pressure (kg/cm^2)
1	Sardar Sarovar dam	Concrete gravity dam	163.00	India	Basalt, argillaceous sandstones, quartzitic sandstone & limestone rocks	30.00	3.00	35.00	-	-
2	Nagarjunasagar dam	Masonry gravity dam	124.66	India	Granite-gneiss & quartzite rocks	Shallow	7.62	63.50	-	7.03
3	Keban dam	Composite dam	209.20	Turkey	Karstified metamorphic limestone & schist rocks	15.00	1.50	-	<50kg/m	2.11
4	Liujiaxia dam	Concrete gravity dam	147.00	China	Micaceous quartz schist rock	10.00	1.50	-	-	-
5	El Atazar dam	Concrete arch dam	133.00	Spain	Quartzite & slate rocks	30.00	2.50	-	-	12.00
6	Bhakra dam	Concrete gravity dam	225.60	India	Sandstone, claybeds & conglomerate rocks	15.20	-	-	<1.49 l/min/m	7.03
7	Glen Canyon dam	Concrete arch dam	216.40	USA	Sandstone, shale rocks	7.62	6.00	-	-	-
8	Itaipu dam	Composite dam	196.00	Brazil	Basalt & breccia rocks	3.00	6.00	-	<12.5 kg/m	1.00
9	Hungary Horse dam	Concrete arch dam	171.91	USA	Limestone rock	-	3.00	35.00	-	-
10	Alcantara dam	Concrete buttress dam	135.00	Spain	Slaty schist rock	12.00	2.00	-	-	-
11	Cortes dam	Concr.arch gravity dam	116.00	Spain	Granite gneiss rock	9.14	-	25.40	-	-
12	Clyde dam	Concrete gravity dam	102.00	New Zealand	Quartz, albite, muscovite & chlorite micas rocks	10.00	-	-	-	-
13	Badush dam	Composite dam	90.00	Iraq	Marls, sandstone, gypsum & limestone rocks	10.00	-	-	-	-
14	East canyon dam	Concrete arch dam	79.19	USA	Quartzite & limestone rocks	9.14	3.00	-	-	Low pressure
15	Urugua-I RCC dam	Concrete gravity dam	76.00	Argentina	Basalt rock & lateritic soil	25.00	6.00	-	<50 kg/m	-
16	Hitokura dam	Concrete gravity dam	75.00	Japan	Slate & sandstone rocks	8.00	2.00	-	5 Lu	-
17	Canyon Ferry dam	Concrete gravity dam	68.50	USA	Shale rock	12.20	6.00	25.40	<0.03m^3/5 min	14.00
18	Kadana dam	Masonry -concr. grav. dam	66.00	India	Quarzites, quartz-mica schist & phyliites rocks	15.00	3.00	-	-	7.00

Note :

- means : details not available

Table 3.4.5: Summary of Drainage Curtain

No.	Name of dam	Dam type	Dam Height (m)	Country	Foundation Material Type	Foundation Drainage Curtain: Depth (%of max.depth of grout curtain holes)	Space of holes (m)	Diameter of holes (mm)	Number of foundation drainage gallery (curtain)
1	Sardar Sarovar dam	Concrete gravity dam	163.00	India	Basalt, argillaceous sandstones,quartzitic sandstone & limestone rocks	100.00	3.00	75.00	3
2	Nagarjunasagar dam	Masonry gravity dam	124.66	India	Granite-gneiss & quartzite rocks	53.32	6.00	76.20	1
3	Liujiaxia dam	Concrete gravity dam	147.00	China	Micaceous quartz schist rock	20.83	2.00	130.00	3
4	El Atazar dam	Concrete arch dam	133.00	Spain	Quartzite & slate rocks	88.89	-	-	1
5	Srisailam dam	Masonry -concr. Gr.dam	144.00	India	Quarzites & shales rocks	40.00	-	-	2
6	Bhakra dam	Concrete gravity dam	225.60	India	Sandstone, claybeds & conglomerate rocks	30.47	3.00	76.20	2
7	Hoover dam	Concrete gravity dam	221.28	USA	Breccia rock	-	-	-	-
8	Glen Canyon dam	Concrete arch dam	216.40	USA	Sandstone & shale rocks	34.12	3.00	76.20	1
9	Alcantara dam	Concrete buttress dam	135.00	Spain	Slaty schist rock	25.00	4.00	75.00	1
10	Pine Flat dam	Concrete gravity dam	130.80	USA	Amphibolite rock	100.00	3.00	-	-
11	Detroit dam	Concrete gravity dam	125.90	USA	Andesite rock	48.68	3.00	-	-
12	Cortes dam	Concr.arch gravity dam	116.00	Spain	Granite gneiss rock	50.00	3.00	-	1
13	Libby dam	Concrete gravity dam	110.30	USA	Argillite rock	65.31	3.00	-	-
14	Green Peter dam	Concrete gravity dam	99.70	USA	Basalt rock	70.49	3.00	-	-
15	Bemposta dam	Concr.arch gravity hollow dam	87.00	Portugal	Crystalline schist rock	66.67	5.00	-	1
16	Bull shoals	Concrete gravity dam	86.60	USA	Dolomite rock	50.00	4.60	-	-
17	East canyon dam	Concrete arch dam	79.19	USA	Quartzite & limestone rocks	-	3.00	76.20	1
18	Table rock dam	Concrete gravity dam	76.80	USA	Dolomite & sandstone rocks	-	2.70	-	-
19	Urugua-I RCC dam	Concrete gravity dam	76.00	Argentina	Basalt rock & lateritic soil	-	4.00	-	2
20	Hitokura dam	Concrete gravity dam	75.00	Japan	Slate & sandstone rocks	10.10	5.00	66.00	1
21	Greers Ferry dam	Concrete gravity dam	74.10	USA	Sandstone & shale rocks	18.00	2.40	-	-
22	Chief Joseph dam	Concrete gravity dam	70.10	USA	Granodiorite rock	52.17	3.00	-	-
23	Canyon Ferry dam	Concrete gravity dam	68.50	USA	Shale rock	98.36	3.00	-	1
24	Clark hill dam	Concrete gravity dam	61.00	USA	Biotite gneiss rock	16.98	3.70	-	-
25	Ranapratap Sagar dam	Masonry gravity dam	42.67	India	Sandstone & shale rocks	-	-	75.00	1
26	Conemaugh dam	Concrete gravity dam	41.80	USA	Sandstone rock	40.54	3.00	-	-

Note :

\- means : details not available

Table 3.4.6: Relation of Maximum Depth of Curtain Grouting Holes with Dam Height and Type of Foundation Rock

No	Max. Depth of Curtain Grouting Holes (Hc)	Frequency	Percentage (%)	Dam Height (H)	Type of Foundation Rock
1	$Hc \leq H/3$	7	18.92	27 m to 200 m	Basalt, sandstone, limestone,granite, shale, granodiorite & gneiss
2	$H/3 < Hc \leq H$	27	72.97	41.80 m to 238 m	Gneiss, quartzite, slate, sandstone, conglomerate, granodiorite, shale, amphibolite, argillite, dolomite & limestone, basalt, breccia, andesite, marl, gypsum & phyllite
3	$Hc > H$	3	8.11	50 m, 75 m, 165 m	Limestone, Schist and Slate
Total :		37	100		

Table 3.4.7: Relation of Minimum Space of Curtain Grouting Holes with Dam Height and Type of Foundation Rock

No	Min. Space of Curtain Grouting Holes (S)	Frequency	Percentage (%)	Dam Height (m)	Type of Foundation Rock
1	$S \leq 1$ m	3	8.57	238, 76.80, 74.10	Llimestone, dolomite, sandstone & shale
2	1 m< $S \leq 2$ m	19	54.29	27 to 165	Schist, limestone, sandstone, conglomerate, granodiorite, basalt, breccia, amphibolite, andesit, gneiss, slate, marls, gypsum, dolomite, shale, quartzite, & phyllite
3	2 m< $S \leq 3$ m	11	31.43	50 to 209.20	Basalt, sandstone, limestone, gneiss, quartzite, schist, sandstone, shale & argillite
4	$S > 3$ m	2	5.71	133, 68.50	Quartzite, slate & shale
Total :		35	100		

Table 3.4.8: Relation of Depth of Consolidation Grouting with Dam Height and Type of Foundation Rock

No	Depth of Consolidation Grouting (D)	Frequency	Percentage (%)	Dam Height (m)	Type of Foundation Rock
1	$D \leq 5$ m	1	6.25	196	Basalt and breccia
2	5 m $< D \leq 20$ m	12	75.00	41.80 to 238	Schist, sandstone, shale, gneiss, quartzite, albite, muscovite, mica, marls, gypsum, limestone, slate, conglomerate & phyllite
3	20 m $< D \leq 30$ m	3	18.75	76, 133, 163	Basalt, quartzite, slate, sandstone & limestone
4	$D > 30$ m	0	0.00	-	-
Total :		16	100		

Table 3.4.9: Relation of Space of Consolidation Grouting with Dam Height and Type of Foundation Rock

No	Space of Consolidation Grouting (S)	Frequency	Percentage (%)	Dam Height (m)	Type of Foundation Rock
1	1 m $< S \leq 3$ m	9	64.29	209.20,171.91,163, 147, 135, 133, 79.19, 75, 66	Basalt, sandstone, limestone, quartzite, slate, phyllite, schist
2	$S = 6$ m	4	28.57	216.40, 196, 76, 68.50	Sandstone, shale, basalt & breccia
3	$S > 6$ m	1	7.14	124.66	Gneiss & quartzite
Total :		14	100		

Table 3.4.10: Relation of Number of Foundation Drainage Gallery with Height and Type of The Dam

No	Number of Drainage Gallery	Frequency	Percentage (%)	Dam Height (m)	Dam Type
1	1	10	66.67	61, 75	Gravity (4), arch (3), Buttress (1), Arch gravity (2)
2	More than 1	5	33.33	76, 144, 147, 163, 225	Gravity
Total :		15	100		

Table 3.4.11: Relation of Depth of Drainage Holes with Grout Curtain Holes

No	Depth of Drainage Curtain (%depth of grout curtain)	Frequency	Percentage (%)	Dam Height (m)
1	D ≤ 20	3	14.29	61, 74.1, 75
2	20 < D ≤ 40	5	23.81	147, 144, 225.6, 216.4, 135
3	40 < D ≤ 60	6	28.57	124.66, 125.9, 116, 86.6, 70.1, 41.8
4	60 < D ≤ 80	3	14.29	87, 99.7, 110.3,
5	80 < D ≤ 100	4	19.05	163, 133, 130.8, 68.5
Total :		21	100	

Table 3.4.12: Relation of Space of Drainage Holes with Dam Height and Type of Foundation Rock

No	Space of Drainage Curtain (S)	Frequency	Percentage (%)	Dam Height (m)	Type of Foundation Rock
1	S = 2 m	1	4.55	147	Schist
2	2 m < S < 3 m	2	9.09	74.10, 76.80	Sandstone, shale & dolomite
3	S = 3 m	12	54.55	41.80 to 225.60	Basalt, sandstone, limestone, conglomerate, shale, amphibolite, andesite, gneiss, argillite, basalt, quartzite & granodiorite
4	3 m < S < 4 m	1	4.55	61	Gneiss
5	S = 4 m	2	9.09	76, 135	Basalt & schist
6	4 m < S < 5 m	1	4.55	86.60 m	Dolomite
7	S = 5 m	2	9.09	75, 87	Slate, sandstone & schist
8	5 m < S < 6 m	0	0.00	-	-
9	S = 6 m	1	4.55	124.66	Gneiss and quartzite
Total :		22	100		

CHAPTER IV

CASE STUDY OF LAKHWAR DAM

4.1. Project Features

Lakhwar-Vyasi project comprises a 192 m high concrete mass gravity dam at Lakhwar across river Yamuna, upstream of its confluence with its major tributary, the Tons, an underground powerhouse at the toe of the dam on the right bank, a 88 m high diversion dam at Vyasi, about 5 km downstream of Lakhwar, a 7 m diameter, 3.60 km long tunnel feeding a surface powerhouse at Hatyari and a balancing reservoir in the downstream at Katapather so that continuous supplies may be released in the river thereafter for purposes of irrigation. The location map of the scheme is shown in Figure 4.1.1 and the salient feature is given in Table 4.1.1.

At Lakhwar, based on investigations and conventional design, the proposed concrete gravity consisting of 20 blocks with transverse joints ungrouted. Five central blocks formed the spillway. The geological and geotechnical investigations revealed presence of shear zone and complex rock formations with diverse deformabilities necessitated dam foundation interaction analysis.

4.2. Foundation Conditions

4.2.1. Geological features of the foundation

The gorge is narrow at the proposed dam site. The abutments rise steeply above the riverbed. The right abutment is strong but the left abutment spur is narrower with steep hill slopes up to dam height and then follows a gentle rising profile. Besides the narrowness, the joints in the trap rock of the left abutment dip at low angle (about 20° in the downstream).

The dam site lies in lesser Himalayas. The rock formations in the dam area comprise phyllites, slates, quartzites and limestones belonging to the Mandhali, Chandpur and Nagthat series. These have been folded into a major syncline named "Jaunsar Syncline".

The phyllites are intruded by a basic rock, petrological composition of which varies from dolerites to hornblende granites. These have been named as "Jaunsar

Traps". One of these is lenticular in shape along the trend of phyllites and cuts across river Yamuna. This trap with the maximum width of 300 m, forms the foundation of the dam.

The geology of the area has been established through the logging of 81 boreholes and 33 drifts at different locations and elevations in the area. Some holes were bored as deep as 150 m to 180 m. The geological features are shown in Figure 4.2.1.

The traps are coarse grained, highly jointed and surrounded by slates. The upstream slates contain quartzitic bands. The downstream slates are thinly foliated. These foliations dip at an angle of 70° to 80° in the upstream. A few pockets of highly jointed quartzitic slates have also been found in the downstream.

Just beyond the toe of the proposed dam, a shear zone of about 1.50 m thickness exists between trap and thinly foliated slates. This shear zone, dipping at an angle of 40° to 50° in the upstream beneath the central blocks of the dam traverses into the left abutment, and is the additional cause of weakness of the left abutment.

On the left abutment, an outcrop of slates and quartzites enclosed on three sides by the trap, forms a xenolith body of rock. This xenolith body is highly jointed and extends in a wide area and up to a depth of el. 660.00 m.

In order to explore more thoroughly the geology of foundation rocks and the presence, nature and extent of shear zone encountered during excavation of an adit to powerhouse, a shaft was excavated on the right bank to lead to two under-river drifts. The 130 m long under-river drift at about 58 m below riverbed confirmed the presence of shear zone.

The 3-D geological logging of the drift traversing in trap, shear zone and downstream slates indicated the rock conditions as below:

i. Basic rocks are dark greenish gray in color and moderately jointed. In the vicinity of the shear zone, these are partly crushed and sheared.

ii. Existence of a shear zone of 1.20 m (+ 0.40 m) thick and filled with gouge material has been confirmed. It dips at 38° to 45° in N15° E to N 22° E direction.

iii. Beyond shear zone, there are thinly foliated slates with foliation plane dipping at 75° to 78° in N 5° W to N 18° E direction. At the contact with shear zone, the slates are graphitic but at places contain some hard siliceous bands. In these slates, the conditions were found dry to moist.

iv. The slates are found harder and better as one goes deep into it.

4.2.2. Geomechanical properties of the foundation

a. The investigations

Usual geotechnical and geophysical investigations have been carried out by various agencies including Geological Survey of India to assess the depths of overburden, to delineate the contact between basic trap rock and slates, the nature of rock, its strength and shear parameters, etc.

For further analysis, the strength and shear parameters data are collected from Irrigation Design Office, Roorkee, and the selected values of modulus of deformation of rocks are as written by B.N. Asthana [IWRS journal, July 1998] as given in the following paragraph.

b. Strength and shear parameters of trap rock

i. Average compressive strength : 570 kg/cm^2

ii. Average tensile strength : 70 kg/cm^2

iii. Specific gravity : 2.76

iv. Average internal friction angle of rock to rock : 56°

v. Average internal friction angle of rock to concrete : 49°

vi. Average cohesion rock to rock : 3.20 kg/cm^2

vii. Average cohesion rock to concrete : 2.50 kg/cm^2

viii. Poisson ratio of each rock within the foundation : 0.28

c. Modulus of deformation of rocks (Ed)

i. Trap : 12.50 x 10^4 kg/cm^2

ii. Quartzitic slate : 2.50 x 10^4 kg/cm^2

iii. Gouge : 0.20 x 10^4 kg/cm^2

iv. Thinly foliated slate : 1.00 x 10^4 kg/cm^2

d. Data on concrete

i. Poisson ratio : 0.20

ii. Unit weight : 2.40 t/ m^3

iii. Modulus of deformation : 20.00 x 10^4 kg/cm^2

4.3. Influence of Foundation Conditions in Design of the Dam

As described in previous subchapter, usual geotechnical and geophysical investigations have been carried out. Wide range tests have been conducted to ascertain the strength parameters, shear parameters and modulus of deformation of various types of rock encountered in the foundation of Lakhwar dam.

Modulus of deformation is an important parameter, which influences the design of dam [Asthana, B.N., IWRS journal, July 1998]. Insitu tests for modulus of deformation of each rock have been carried out in open trench and in the drifts, using the method of:

i. Plate jack test (PJT)

ii. Plate jack test with borehole extensions (PJTB)

iii. Flat jack test (FJT)

The total number of test conducted and the range of modulus of deformation values obtained are given in Table 4.3.1.

The modulus values of the downstream thinly foliated slates are low. These rocks on visual inspection appeared satisfactory. These, however, showed weakness when seepage occurred through them. The test conditions were thoroughly examined to investigate the reasons for such low modulus values.

It was found that the tests carried out immediately after excavation gave higher values. These slates are found to disintegrate when they become wet through seepage. Thus, it was decided that a series of tests should be carried out promptly soon after excavation. A drift was manually excavated and the PJT tests were conducted with minimum lapse of time. The results are shown in Table 4.3.2.

The existence of trap with its high value of deformation modulus and the occurrence of shear zone (gouge material) in between the trap and the slate makes the existing condition of dam foundation more complex.

On one hand, according to the discussion in Chapter II, the high value of deformation modulus of trap rock makes this site suitable for a concrete gravity dam, but on the other side the occurrence of thinly foliated Slate rock and shear zone with low values of deformation modulus make the foundation incompetent.

However, with the existing foundation conditions at Lakhwar dam site, the analysis of the dam-foundation behavior (stresses and deformations) can only decide the suitability for a concrete gravity dam. This analysis has been carried out and presented in Chapter V.

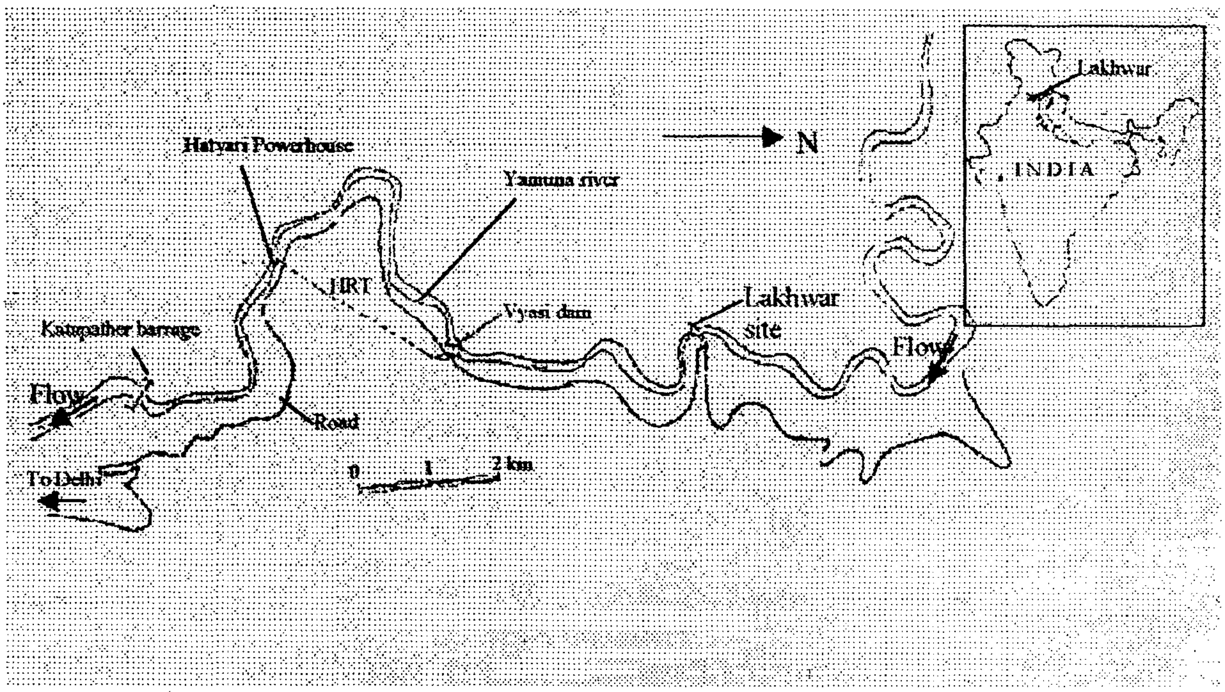


Figure 4.1.1: Location Map of
Lakhwar-Vyasi Project

Table 4.1.1: Salient Feature of Lakhwar - Vyasi Project

1.	Location			4.	Lakhwar dam		
	District	:	Dehradun		Dam base level	:	El. 608.00 m
	State	:	Uttaranchal, India		Crest of dam	:	El. 800.00 m
	Latitude	:	30°31'3".N		Type of dam	:	Concrete gravity dam
	Longitude	:	77°56'58"E				
2.	Hydrological characteristics			5.	Spillway		
	Total catchment area	:	2,080 sq.km		Type	:	Gated spillway
	Snowfed catchment area	:	130 sq.km		No. and size of bay	:	5 bays of 14 m x 15 m each
	Normal annual rainfall	:	1250 mm to 2000 mm		Crest level	:	El. 781.25
	River flow	:	3,000 cumec during monsoon, 25 cumec during winter		Capacity	:	8,000 cumec
	Average annual runoff	:	1,882 million cu.m		Energy dissipator	:	Splitter bucket type
3.	Reservoir at Lakhwar			6.	Benefit		
	Total length of reservoir	:	20 km		Irrigation	:	49,600 hectares
	Full reservoir level	:	El. 796.00 m		Installed capacity	:	At Lakhwar 3 x 100 MW At Hatyari 2 x 60 MW
	Dead storage level	:	El. 752.00 m		Power generation	:	852 million kWh on 90 % water availability
	Total storage	:	580 million cu.m				
	Live storage	:	247 million cu.m				

Reference :
B.N. Asthana, IWRS Journal, July 1998

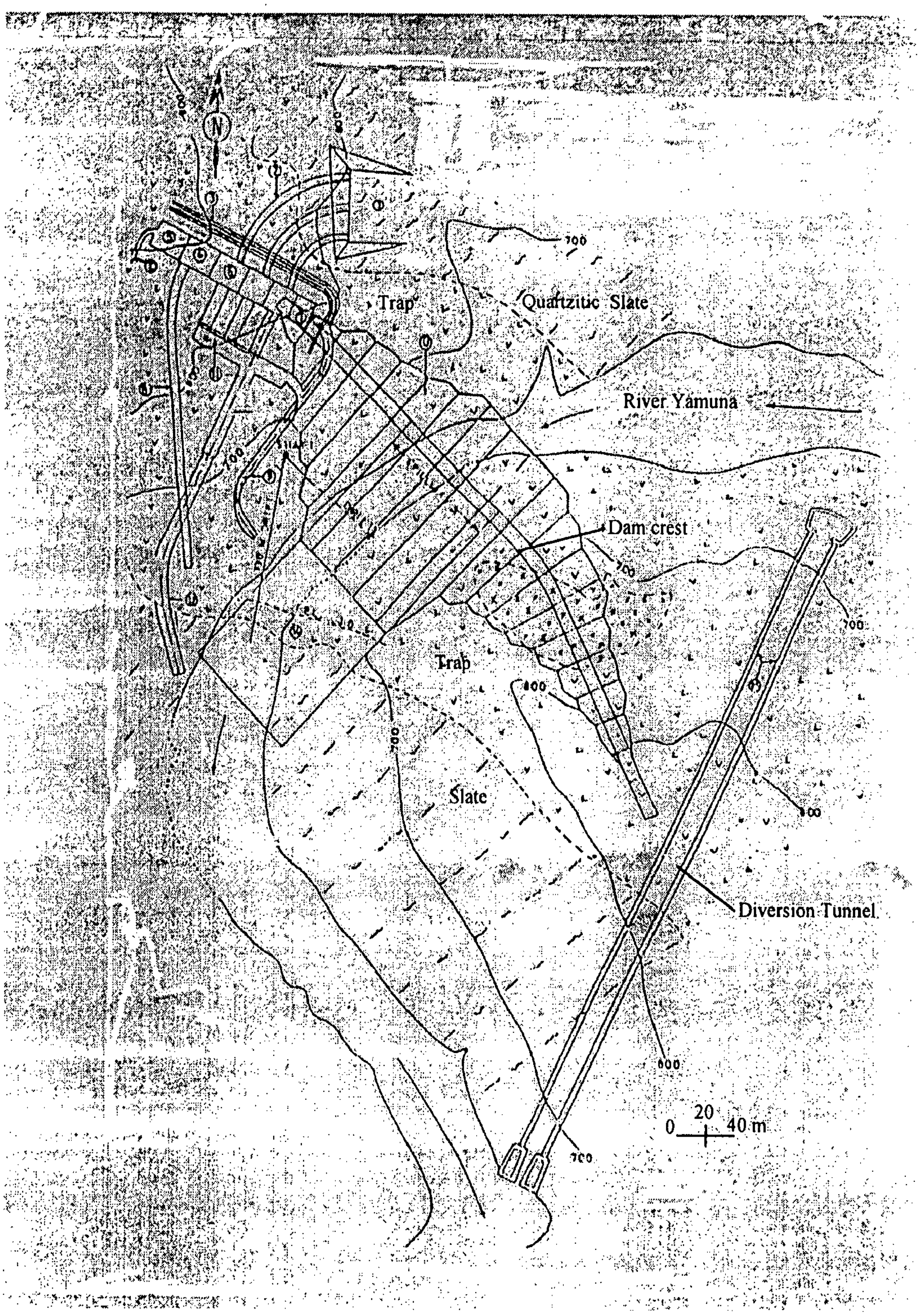


Figure 4.1.2: Lakhwar Dam.
General Layout

Table 4.3.1: Modulus of Deformation of Various Rocks Based on Insitu Tests before Excavation Work

No.	Test	No. of Test	Stress level (kg/sq.cm)	Modulus of Deformation (kg/sq.cm)
1	In Trap rock :			
	PJT	20	56 - 61	7,470 - 69,740
	PJTB	12	20 - 50	17,199 - 478,000
	FJT	5	35 - 70	26,250 - 306,250
2	In u/s Quartzitic Slate rock :			
	PJT	6	28 - 34	6,730 - 40,920
3	In d/s Thinly Foliated Slate rock :			
	PJT	7	10 - 63	1,200 - 14,600
	FJT	4	3 - 30	1,990 - 3,490

Reference :
B.N Asthana [IWRS journal July 1998]

Table 4.3.2: Modulus of Deformation of Thinly Foliated Slate Based on PJT Tests after Excavation Work

No.	Location of the Test	No. of Cycle	Time lapse	Modulus Value of Last Loading (kg/sq.cm)
1	Ch. 19.15	5	within 15 days	17,700
	Ch. 21.00	5	within 25 days	11,990
2	Ch. 23.75	5	within 12 days	24,445
3	Ch. 26.20	5	within 9 days	29,170

Reference :
B.N Asthana [IWRS journal July 1998]

CHAPTER V

ANALYSIS OF DAM-FOUNDATION BEHAVIOR

5.1. Analysis

The foundation of Lakhwar dam consists of different types of rock and a shear zone. The analysis of dam-foundation system based on the characteristics of foundation which influence the behavior (stresses and deformations) of the dam has been carried out with the objective of planning possible treatment of the foundation. To achieve this objective, the analysis of dam-foundation system is addressed to following questions:

i. Whether shear zone has dominant influence on the stability of the dam.

ii. Whether treatment by backfilling concrete up to a particular depth of shear zone will improve the foundation.

iii. Whether resting the dam on one type of rock foundation will be advantageous.

To answer the above questions, the following cases of foundation conditions are analyzed:

Case 1: Existing condition.

Case 2: Homogeneous (Trap rock) condition.

Case 3: Existing condition with 40 m deep backfill concrete.

Case 4: Existing condition with 80 m deep backfill concrete.

Case 5: Slate rock is taken as Trap rock.

Case 6: Existing condition with shifting the dam 40 m upstream.

Case 7: Existing condition with shifting the dam 228 m upstream.

The input data for analysis is given in Figure 5.1.1. The results of the analysis are presented graphically, in tabulation form and in contour form in term of vertical displacement (settlement), horizontal displacement, and normal vertical stress for a few conditions. The details of the analysis and the results are described in following para.

5.2. Dam Section and Boundary Conditions

The non-overflow section of Lakhwar dam and the foundation have been analyzed. The foundation extend considered in the analysis is horizontally 1.5 H from upstream edge of the foundation, 1.5 H from down stream edge of the foundation, and vertically 1.5 H deep below the dam foundation level where H is the height of the dam = 192 m. Both vertical sides and bottom side of the foundation are considered as boundary of the section. The bottom boundary is assumed fixed in both horizontal and vertical directions, whereas the side boundaries are assumed fixed in horizontal direction and free to move in vertical direction. Any phenomena encountered in the foundation beyond this extend is assumed not to influence the behavior of the dam and its foundation. The section is given in Figure 5.1.1.

5.3. The Loads

To evaluate the dam behaviour with various cases of foundation condition, only the following loads are considered in the analysis:

i. Dead weight of the dam
ii. Reservoir water pressure at full reservoir level (El. 796 m)
iii. Normal uplift pressure with drain operative

The foundation rocks are initially stressed by its weight and other tectonic activities before the construction of the dam, therefore, stresses and displacements worked out in this analysis are those of additional stresses and additional displacements due to the loads coming from the dam and other loads mentioned above. Accordingly, in this analysis the density of the foundation materials is taken as zero.

5.4. Sign Conventions

The sign conventions considered in this analysis are:

i. Compressive stress is negative, and tensile stress is positive.
ii. Upward direction of vertical displacement is positive, and downward direction is negative.
iii. Right direction of horizontal displacement is positive, and left direction is negative.

5.5. Finite Element Method

The findings of recent years clearly show that for determination of load carrying capacity and the stability of a concrete dam, it is essential to know the stress distribution and deformations inside the dam and those on dam-foundation contact surface. With the development of finite element method, the method of calculation of stress and deformation is available today, which allows every structure to be calculated independently of the geometry, including the foundation influences, and under any kind of load. Because of this, the degree of accuracy in giving the load carrying capacity and stability can be improved.

The steps involved in the analysis are briefly given as per the followings [Singh, B., et al, 1995]:

i. Subdivision of the section into finite elements of suitable configuration.
ii. Evaluation of element properties.
iii. Assembly of element properties to obtain the global (structural) stiffness matrix and load vector.
iv. Solution of resultant linear simultaneous equations for primary unknowns (displacements) after introducing the boundary conditions.
v. Determination of secondary unknown quantities, such as stresses and strains.

In this study, it is assumed that the geological features of Lakhwar dam foundation do not vary in cross section over the length of the dam. Accordingly the analysis to compute stresses and displacements is carried out using 2-D finite element method using the computer program available with Prof. Ram Pal Singh of WRDTC. The procedure of data preparation and user-computer interaction of the program is given in the Appendix.

5.6. Result of Analysis and Discussion

a. Settlement (vertical displacement)

Figure 5.6.1 shows the settlement of the dam along dam base level under foundation condition of cases 1, 2, 3 and case 4. The maximum settlements of the dam of the existing condition (case 1), 40 m deep backfill concrete (case 2) and 80 m deep backfill concrete (case 3) are relatively same i.e. approximately 210 mm, and occurs at the same point i.e. at toe of the dam. This magnitude of settlement of the dam of these cases is much more than that for homogeneous (Trap rock) condition (case 2) of about 38 mm. It means that the treatment of the shear zone by backfilling concrete is meaningless, and the high magnitude of settlement is not primarily due to the shear zone.

Figure 5.6.2 shows that when the characteristic of Slate rock is taken as that of Trap rock (case 5), the maximum settlement of the dam along dam base level reduces to a magnitude of approximately 45 mm. It revealed that the high magnitude of settlement of the dam at toe is primarily caused by the existence of the Slate rock in the foundation. Therefore, the other cases (case 6 and case 7) were analyzed by shifting the dam toward upstream in order to avoid the Slate rock as much as possible meeting the requirement that the dam shall rest on one type of rock foundation.

When the dam was shifted 40 m toward upstream (case 6), the maximum settlement improved to a magnitude of approximately 170 mm. In this condition, the dam still rested on Trap rock and still the settlement was influenced by Slate rock. When the dam was shifted 228 m (case 7), the magnitude of maximum settlement was about 100 mm but more then that for case 2 (homogeneous condition). In case 7, the dam was resting on Quartzite rock and had less influence of Slate rock. However, the shifting of the dam in upstream could not significantly reduce the settlement of the dam.

b. Horizontal displacement

Figure 5.6.3 shows the horizontal displacement of upstream face of the dam under existing foundation condition (case 1), homogeneous (Trap rock) condition (case 2), 80 m deep backfill concrete (case 4) and when the dam is shifted 228 m upstream (case 7). The magnitude of horizontal displacement of upstream face of the dam of the four cases increases as the height increases from dam base level and is maximum at dam crest. The Figure clearly shows that horizontal displacement is not changed materially when shear zone was replaced by concrete in 80 m depth (case 4). But there was significant reduction in horizontal displacement when dam was shifted in the upstream by 228 m (case 7).

c. Normal vertical stress along dam base level

Figure 5.6.4 shows the normal vertical stress along dam base at foundation level under existing condition (case 1), homogeneous (Trap rock) condition (case 2), existing condition with 80 m deep backfill concrete (case 4) and in case the dam is shifted 228 m toward upstream (case 7). The pattern of the graphs is relatively similar i.e. maximum compressive stress takes place at dam toe and this magnitude much decreases substantially around heel of the dam. The magnitudes of maximum normal vertical stress of case 1, case 2 and case 3 conditions along dam base level are not much different (the maximum difference is about 20%). In case 7, the result is worse than others, the stresses at dam toe and heel are larger than those on other cases. The case 2 (homogeneous) condition is though considered as the best condition, but the maximum normal vertical stresses are found not much different than in the existing condition. This indicated that any treatment of the foundation (backfill concrete or shifting the dam upstream) will not materially improve the existing condition.

d. Maximum normal vertical stress in dam body and foundation

Table 5.6.1 shows the magnitude and location of the maximum compressive and tensile stresses in dam body and foundations under existing condition (case 1).

In dam body (concrete) the maximum compression stress (537.91 T/sq.m) occurs on downstream face near the toe of the dam slightly above foundation level. This magnitude is less as compared to the permissible value of 7 N/sq.mm (approximately equivalent to 700 T/sq.m) as per IS code 6512-1984. The tensile stress takes place not on upstream face but on downstream face in which it meets the requirement (IS code). The magnitude of tensile stress of 5.81 T/sq.m is much less than 1% of concrete strength of approximately 15 T/sq.m (assumed concrete grade M15 is used).

The maximum compressive stress in foundation takes place in Trap rock on contact surface with the dam. The magnitude is 297.85 T/sq.m. As given in Chapter IV, the compressive strength of Trap rock is 570 kg/sq.cm (equivalent to 5,700 T/sq.m). With the safety factor of 4, the magnitude of 297.85 T/sq.m is much less than 1,425 T/sq.m. The maximum tensile stress of 42.74 T/sq.m takes place on joint line with Quartzite rock, 203.22 m u/s of dam axis, 217 m below foundation surface. As given in Chapter IV, tensile strength of Trap rock is 70 kg/sq.cm, which is larger than the tensile stress in the foundation rock.

5.7. Conclusion of The Analysis

According to the above results of the analysis, it can be inferred that as long as treatment of the entire Slate rock by grouting or other methods to increase its modulus of deformation up to that of Trap rock is not feasible, the concrete gravity type dam is not suitable for Lakhwar dam site. Other types of dam could be examined to find the most suitable type, which may be stable on this foundation.

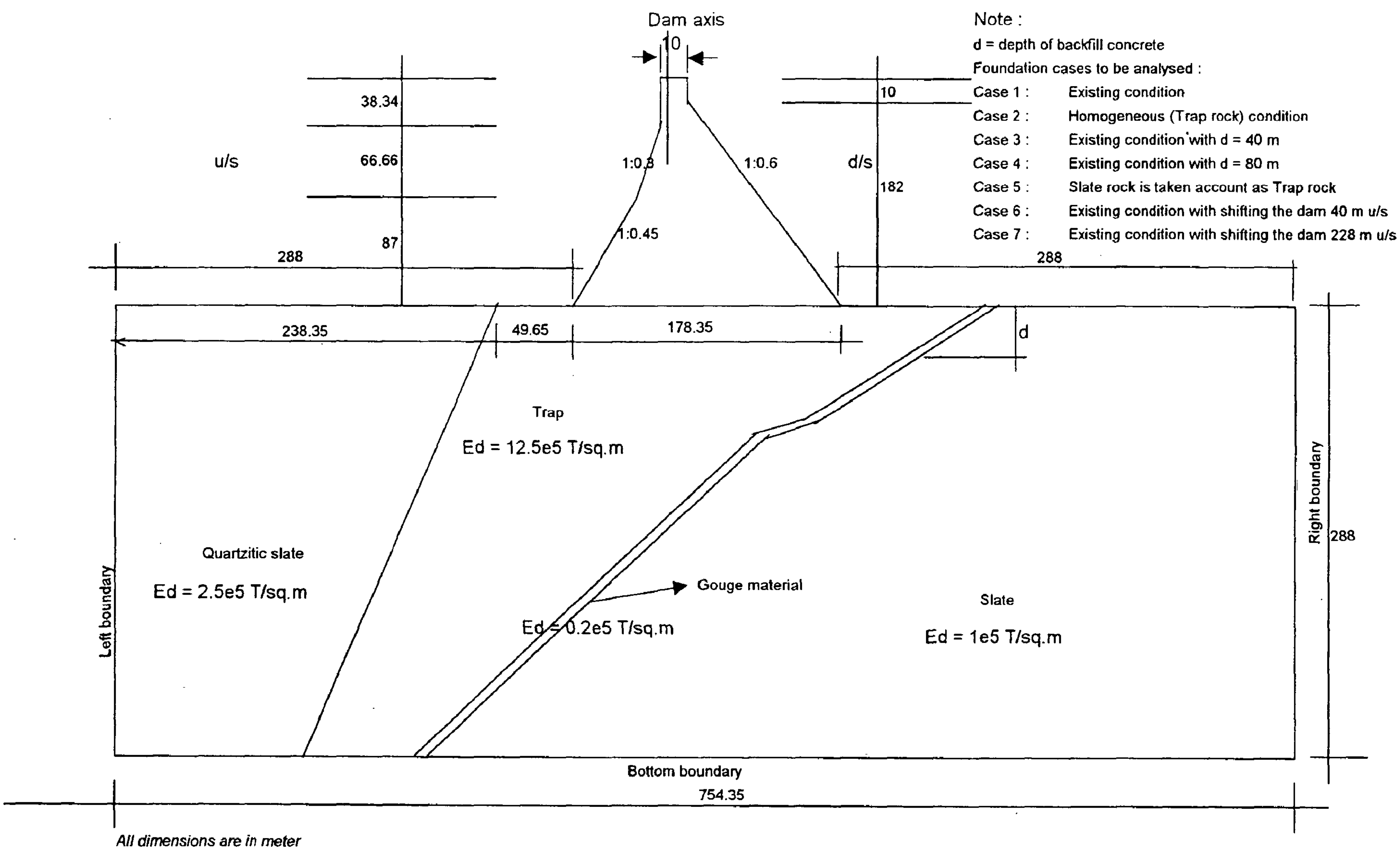


Figure 5.1.1: Dam Section, Boundaries and Cases To be Analysed

Figure 5.6.1: Settlement Along Dam Base Level (Case 1, 2, 3 & 4)

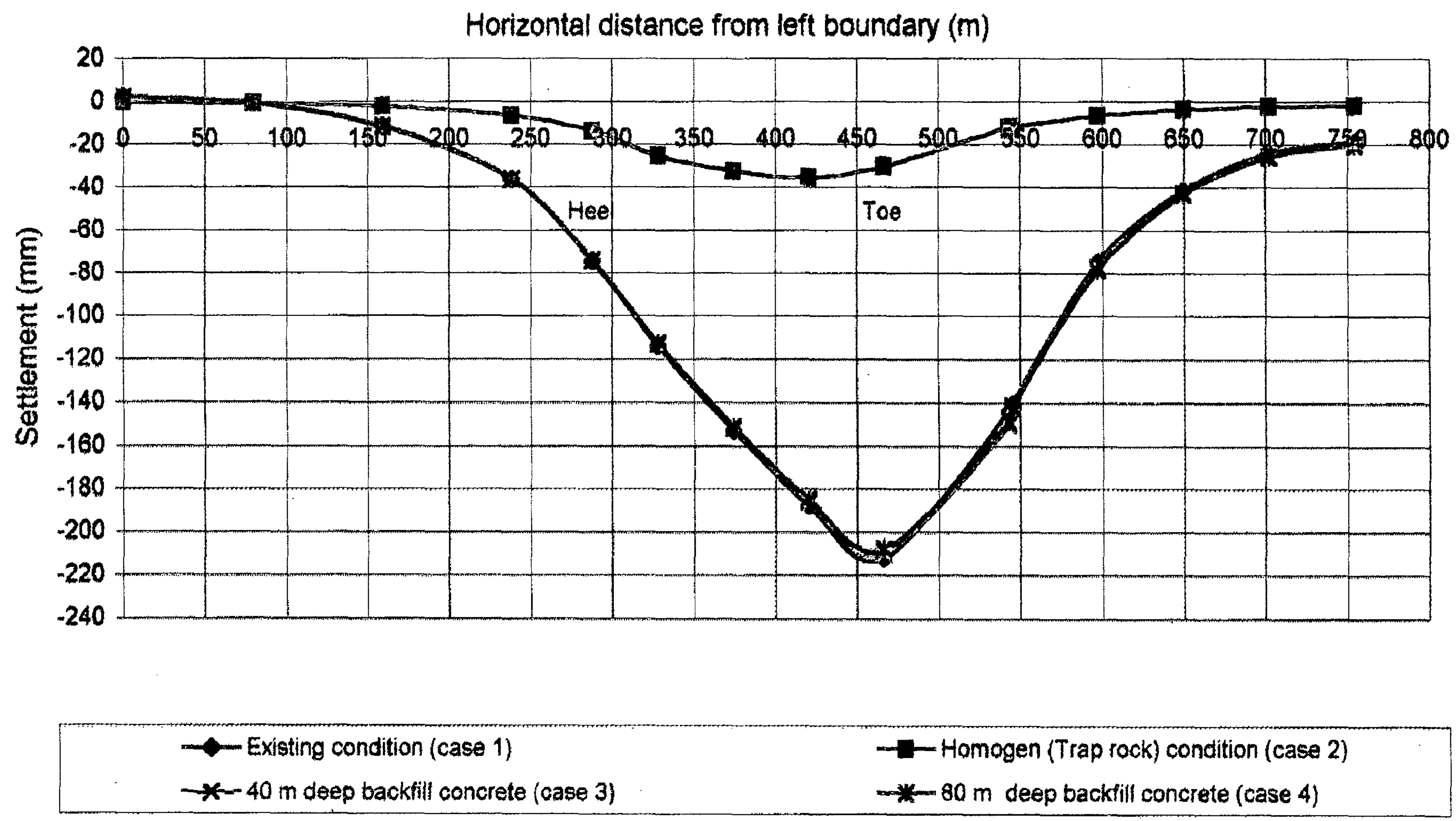


Figure 5.6.2: Settlement Along Dam Base Level (Case 5, 6 & 7)

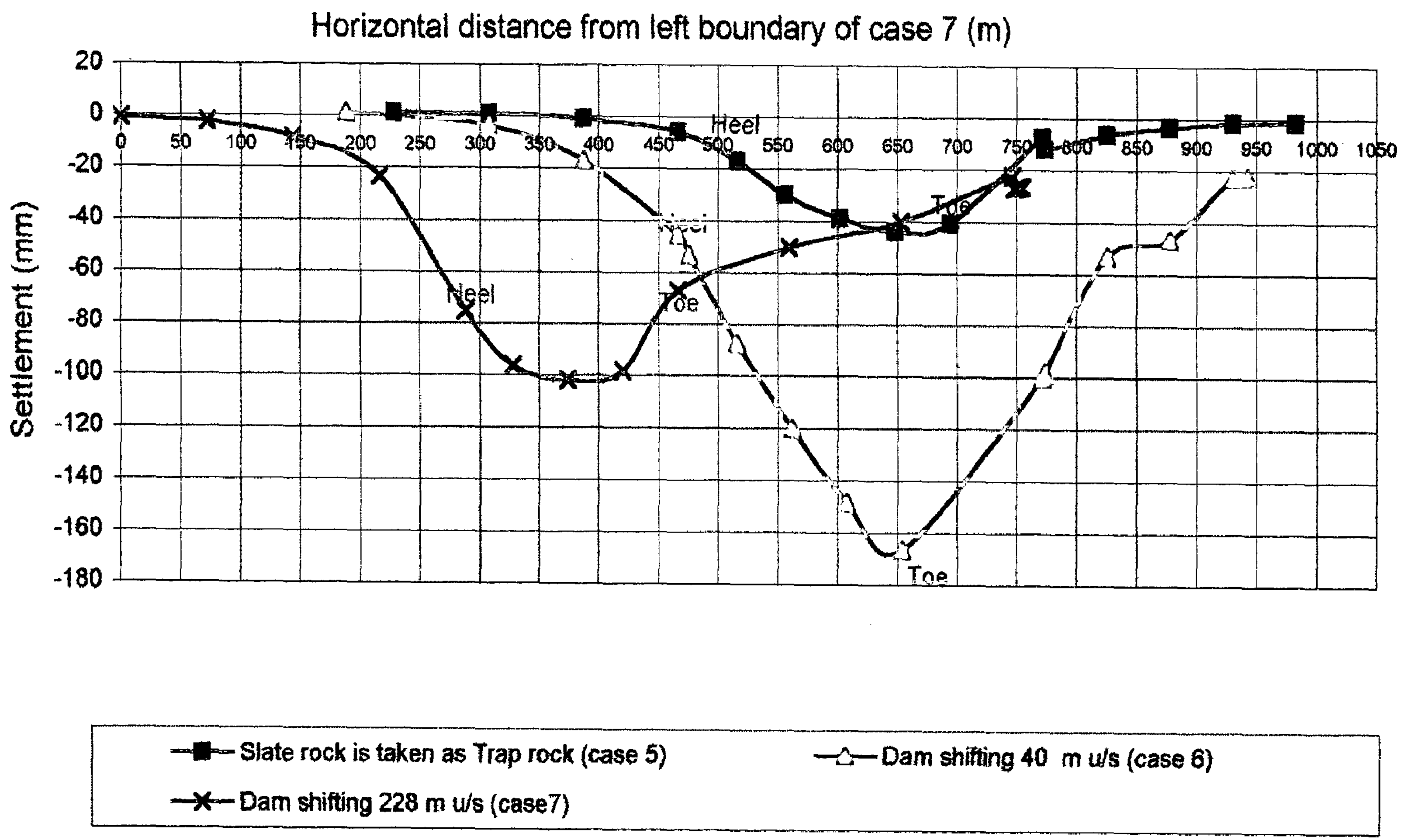


Figure 5.6.3: Horizontal Displacement Along U/s Face of The Dam

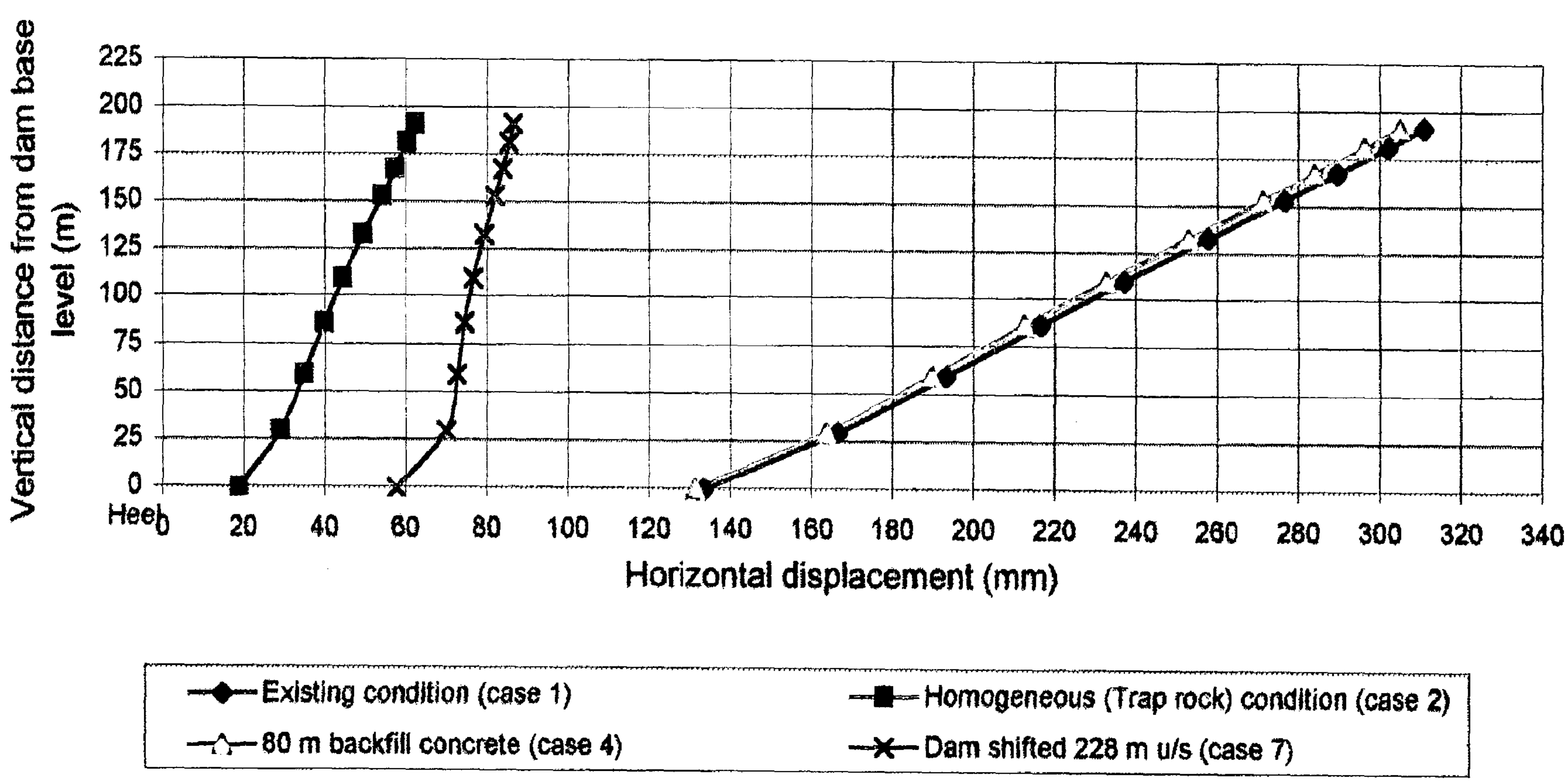


Figure 5.6.3: Horizontal Displacement Along U/s Face of The Dam

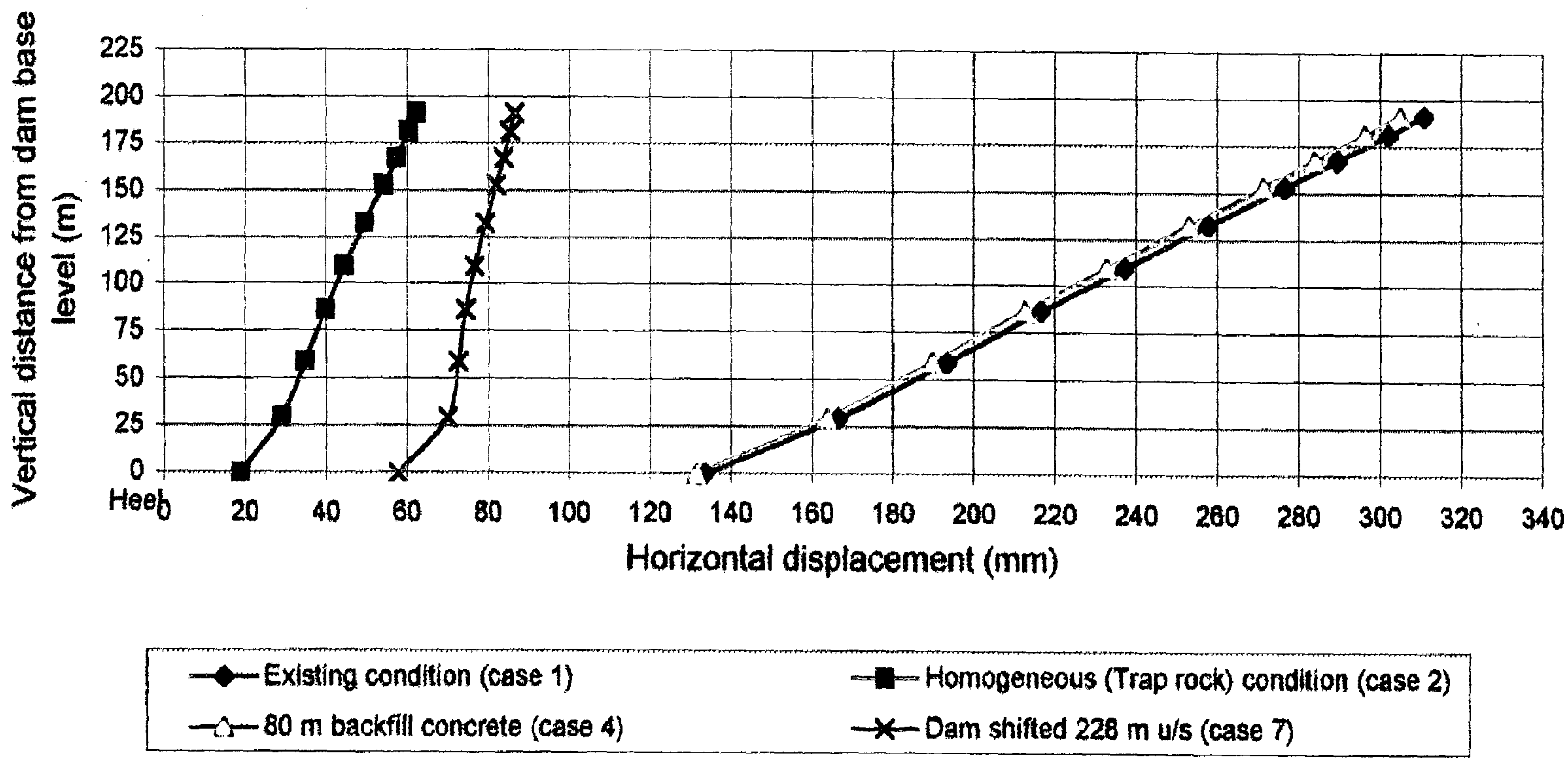


Figure 5.6.4: Normal Vertical Stress Along Dam Base Level

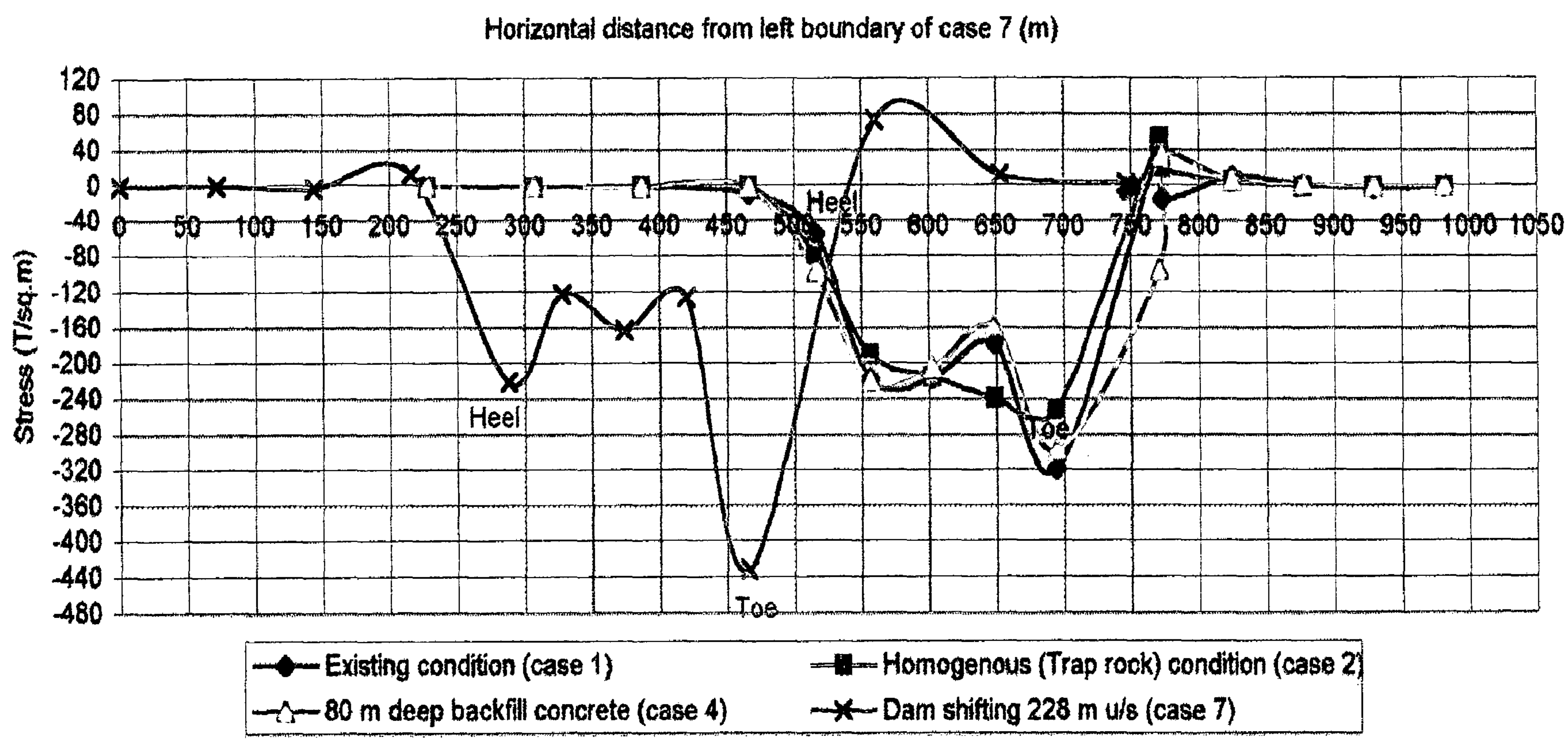


Figure 5.6.4: Normal Vertical Stress Along Dam Base Level

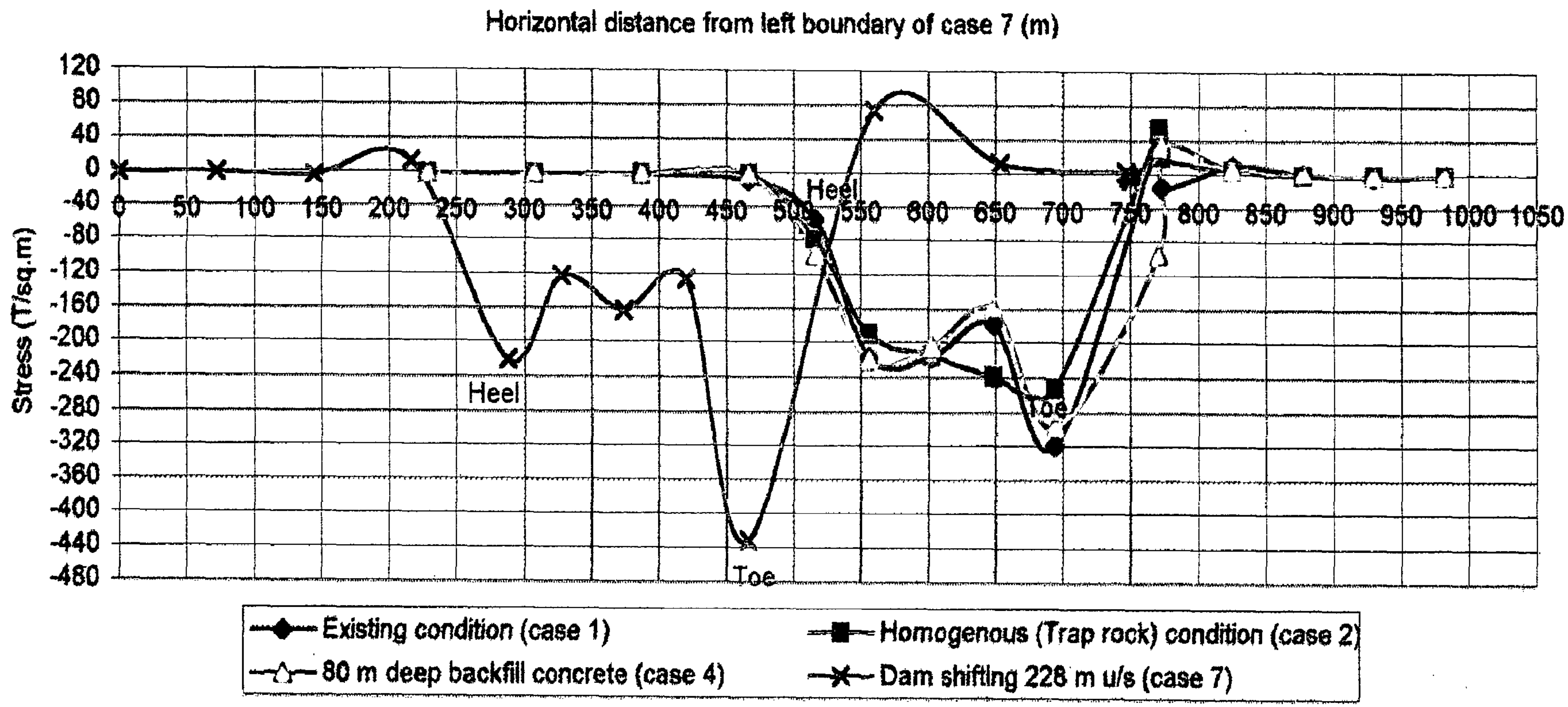


Table 5.6.1: Maximum Normal Vertical Stress in Dam Body and Foundation (Case 1)

Item	In dam body	In foundation			
		Slate rock	Shear zone	Trap rock	Quartz. slate rock
Comp. Stress (T/sq.m)	-537.91	-123.06	-123.06	-297.85	-53.38
Location	d/s face, near the toe, slightly above foundation level	91.80 m d/s of dam axis, 80 m below dam base, joint line with shear zone	91.80 m d/s of dam axis, 80 m below dam base, joint line with Slate rock	at dam toe	at dam heel
Tensile stress (T/sq.m)	5.81	11.03	42.87	42.74	42.74
Location	at d/s crest	on foundation surface, 130.95 m d/s of dam edge	on foundation surface, 76.60 m d/s of dam edge	203.22 m u/s of dam axis, 217 m below found. Surface, joint line with Quartzite rock	203.22 m u/s of dam axis, 217 m below found. Surface, joint line with Trap rock

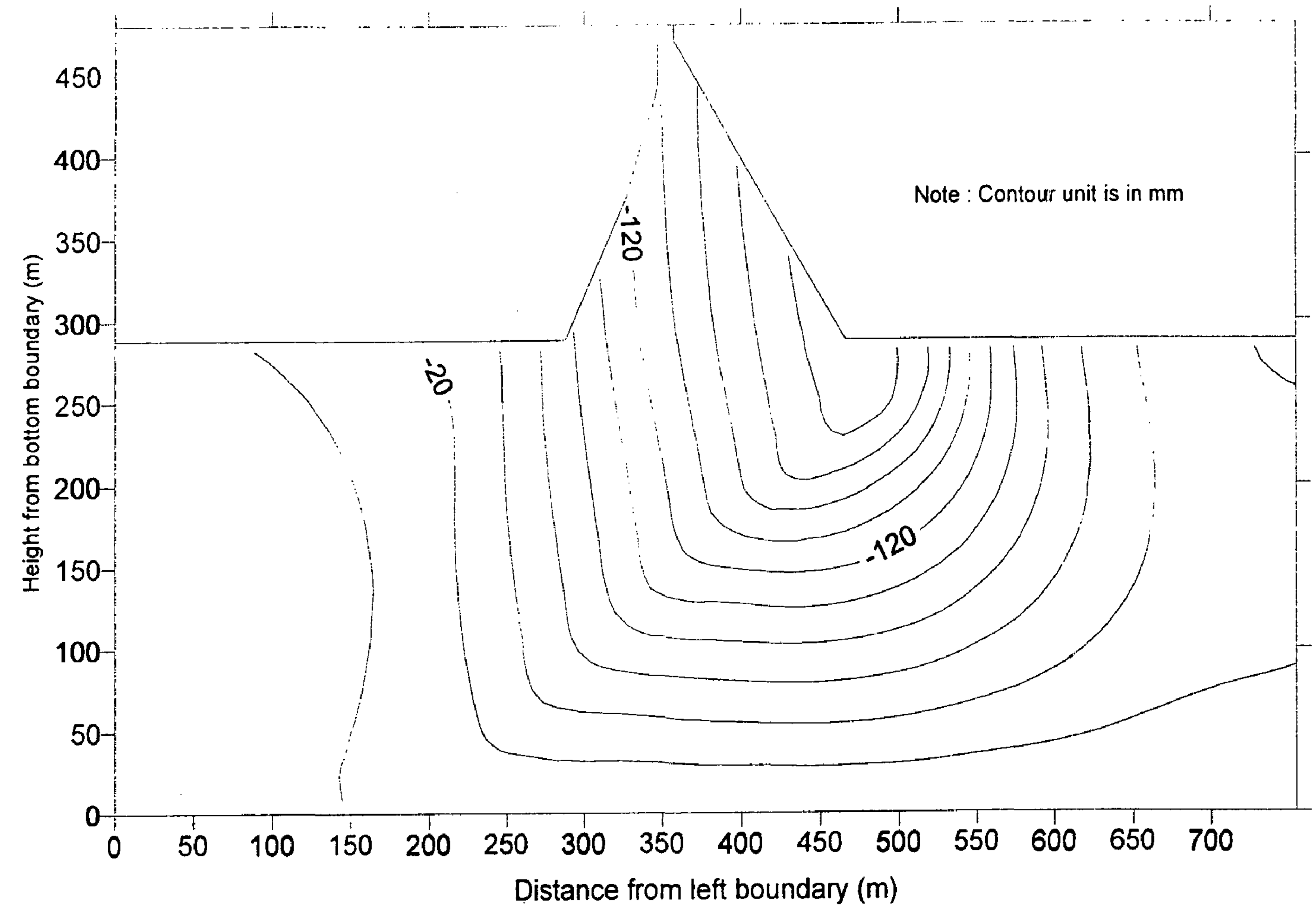


Figure 5.6.5: Contour for Settlement under Existing Condition (Case 1)

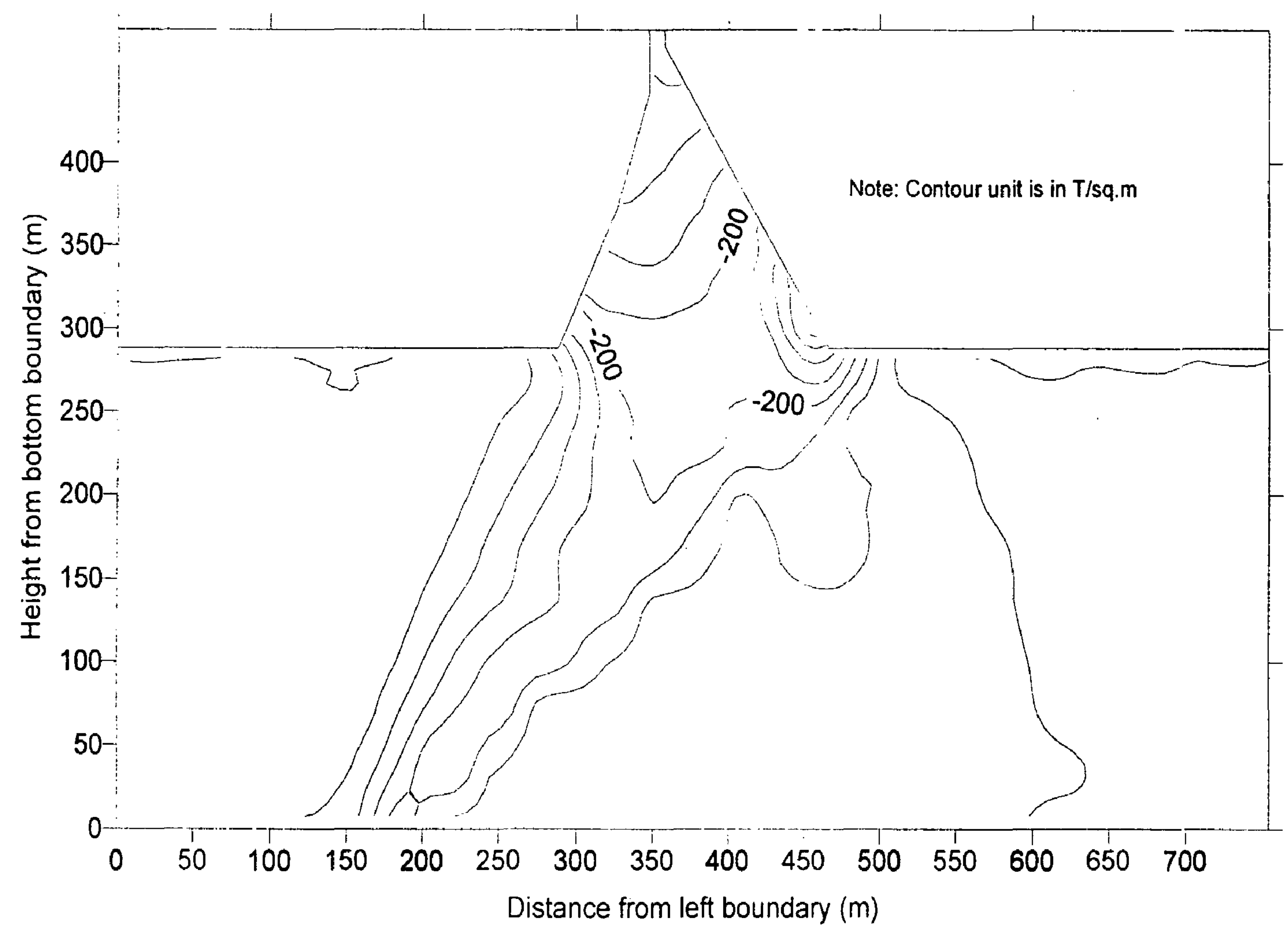


Figure 5.6.6: Contour for Normal Vertical Stress under Existing Condition (Case 1)

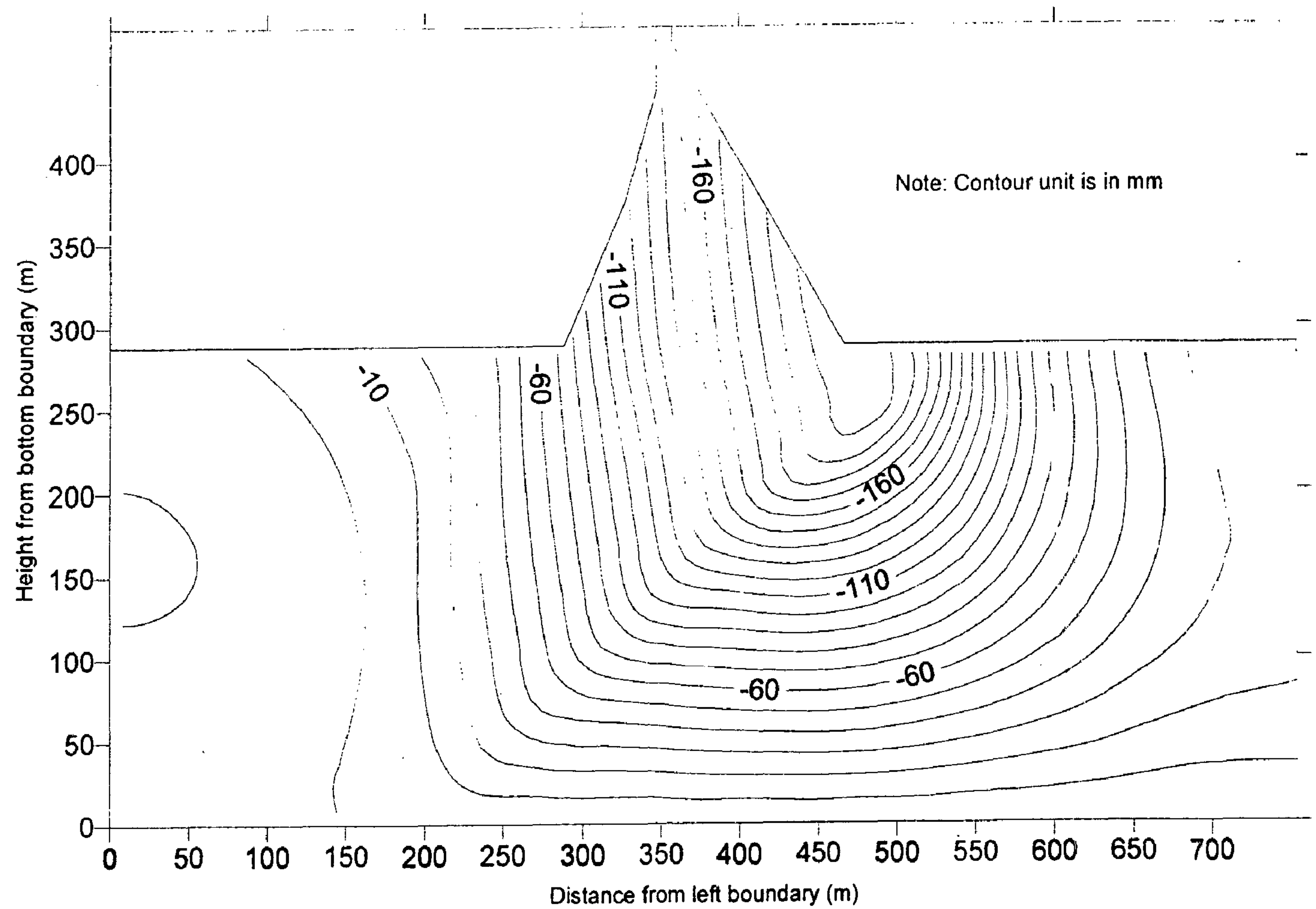


Figure 5.6.7: Contour for Settlement under Existing Condition with 80 m Deep Backfill Concrete (Case 4)

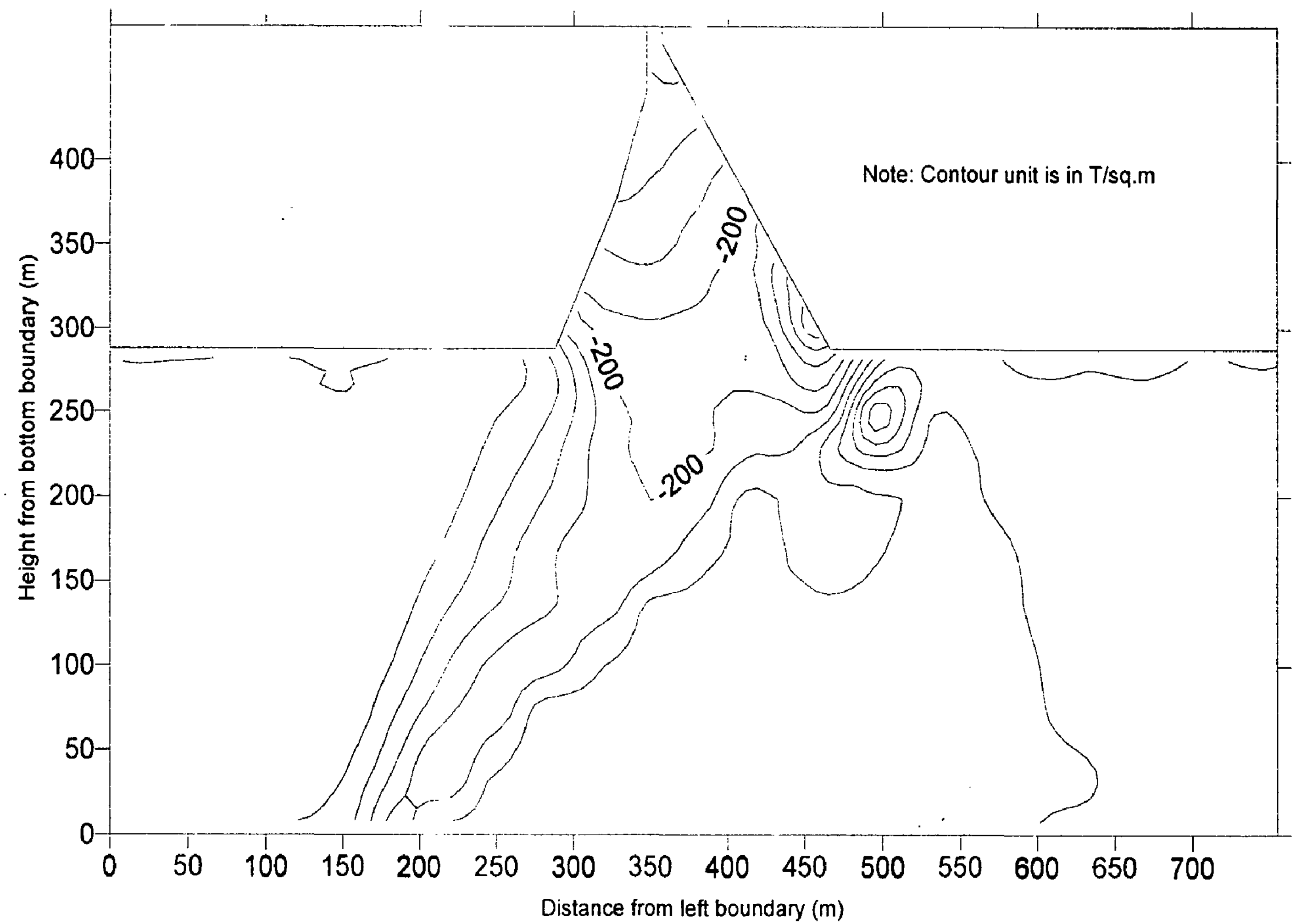


Figure 5.6.8: Contour for Normal Vertical Stress under Existing Condition with 80 m Deep Backfill Concrete (Case 4)

CHAPTER VI

CONCLUSIONS

6.1. General Conclusions on Foundation Treatment

The following inferences are made regarding various parameters of foundation treatment on the basis of this study:

a. *Dimension of curtain grouting*

i. Maximum depth of curtain grouting holes (Hc):

- $H/3 < Hc \leq H$, for dam heights (H) equal to and greater than 41.80 m, and for most types of foundation rock.
- $H/3 < Hc \leq H$ or until penetrating the sound rock whichever is more for the foundations on weak rocks such as Schist.

ii. Minimum space of curtain grouting holes (S):

- $1 \text{ m} < S \leq 2 \text{ m}$ for most dam heights and most types of foundation rocks.

iii. Number of row of curtain grouting:

- Single row for most type of foundation rocks.
- Multi row or by concrete diaphragm wall for foundation rocks like Limestone with Karstic phenomena.

b. *Consolidation grouting*

i. Depth of consolidation grouting holes (d):

- $5 \text{ m} < d \leq 20 \text{ m}$ for most dam heights and all types of foundation rock.

ii. Space of consolidation grouting holes (s):

- $1 \text{ m} < s \leq 3 \text{ m}$ for most dam heights and all types of foundation rock.

c. *Foundation Drainage*

i. Type of foundation drainage:

- Drainage holes have been provided in dams of all heights and most types of foundation rocks.

ii. Depth of drainage holes (Hd):

- $20\% \, Hc < Hd \leq 40\% \, Hc$ for dam heights equal and greater than 135 m.
- $40\% \, Hc < Hd \leq 60\% \, Hc$ for dam heights less than 135 m.

Where Hc is maximum depth of drainage curtain holes. In exceptional cases, drainage holes are made equal to the depth of maximum curtain grouting hole (Hc)

iii. Space of Drainage holes (S):

- S = 3 m for dams of all heights and most types of foundation rock.

iv. Number of drainage gallery (N):

- N = 1 for dam height (H) < 76 m
- $1 < N \leq 3$ for $H \geq 76$ m.

d. *Treatment against weak zones*

- It is backfilling concrete. The type of backfilling could be a dental concrete, a deep backfilling concrete, a concrete key/strut or a grid of shear keys. The choice and dimension of the type depend on the characteristics of the foundation and should be decided through the analysis of dam-foundation behavior. Generally it is much more than the USBR formula.

6.2. Inferences from The Analysis of Lakhwar Dam

a. The settlement under the conventional concrete gravity dam resting on existing foundation has been worked out approximately 210 mm. This maximum magnitude has occurred under the dam toe.

b. The above high value of settlement has been found primarily not due to the existence of shear zone near the dam toe. It could be inferred when analysis with backfilled concrete at various depths did not show reduction in settlement.

c. The above high settlement is found to be the effect of the existence of weak Slate rock downstream of the shear zone. It is inferred from the analysis done by replacing the modulus of deformation of the Slate equal to the Trap rock. In this condition, settlement was found about 42 mm only against 210 mm.

d. As long as treatment on the entire Slate rock to increase the modulus of deformation up to that of Trap rock is not feasible, Lakhwar dam site is not suitable for conventional concrete gravity type.

e. Normal vertical stresses in the dam body and of the foundation rock in case of existing foundation condition are found within the permissible values.

6.3. Scope for Further Study

i. Dam of types other than dam other than conventional concrete gravity type should be investigated at Lakhwar site, or change in site be considered.

ii. The suitability of concrete block dam in the left side in case of Lakhwar dam may be studied

iii. A detailed study to evaluate the middle and downstream drainage galleries in reducing uplift pressure should be taken up.

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APPENDIX

DATA INPUT INSTRUCTIONS FOR PROGRAM "fem2d"

The program is used for performing 2-D finite element analysis of gravity dams. The analysis can be performed for the following types of loads :

1. Gravity loads (dead weight plus uniform earthquake loads)
2. Water pressure (including uplift)
3. Point loads
4. Silt load

The program uses 8 noded isoparametric. The program accepts data from an input file whose name is interactively given by the user during execution. The name of the output file is also given interactively by the user during the execution.

The numbering of the nodes of an element is given in anticlockwise direction as shown in figure 1.

The mesh is made finer in the regions of interest and can be made coarser elsewhere. The aspect ratio (the highest side/lowest side of the element) should be ideally near to one. High aspect ratios lead to ill conditioned equations and should be avoided.

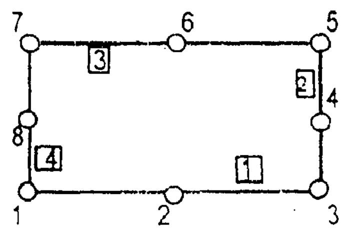

Fig. 1: Element Nodal Numbering

The program uses frontal solution technique and, therefore, the numbering of elements and not that of nodes is important. The problem size will depend upon, the maximum difference in the numbering of two elements having a common node.

The program has the facility of generation of nodal connection data and the numbers of nodes whose coordinates are to given. For this the elements and nodes have to be numbered from left to right and from bottom towards top.

The gravity load is specified as a ratio of acceleration due to gravity. For dead load the vertical acceleration is specified as -1 as the direction of positive load is upward.

Since the water pressure does not act on all the elements and the solution proceeds element wise, the program has to know whether the water pressure load is acting on the element. This in indicated by a non-zero value of index for the element. A non-zero value means that the element is subjected to water pressure loads. The case of silt pressure is also similar to that of water pressure.

The detailed instructions for data preparation follow :

1. **Data Type 1 : Title data**
   Heading
   Heading = title of the problem of upto 72 characters

2. **Data Type 2 : Control data**

NELZ, NLAST, NCZ, NBZ, NGAUSS, NMATS, NPP, NDLT, NLN, NTV, ISTRSG, NPLN, NSLN

NELZ	=	Total no. of element
NLAST	=	Total no. of nodes
NCZ	=	Total no. of coordinates given
NBZ	=	Total no. of boundary nodes
NGAUSS	=	Order of ganssian integration (2)
NMATS	=	Total no. of materials
NPP	=	1 for plane stress analysis (2 for plane strain)
NDLT	=	Total no. of load types
NLN	=	Total no. of point loads if any
NTV	=	Counter for thickness elements 0 – for same thickness for all elements 1 – for varying thickness
ISTRSG	=	A counter for taking print of stresses on gaussian point 0 if not required 1 if required
NPLN	=	Nos. of pressure loaded nodes
NSLN	=	Nos. of silt pressure loaded nodes

Note : The program prints the stresses at nodal points in any case

Data Type 3 : Nodes for coordinates Data

(NFIX (I), I = 1, 50)

NFIX (1)	=	First end node from where coordinates are to be given
NFIX (2)	=	2nd end node upto which the nodes are to be generated
NFIX (2*I-1)	=	First end node of ith pair from which coordinate nodes are to be started
NFIX (2*I)	=	2nd end node of ith pair upto which coordinate nodes are to be generated

Note : (a) If a node is to be given singly, it should be repeated twice to make its pair with the number itself.

(b) If the no. of pairs is less than 25, the remaining pairs should be given as zeros.

Data type 4. coordinate data

(a) Give x-coordinates for all NCZ nodes

(b) Give y-coordinates for all NCZ nodes

Data type 5 : Material property data

((PROP (I, J), J = 1, 3), I = 1, NMATS)

Give the following 3 properties of each of NMATS materials

(i) E (ii) ν (iii) γ

Data Type 6 : Fixity Data

1. Number of all NBZ boundary nodes
2. Fixity in x – direction for all NBZ nodes
3. Fixity in y-direction for all NBZ nodes

Note : Fixity 0 – free
1 – fixed

Data Type 7 : Material Data

(MATN(I), I = 1, NELZ)
Give material number for all NEIZ elements

Data Type 8 : Nodal connection Data

JADD, (LNODS(J, NEL) J = 1, LNODZ)

JADD	=	Number of additional element for which data can be generated from the given element nodal connection data
LNODS(J, NEL)	=	Node No. at the jth position of element no. NEL. This data is to be given for all LNODZ nodes of the element

Node : 1. LNODZ = 8 for the 2 – D element
2. If no nodal connection data is to be generated from the element no NEL, then JADD is given zero value.

The above data is given till all the elemental data has been generated/given.

Data Type 9 = LOAD Data

This data is to be given for all NDLT load types

Type 9A. **Load code**

LCODE = 1 → for point load
2 → for silt load
3 → for water load
5 → for gravity load

9B. The following data is given only for LCODE = 1
For all NLN nodes
NF(K), (PLOAD(k, J), T = 1, 2)
NF(K) = Node no of Kth point loaded node
PIOAD(k, 1) = Load in x-direction at Kth point loaded node
PIOAD (K,2) = Load in y-direction at kth point loaded node

9C. The following data is given only for LCODE = 5
ACLX, ACLY
ACLX = Acceleration in x-direction
ACLY = Acceleration in y-direction

Nodes : 1. ACLX & ACLY are positive along positive direction of x & y axis Respectively

2. For dead weight ACLY = -1

9D. The following data is given only for LCODE = 2

1. (KSIND (I), I = 1, NELZ)

KSIND (I) = index for silt load for the ith element

: 0 if the element is not subjected to silt load

: 1 if the element is subjected to silt load

2. For all the NSLN nodes the following data is given

(a) Node no. (b) silt depth on the node

9E. The following data is given only if LCODE = 3

1. (KWIND (I), I = 1, NELZ)

Give index for water load for all NELZ elements. The index is zero if the element is not subjected to water pressure load & non-zero if the element is subjected to water pressure load

2. For all NPLN nodes the following data is given

(a) Node No. (b) water head on the node

Notes : The data type 9B to 9E can be given in any order depending on the loading code LCODE being given

The sample data for the problem given in Fig. 2 is given in Appendix A. An extract of the result is given in Appendix 2.

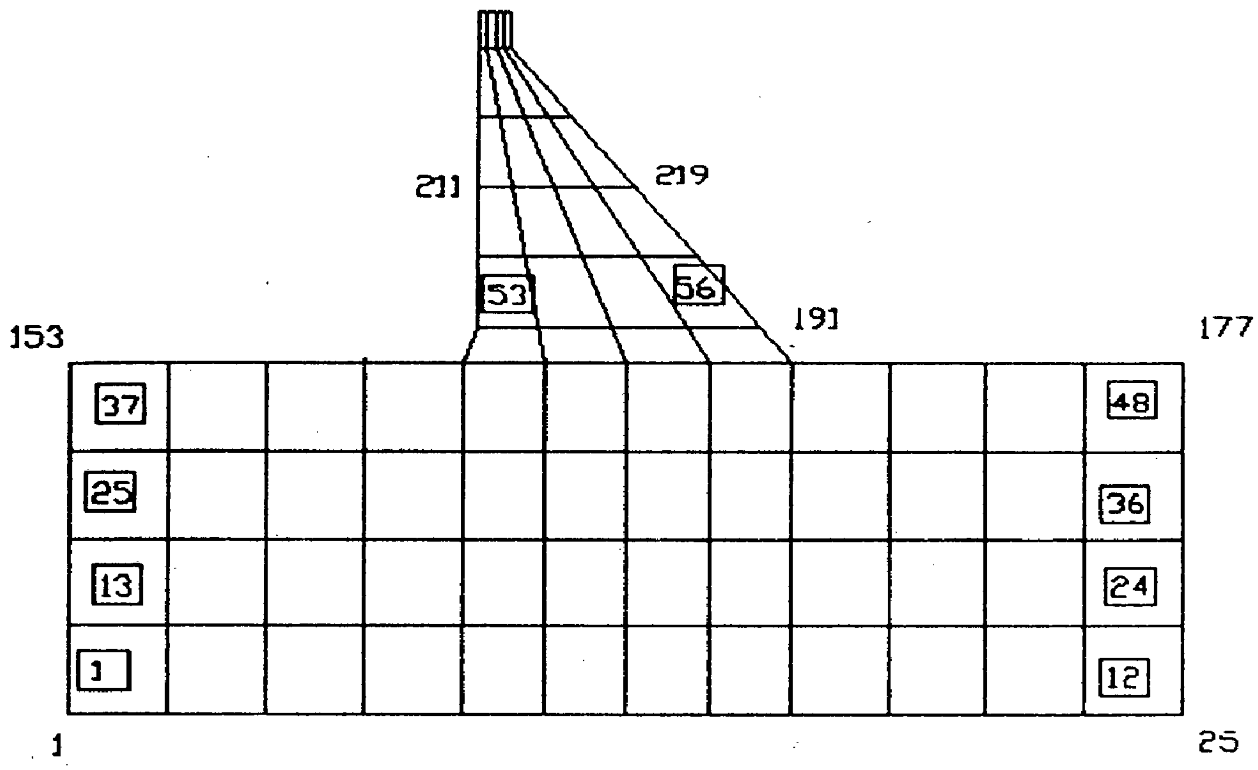

Fig.2: Finite Element Mesh