DESIGN OF PRESTRESSED CABLE ANCHORS FOR STABILIZATION OF HIMALAYAN SLOPES

A DISSERTATION

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

of

MASTER OF TECHNOLOGY

in

WATER RESOURCES DEVELOPMENT

By

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CANDIDATE'S DECLARATION

I hereby declare that the work, which is being presented in the dissertation entitled "**DESIGN OF PRESTRESSED CABLE ANCHORS FOR STABILIZATION OF HIMALAYAN SLOPES**" submitted, in partial fulfilment of the requirements for award of the degree of **MASTER OF TECHNOLOGY** in **WATER RESOURCES DEVELOPMENT** at **DEPARTMENT** of **WATER RESOURCES DEVELOPMENT AND MANAGEMENT,** of Indian Institute of Technology Roorkee, is an authentic record of my own work carried out for a period from July, 2005 to June, 2006 under the supervision of **Prof. Gopal Chauhan**, Professor, and **Dr. B. N. Asthana**, Ex-Visiting Professor, in the department of Water Resources Development and Management, 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.

Date: 27th June, 2006

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This is to certify that the above mentioned statement made by the candidate is correct to the best of our knowledge.

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SYNOPSIS

The assessment of stability of slopes particularly in the geologically poor environment for the safety of important and major projects needs careful attention. The adopted measures should be capable to resist the destabilizing forces and compatible with the life of adjoining structure. Stabilization methods viz. wire crates or constructing retaining walls/ breast walls with drainage arrangement, rock bolts etc. are adequate for small slopes but are insufficient to cope up large de-stabilizing forces of unstable slopes. Further, the excavations needed in adverse jointing pattern system for founding some important structures (e.g. dam, power intake, power house etc.) may require extensive stabilization measures. The prestressed anchors plays vital role to resist the de-stabilization forces of sufficiently large magnitude.

The details covering design principles, selection of particular type of anchor, load transference mechanism, adoption of bond stresses, corrosion protection measures and stressing of anchors alongwith three case studies have been discussed in this study.

The case studies cover slope stabilization measures in Himalayan Geology involving use of prestressed anchors for Hydro Projects viz. Nathpa Jhakri Hydro-Electric Project, Yamuna Hydel Scheme Stage II covering Ichari Dam and Maneri Bhalli Hydro Electric Project Stage-II. The overall objective of the case studies is to present analysis, design and construction aspects of different types of prestressed anchors using the available equipments and technical know how. In most of the Hydro-Electric Projects, their use in addition to other stabilization measures is becoming common. The case studies involve such geological features that three types of failures have been anticipated i.e. planar, wedge and slip failure. The design aspects of case studies, construction methodology, difficulties faced and further improvements described in this study for stabilization of slopes can be useful for other projects. The methodology developed for planer type of failure for the computation of anchorage forces involving both seismic and non-seismic cases for the Nathpa Jhakri case study is described which will be useful for design of similar works in future.

CHAPTER -1

INTRODUCTION

1.1 GENERAL

The stability of slopes and rock faces is normally lost due to natural phenomena such as penetration of water, icing and thawing or erosion. But in certain cases, the stability of slopes is affected due to modifications in the ground slopes or the loading conditions by human intervention. Therefore, slopes face equilibrium disturbances due to natural causes and human intervention. Mainly equilibrium disturbances in the slopes are associated with the construction of roads, major structures or extensions to existing systems. Major structure in Himalayan terrain covers construction of Hydro-Power Projects to harness the available potential to cope up with steep demand of electricity in the country. The projects on tough sites having adverse geology are under construction or to be constructed in near future, as most of the easy sites stand constructed. Therefore, the stabilization of slopes for the construction of new Hydro Projects in Himalayas is a challenge for engineers.

The hilly terrain in Himalayan Region falls in the category of youngest mountains and needs protection measures, most of the time, whenever some construction is performed for the desired cut in the slopes. These young mountains having fragile geology are still on the verge of their formation and are not stable as compared with southern mountains of India. The construction activities in hilly terrain mostly require cutting or filling in the slopes for the construction of building, road or any other civil structure. After any cutting or filling, slopes shall be examined for stability and protection measures, if necessary, are implemented. The excavated slopes are frequently of considerable extent and in certain circumstances can even affect the stability of adjacent zones and particularly slopes and rock faces situated above them.

In hilly areas, the life and safety of structure not only depends upon its own safe design & construction, but also on stable slopes in the vicinity of structure. Normally in hilly terrain the protection of slopes is generally achieved by placing wire crates or constructing retaining walls/ breast walls including proper drainage arrangement, rock bolts, anchors depending upon magnitude and nature of structure. Whereas for important and major project, the slope protection arrangement as indicated earlier may not serve the purpose completely to counteract the major de-stabilization forces due to adverse geological features. Cutting at a gentle slope and removal of overburden may be an alternative to avoid means like anchors etc. But in certain situations where it is not possible to cut the slopes due to presence of some important road or civil structures, then anchorage methods become necessary to deal the stabilization problem.

In order to neutralize or minimize the de-stabilization forces of sufficiently large magnitude, the recent developments in ground anchor technology and the related techniques of cable bolting and rock bolting are being practiced in adverse geological environment.

Nowadays, prestressed anchors for stabilization of slopes are being used to facilitate the different construction activities in major projects on adverse geological sites.

1.2 SLOPE FAILURES AROUND HYDRO PROJECTS IN INDIA

For the generation of electricity, head (elevation difference) is very important. Therefore, most of the hydro power projects are located in hilly areas and there is a history of slope-failures especially where high relief topography coupled with other adverse factors such as complexly folded, faulted, and jointed rock formations are present.

The occurrence of these incidences is further enhanced by the high rainfall received by such areas. Also there are evidences that many slope failures in Himalayas have occurred without man's intervention as a natural process. The following Hydro-Electric projects have been affected by mass movement processes (Bhatt and Vipin, 1992).

(i) Loktak Hydel Project slide, Manipur.
(ii) Chineni Hydel Project slides. Distt. Udhampur, J&K.
(iii) Pair and Tanger slides around proposed reservoir of Sawalkot Project, J&K.
(iv) Lower Jhelam Hydro project slide, Distt. Baramullah, J&K.
(v) Baira Siul Hydel Project slide, (H.P.)
(vi) Pandoh Dam slide (H.P.)
(vii) Giri Power House slide, Distt. Sirmaur (H.P.)
(viii) Pong Dam intake slide (H.P.)
(ix) Rishikesh–Chilla Hydel Project slide (U.P.)

Apart from above, major slide to the tune of about 10 lacs cum occurred just about 100m upstream of dam of Nathpa Jhakri Hydro Electric Project (H.P.) just prior to construction of main dam.

From the above failures, it becomes necessary to develop vision for protection of slopes particularly in important projects, which in the eventuality of failure at operation and maintenance stage can cause huge financial and generation losses to country. Therefore, thorough geological and topographical site investigations alongwith taking account of past slides, if any, in or around the project shall be done in order to avoid any catastrophic or future repercussions. Expenditure spent on slope protections during or prior to construction of main project is nominal as compared with total cost of project. To tackle destabilising forces of higher magnitude and obtain higher factor of safety for slopes of such important projects, the prestressed anchors having long term performance compatible with main structure's life are in vogue for slope protection.

1.3 SCOPE OF THE STUDY AND OBJECTIVE

In this study the design and construction procedure of prestressed anchors for stabilization of slopes under different geological conditions have been reviewed. Different types of prestressed anchors suitable with site conditions have been described. The important design parameters of anchors related with strength, economics and performance have been discussed. Special emphasises have been laid upon bond characteristics, corrosion protection requirements and stressing of anchors. Evaluation of important parameters viz. fixed and free length of anchors has been discussed.

Three case studies covering slope stabilization measures in Himalayan Geology involving use of prestressed anchors of different categories for three Hydro Projects viz. Nathpa Jhakri Hydro-Electric Project, Yamuna Hydel Scheme Stage II covering Ichari Dam and Maneri Bhalli Hydro Electric Project Stage-II covering Dharasu Power House have been described.

(i) For first case study, Design of Prestressed Cable Anchors for the Stabilization of slopes of 1500 MW Nathpa Jhakri Hydro-Electric Project (NJHEP) in the vicinity of Nathpa Dam, Power Intake and Plunge Pool have been discussed.

(ii) In the second case study, details of Prestressed anchors on Left Training Wall of Ichari Dam on river Tons in Uttaranchal State have been discussed. Also comparative Study on Anchors i.e. Prestressed anchors v/s Concrete anchors for left bank slopes of Ichari Dam area has been incorporated.

(iii) The third case study covers the details of prestressed anchors for the treatment of upstream slopes of Dharasu Power House in unit nos. 1 to 4.

In the case studies, overall objective is to present analysis, design and construction aspects of different types of prestressed anchors used at three projects using the available equipments and technical know how alongwith scope in future considering improvements.

1.4 FINDINGS OF THE STUDY

The detailed design principles including computational methodology, construction aspects, difficulties faced and further improvements described in this study can be useful for similar works.

For the protection of important and major structures where other general methods for slope protection measures are inadequate, the provision of prestressed anchors is best suited particularly for safeguarding against the destabilising forces of large magnitude in the poor geological environment. The use of strand as tendon in anchors is better option as compared with solid bar or wire type tendons, particularly for high capacity anchors.

1.5 ORGANISATION OF THE DISSERTATION

This dissertation is organised into the chapters as follows:

Chapter 1: Introduction to the problem and scope of the study.

Chapter 2: Description of literature review.

Chapter 3: Design and Construction considerations of prestressed anchors.

Chapter 4: Description of different case studies.

Chapter 5: Conclusions and suggestions for further study

References

Appendices

CHAPTER - 2

LITERATURE REVIEW

2.1 GENERAL

Anchoring systems are extensively used in civil and mining applications to stabilise rock slopes and underground openings and to resist uplift and overturning forces in foundations and retaining structures. The use of prestressed anchors has been a common practice in civil and mining engineering. Prestressed anchors function as temporary or permanent structural members to ensure the stability of various structural systems such as slopes, retaining walls, bridge abutments, tunnels, underground excavations, and reinforced concrete foundations. They are also used for the strengthening and rehabilitation of existing structures such as concrete dams and bridges.

Now days apart from extensive use of prestressed anchors, also cable bolting and rock bolting are used for the slope stabilization. Overall all these techniques enable design engineers to address stability problems over a range of scales and in a range of geotechnical environments. The techniques have similar aims but have developed into separate disciplines with unique attributes. The differences between the above techniques are predominantly associated with scale and the standards of design and installation. Prestressed anchors tend to be longer with the highest capacity, rock bolts tend to be shorter with the lowest capacities and cable bolts have evolved to address stability problems that lie between the two. But now a days extensive use of prestressed anchors is common for various purposes. In the literature review, different stages of anchor developments are covered.

2.2 DEVELOPMENTS IN THE ANCHOR TECHNOLOGY

The first application of a prestressed anchor dates back to 1934 when the late Andre Coyne, a French engineer, used prestressed anchors to stabilize the Cheurfas Dam in Algeria (EM 1110-1-2907, Feb., 1980). The high capacity vertical holding down anchors of 1100 tons were used in the Cheurfas Dam during the year 1934-35. This has marked the advent of modern prestressed anchor technology. With the improvement in technology, the prestressed anchors comprising with a debonded free length and cement grouted fixed length in rock increased the stability of the structures. The utilization of combination of free and fixed length founded in soil commenced in Europe (Germany and France) in the late 1950s, albeit passive underreamed tension piles were being utilized in Texas and India to resist ground heave at that time (Barley & Windsor,2000). Thereafter, Germany initiated more research on soil anchor technology followed by some construction activity i.e. inability to recover a temporary drill casing while constructing an anchor. UK anchor technology in the 1960's was influenced by the systems developed in both Germany (end of casing pressure grouting) and France (post-grouting). But still UK took the lead in successful development of the multi underreaming systems associated with suitable anchor size bore diameter.

High capacity dam anchors were generally constructed during early 1960s utilizing simple two stage grouting of the fixed and free length incorporating simple methods of corrosion protection. Further gradual advancement occurred in rock anchor technology with the utilization of higher anchor capacities using nominal borehole size, to accommodate an increasing number of high capacity prestressing strands. Australia probably lead the way with the most extensive use of long ultra high capacity dam anchors (1200 tons working load, up to 120m long) complete with modern corrosion protection systems.

In the city Melbourne, intense application of anchor technology has been used with some 7000 anchors, each with a 120 year life span requirement, to stabilize the tunnel beneath the River Yarra. Generally, the working capacities of soil anchors increased little through the 1970s and 1980s (Littlejohn 1970, Ostermayer 1974, Barley 1987);

- In gravels working loads up to 80 tons using grout "injection" techniques
- In sands working loads up to 500 tons using grout "compaction" techniques
- In stiff to very stiff clays working loads up to 600 tons by use of post grouting or under reaming techniques.

Construction techniques improved significantly particularly due to the gross advancement in performance of drilling rigs and equipment; increased power, higher torque and the introduction of top drive rotary percussion. The confidence in the use of anchors increased with the increase in factor of safety. Also after 1980s a lot of research work investigating the performance of anchors in various soils and rocks and the associated load transfer mechanism, has been carried out at numerous locations. With detailed investigations and analysis on the distribution of load along the length of the anchor borehole in the late 1980s allowed evolution and advancement of a new anchor concept. This "multiple" anchor system has lead to the attainment of 300 to 400 tons capacity in anchors installed in soils and weak rock and the frequent installation of soil anchors with working loads in the 80 to 200 tons range (Barley and Windsor, 2000).

2.3 ANCHOR TENDONS

2.3.1 Solid Bar Tendons

Steel bars were frequently used during initial development of soil anchor technology in the 1950-1960s. These bars were generally smooth, but with threaded ends to enable transfer of load both at the anchor head, and within the fixed length. Thereafter use of longer bars to cope up higher capacity working load started with the extension of individual bars (up to 12m long) by use of couplers. The bar systems were readily available because of development of tie rod and other engineering works requiring tensile members. In order to prevent tendon bonding in the "free length" it was required to place plastic sleeves around the free length bars and further to reduce friction bond the smearing of grease was carried out on the bar surface prior to sleeving. During the initial stages the debonding of the coupler from the free length grout column was done by densotape wrapping but later on the placement of a compressible packer above the coupler to allow its free movement considered necessary. The use of smooth steel bars is not normally conducive to effective load transfer from the tendon to the cement grout since bond stresses at the interface rarely exceed an average of one MPa. Further, enhancement of bond in a "smooth" bar was generally achieved by the increase in the length of the threads turn on the bar in the fixed length. Therefore concept of fully threaded bars of various fine and coarse thread patterns for use in anchors involved. A very broad selection of size and capacity of fully threaded bars now exists, up to a maximum capacity of 430 tons (Barley and Windsor, 2000).

2.3.2 Prestressing Strand Tendons

After the use of anchor systems being used as steel bars, the difficulties of handling and coupling of long steel bars was realized and the alternative use of prestressing strands was introduced. The presence of V grooves between the helically wound peripheral wires provided surface deformation to enhance the bond between the strand and the grout. This achieved average bond stress in the 1 to 2 MPa range when the spacing among strands were appropriately greater than 5mm

(Bruce 1976). Also it was found that the capacity of an upstand deformation is greater than that of an indent (Barley, 1997A) and the surface indent alone was not adequate to ensure that the full capacity of each strand could be mobilized. Further it was experimented that unravelling and rewinding of the strand wires grossly enhanced bond (Fig. 2.1).

Fig. 2.1: Use of strand "bushes" to enhance strand & grout Bond

This operation is known as "caging" or "bushing. In the early days, the provision of long (1m) "bushes" at staggered locations in adjacent strands served a multiple purpose of partially spacing the strands themselves and keeping the closed strand body away from the borehole wall (Fig. 2.1).

However, later it was realized that this method could not guarantee the elimination of strand smearing or guarantee grout cover. Then introduction of shorter size controlled cages along with plastic spacer/centralisers was required to conform to code requirements and produce bond capacity in excess of 3 Mpa.

Later on an alternative mode of enhancement of strand to grout bond was developed in the form of "tendon noding". In this arrangement the peripheral strands being spaced between 15 and 20mm apart at one location and tightly banded together about 1.5m above and below the spacer. This is known as multiple strand arrangement comprising with deformed outer profile and this arrangement ensures an interlocking effect, all of which enhances grout to tendon bond.

All these load transfer systems are well proven and well established although efficiency of tendon bond may fall with increase in number of strands and increase in steel density within the bore. All internal strands contained within an outer periphery of strands must transfer their load capacity across that outer periphery. Consequently, the spacing between the peripheral strands should increase as the number of strands contained within increases. Therefore, size of the borehole shall be suitably established.

With regard to free length debonding, this was initially affected by two stage grouting, i.e. grouting the fixed length then stressing the anchor, followed by free length grouting. However, such precise control of the grout level in anchoring is difficult to achieve and this method was replaced by tape-wrapping of the group of strands. By 1970, the provision was developed to get pregreased and plastic coated strands. This, however, necessitated the development of an efficient system for removal of the plastic coating and the degreasing of the strand wires over the bond length.

The photograph at Fig.2.2 shows Modern strand pusher machine allowing effective greasing and sheathing of designed debonded lengths.

Fig. 2.2: Modern strand pusher machine

Above mentioned developments in the system evolved better choice to make use of prestressing strand as the anchor tendon than the use of bars.

2.4 TENSILE FAILURE OF ROCKS

In majority of civil engineering structures forces applied to the ground are generally compressive in nature with slight eccentricities. The design principles for such foundations are clear and well established. There are, however, a large number of more specialized foundations where the forces in general are not compressive but tensile. These types of foundations are much more difficult to deal with because all ground materials are very weak in tension.

While dealing with slopes stabilization, the main interest is to keep a stable rock mass stable as well as stabilizing an unstable rock. The creation of slope cuts for supporting engineering structures as well as creating underground openings for some engineering purposes cause stress changes along joints and fissures with a general opening of joints and a concentration of stress in certain regions.

There are large nos. of stabilization methods available, but adopting anchoring and rock bolting for slope stability, stress field gets modified. Anchoring & rock bolting is generally done in association with drainage to reduce the pore water pressures. In case of stabilization by grouting, the quality of rock mass is improved particularly along the joints & fissures.

Anchors are structural members which transmit tensile forces into the rock mass. They are inserted into boreholes and bonded to the rock by grout or other chemicals. Their action is twofold. Firstly, on tensioning an anchor, the stress field gets modified in the vicinity of the anchor. Secondly, anchor also acts as a preventative measure against the further disintegration of the rock while holding a block of rock in its original position. One of the applications of anchors is to stabilize structures on the rock surface by transmission of the external force to the rock in depth. For the stabilization of rock slopes, it is essential to have a full understanding of the rock mass, particularly orientation, spacing and size of joints and fissures alongwith ground water regime within the rock system. It is not only sufficient to examine the mechanical conditions of the rock mass in its natural state but also changes which are caused during and after construction.

The general arrangement showing use of ground anchors for rock slope stabilization has been given in Fig.2.3.

2.5 PERMISSIBLE STRESS LEVELS

Permissible stresses may be quoted in terms of the specified characteristics strength, f_{pu}. This is a guaranteed limit below which less than 5% of the results may fall. Other terms such as "elastic limit", 0.1% of proof stress can be quoted to define stresses. According to Littlejohn and Bruce (1975), a 0.1% proof is equivalent to 83.5% f_{pu}. The stress-strain curve of high-tensile steel differs with curve of mild steel as it does not show a definite yield point. The proof stress concept is useful to quantify the curvature of the stress-strain curve. This is defined as the stress at which the applied load produces a permanent elongation equal to a specified percentage of the gauge length, normally 0.2%, Fig. 2.4 shows how this value is increased for various forms of treatment and it can be noted that the elastic limit is increased from as-drawn wire to low-relaxation wire. The points B1, B2, B3 are the value of the 0.2% proof stress and are 90%, 85% and 75% of the specified characteristics strength respectively (Allen, 1978).

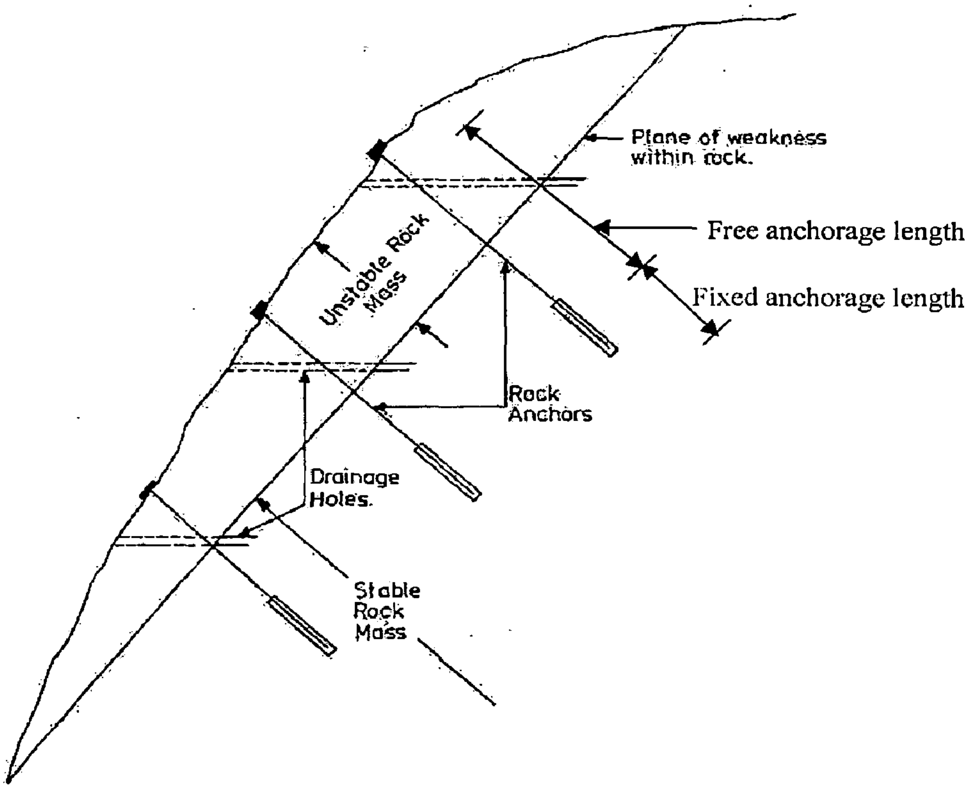


Fig. 2.3: Use of ground anchors for rock slope stabilization

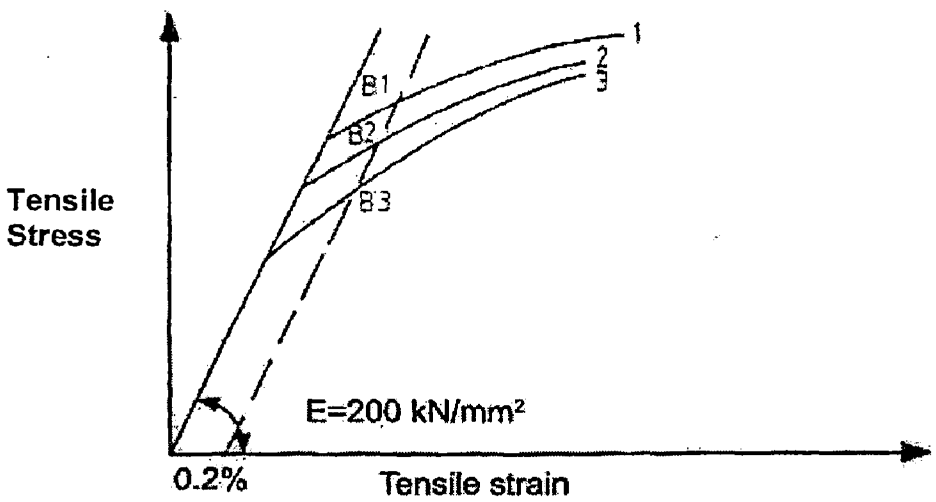


Fig. 2.4: Typical stress-strain curves for 7 mm diameter plain wire
1. Low relaxation, 2. Normal relaxation, 3. As drawn

In order to evaluate the load test results due considerations has to be given for the values of modulus of elasticity (E). These values are available from the steel manufacturers.

Understanding of two additional factors i.e. creeps and relaxation is important for anchor tendon stretch under long-term loading. It is well known that in metals under constant load, even with the stress level well below the elastic limit, slow plastic deformations, known as creep, can occur. Creep can take place over a wide range of temperatures. From the relaxation characteristics of the steel, loss of prestress values can be computed for anchor tendon. The stress relaxation of steel is a very complex subject and depends on the treatment of the steel during manufacture, on the temperature and on time (Hanna, 1982).

In general, working stresses are controlled by the working life of the anchor (short or long term use), the level of corrosion protection provided, and the test overload to be applied to the anchor during stressing. The some of the anchors used earlier didn't have a sufficient high factor of safety, as observed. In general, the load values shall not exceed 62.5% and 50% of the characteristic strength of the tendon for temporary and permanent anchors, respectively (Littlejohn and Bruce, 1975).

2.6 WATER TESTING OF ANCHOR HOLES

In the fixed anchor length loss of grout from around the tendon can affect the corrosion resistance of steel alongwith load transfer efficiencies in the system. Therefore anchorage length (fixed length) for all permanent anchors has to be tested for water proofing to minimize corrosion effect. In order to ensure that cement is lost in a fissure, the fissure dimension must be greater than 160 microns. Therefore one fissure of this dimension and under an excess head of one atmosphere will result in a flow of 3.2 litres/min (Littlejohn, 1975), whereas a fissure width of 100 microns gives a flow of 0.6 litres/min/atmosphere. This establishes the variable nature of rocks and therefore it becomes necessary to interpret the flow rates. The tests should be carried out over the fixed anchor length only with the use of packers. Now situation can arise with two rock types giving the same loss of water, yet one has a large fissure which can permit cement grout loss while the other is uniformly porous and will not permit grout loss. In this regard useful guidance has been given by Littlejohn (1975) based on a worldwide survey.

(i) An anchor borehole requires water proofing when the flow exceeds 3 litres/min/atmosphere, while dealing with a single fissure. In case a number of fissures is known to exist, larger limiting flow rates are possible and, in order to make a proper judgement, it is necessary to assess the structures of the rock mass by examination of the borehole sides using a camera or closed circuit T.V. Fig. 2.5 shows the theoretical relationship between flow rate and single fissure opening, based on the theory of flow in fissured formations, Baker (1955). One luegon is defined as one litre per minute per meter length of borehole under a pressure head of 10 atmospheres.

(ii) In order to establish the excess head causing flow care shall be taken to measure the position of the ground water table.

(iii) It shall be ensured that applied pressures are not too high which may lead to opening up of closed fissures. In turn this will suggest that waterproofing of the anchor hole is required. During the execution of such tests major problem is fitting of mechanical packers required for sealing the borehole.

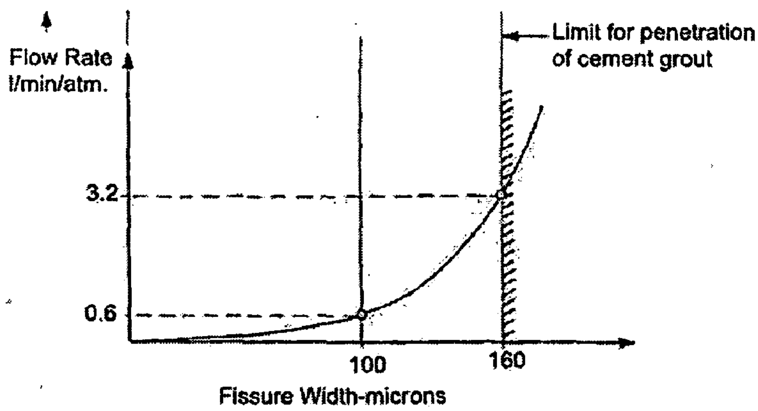


Fig. 2.5: Theoretical relationship between flow rate and single fissure opening

High grout pressures shall not be applied and water pressures of 300 kN/sqm are often used in a rocky strata. The testing has been restricted to the fixed anchor length or the fixed anchor plus part of the free anchor. Sometimes the corrosion protection requirement depends upon the integrity of the grout over the free anchor length also, and then water proofing shall also be done in the free length zone also.

As per IS:10270, if the water loss is less than 3 luegons, the hole is considered to be satisfactory and where salinity in the ground is high, minimum acceptable permeability value is taken as 1 luegon.

The water testing of holes is needed to ensure limited grout loss for proper anchorage of the tendon. Also by limiting the loss of anchor grout, the corrosion protection can be ensured. Based on results of water tests, the consistency of consolidation grout can be monitored. In case high volume of leakage is observed in the hole, a thick consolidation grout shall be used i.e. maximum of five gallons of water per sack of cement. Otherwise, in the event of low volume of leakage, a very lean consolidation grout shall be used i.e. eight gallons of water per sack of cement. The redrilling of consolidation grouted hole is generally done after 24 hours, i.e. when grout stands set up (EM 1110-1-2907, Feb., 1980).

The grouting performed for anchors shall be differentiated with the grouting requirement for the reduction of water leakage below dam. Generally, for fissured rocks beneath dam, flow rates as low as 1 luegon is specified. Such low values of leakage shall not be specified in anchor works to prevent cement grout leakage.

CHAPTER - 3

DESIGN & CONSTRUCTION CONSIDERATIONS–PRESTRESSED ANCHORS

3.1 GENERAL

The slopes are most common sub-aerial landforms on the surface of the earth. Slopes either occur naturally or are engineered by humans. The slope stability problems have been faced throughout history when nature or human has disrupted the delicate balance of natural soil slopes. Also with the increasing demand of cuts or fills of slopes on construction projects has increased the need to understand analytical methods, investigative tools, and stabilization methods to tackle slope stability problems. Further with the advancement of slope stability analysis in geotechnical engineering has followed the development in soil and rock mechanics as a whole. Slope stabilization methods involve special construction techniques that need to be worked and modelled in realistic ways.

An understanding of geology, geo hydrology, and soil/rock properties is central to applying slope principles properly. Analysis should take considerations of accurately representing site subsurface conditions, ground behaviour, and applied loads. Good judgement regarding acceptable risk or safety factors must be made to access the results of analysis.

The slope failure has been recognised as one of several forms of natural disasters. The disaster can cause loss of life / property and damage to works. A large number of methods are available for stabilization of potential unstable slope as a part of preventive measure. Some are listed below:

* Vegetation and plantation
* Netting by jute and coir
* Drainage improvement
* Retaining wall construction
* Piling and ground improvement
* Anchoring

Depending on the site conditions a combination of a few above measures is generally adopted. While dealing with the case studies (Chapter 4) mainly stabilization of Himalayan slopes in the vicinity of Hydro Power components it has been seen that generally combinations of measures are adopted alongwith the prestressed anchors. Hence in this chapter the details of anchors and their construction methods are discussed.

3.2 DESIGN CONSIDERATIONS

3.2.1 Ground Anchors

The primary function of ground anchors is to provide a concentrated force to the ground. They are extensively used in mining, to stabilize rock slopes and underground openings and to resist uplift & overturning forces in foundations and retaining structures. One of the most common forms of anchorages utilizes grouted cable or rock bolts. Such anchors derive their capacity from the frictional resistance developed at the ground/grout & tendon/grout interfaces. It is at these interfaces that the axial load of the anchor tendon is transferred to the ground mass.

An anchor is a sub-structural member, which transmits a tensile force from the main structure to the surrounding ground whether it is placed in soil, rock or a marine environment (Hanna, 1982). The tensile force introduced is resisted by shear strength of the surrounding ground. Also it is desired to fasten the anchor to competent ground well away from the structure. Ground anchors are divided in two broad categories viz. rock and soil anchors. Soil anchors are generally used in clay, sand or other granular soils. This study is limited to rock anchors because stabilization of Himalayan slopes which comprise of different type of rocks is the subject of this study.

3.2.1.1 Rock Anchors

A prestressed rock anchor is a high strength steel tendon, fitted with a stressing anchorage at one end and a means permitting force transfer to the grout and rock on the other end. The rock anchor tendon is inserted into a prepared hole of suitable length and diameter, fixed to the rock and prestressed to a specified force. The anchors shall be installed at the required inclination and to the required depth to resist the applied load in an efficient manner so that the tendon material is stressed to permissible levels and the ground in which it is embedded is also realistically stressed, (Fig.3.1). The tensile force introduced through anchor system should maintain necessary equilibrium between the anchor, the structure to which it is attached and the ground in which the anchor is embedded so that the movements of the structure and the surrounding ground are kept to acceptable levels. The tendon is usually a high strength steel member (bar, wire or strand) surrounded by cement grout or other fixing agent. The tendon has to be protected against corrosion effects, otherwise the basic purpose of transferring the tensile force to ground will get defeated.

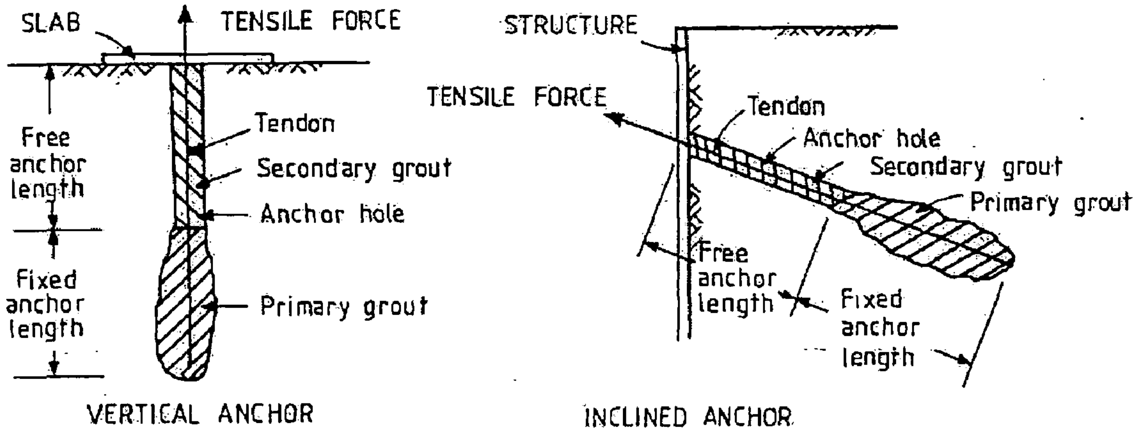


Fig. 3.1: Ground anchor (shown in vertical and inclined direction) (Hanna, 1982)

The basic components of prestressed rock anchor tendons are the following:

i) Prestressing member which may be one of these (a) a single or a plurality of wires (b) strands (c) bars.

The total length of the prestressing tendon is composed of two parts:

Fixed anchor length (also known as Bond Length):-

The portion of the tendon where tensile force from anchor is transmitted into the ground/surrounding rock through the bond between prestressing steel & grout and the grout & the surrounding rock/soil. It is also called the bonded length.

Free anchor length (also known as Stressing Length):-

The portion of tendon which can be elastically elongated and during elongation tendon is not in contact with surrounding ground. This is also called the unbonded length (or stressing length). The free anchor length is upper portion from structure to top of the fixed anchor length over which no tensile force is transmitted to the surrounding ground. This is achieved by placing frictionless sleeves around the tendon. These sleeves also act as corrosion protection in the free anchor length. This length is that part of the tendon which is free to elongate during stressing. To guarantee a free elongation this length should not be smaller than 5m (IS: 10270).

ii) A stressing anchorage is a device which permits the stressing and anchoring of the prestressing steel under load.

iii) A fixed anchor is at the opposite end of the tendon than the stressing anchor and is a mechanism which permits the transfer of the induced force to the surrounding grout.

iv) Grout & vent pipes and miscellaneous appurtenances required for injecting the anchor grout or corrosion protective filler.

The typical details of Prestressed Rock Anchor have been shown in Fig. 3.2.

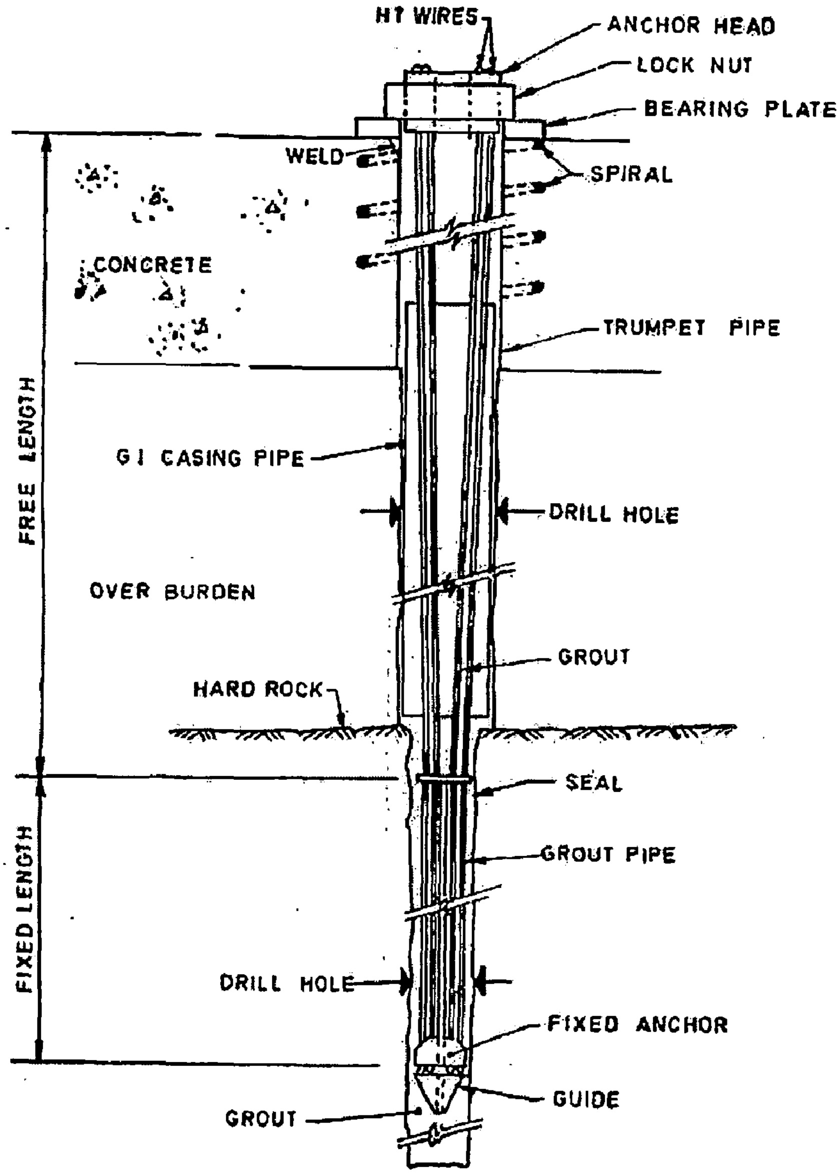


Fig. 3.2: Prestressed Rock Anchor showing typical details (IS: 10270)

3.2.2 Some definitions related with anchors

Permanent ground anchors:- Permanent ground anchors have to guarantee their function during the lifetime of the structures to be anchored. Therefore, they have to fulfil the specific project requirements with respect to failure and corrosion protection and thus require special design and supervision measures.

Temporary ground anchors:- Prestressed anchors which have to fulfil their function only for a limited time, in general, up to 2-3 years are considered temporary anchors. Typical applications are retaining of walls during excavation, holdbacks of temporary stay structures etc. Temporary anchors in corrosive environments may require additional corrosion protection, similar to permanent anchors.

Test anchors:-Test anchors are specially design anchors subject to extensive tests in order to obtain, either comprehensive information on anchor capacity and geotechnical conditions, or to prove the quality and adequacy of design, materials and construction.

Downward Sloped Anchor:- Any prestressed anchor which is placed at a slope greater than 5° below the horizontal.

Upward Sloped Anchor:- Any prestressed anchor which is placed at a slope greater than 5° above the horizontal.

Horizontal Anchor:- Any prestressed anchor which is placed at a slope between ±5° with the horizontal.

Anchor Grout (Also known as primary injection):- Portland cement grout that is injected into the anchor hole to provide anchorage at the non-stressing end of the tendon. In case of a sheathed anchor, also included in the grout between the sheath and the anchor hole. Resins are also used as anchor gout.

Corrosion Protective Filler Injection (Also known as secondary injection):- Material that is injected into the anchor hole to cover the stressing length of the prestressed anchor, providing corrosion protection to the high strength steel. This material may be grout or other suitable materials.

Consolidation Grout:- It is Portland cement grout that is injected into the hole prior to inserting the tendon to waterproof or otherwise improve the rock surrounding the hole.

Inserting:- The physical placement of the anchor tendon in the prepared hole.

Lift-Off Check:- Checking the force in the prestressed anchor at any specified time with the use of a hydraulic jack.

Proof Load:- Initial prestressing per anchor, representing the proof loading.

Transfer (lock-off) Load:- Prestressing force per anchor after the proof loading has been completed and immediately after the force has been transferred from the jack to the anchorage.

Design Load:- Prestressing force per anchor after allowance for time dependent losses.

Coating:- Material used to protect against corrosion and/or lubricate the prestressing steel.

Sheathing:- Enclosure around the prestressing steel to avoid temporary or permanent bond between the prestressing steel and the surrounding grout.

Coupling:- The means by which the prestressing force may be transmitted from one partial length prestressing tendon to another.

Sheathed Anchor:- An anchor in which the stressing length of the high strength steel is encased in a grout-tight sheath. The annulus between the sheath and the periphery of the drilled hole may be grouted together with the anchor grout.

Un-sheathed Anchor:- An anchor in which the stressing length of the high strength steel is not encased in a sheathing.

3.2.3 Load Transference in Anchors

Under this topic, the mechanics of load transference from anchor to ground has been dealt along with existence of certain uncertainties and how they can be overcome.

One or more of the following modes can lead to failure of an anchor:

(i) by failure of the grout/tendon bond;

(ii) by failure of the ground/grout bond;

(iii) by failure within the ground mass;

(iv) by failure of the tendon steel or a component;

(v) by crushing or bursting of the grout column surrounding the tendon;

(vi) by failure of a cluster of anchors.

While designing the anchor it is necessary to examine each of above factors in association with others, so that specified design load can be carried by anchor system.

3.2.3.1 Load transference from Tendon to Grout; Mechanics thereof

An idealized mechanism for load transference from tendon to grout is as below.

The tendon steel on the microscopic scale is rough; hence the grout column around the steel fills these rugosites. Initially, on loading of the tendon relative to the grout, bond (adhesion) is mobilized. Thereafter, further at finite relative displacements, the bonding is effectively destroyed and resistance is developed by friction between the tendon and the confining grout. This frictional resistance, which is augmented by dilatancy of the grout, may be increased by irregularities of the tendon steel, which cause part of the shear strength of the grout to be developed. The plot of 'resistance' v/s 'displacement of tendon relative to grout' of an element of the tendon/grout interface is shown in Fig. 3.3. The magnitude of movement of the tendon relative to the grout will vary along its length, as tendons are relatively long and elastic members. In case of small movements, the resistance is developed mainly in adhesion and, with tension anchors under working load conditions, this will occur near to the lower end of the fixed anchor length. Near the upper end of the fixed anchor length, mostly relative movements are greater due to the elastic stretch of the tendon, then friction and interlocking effects will also apply. In case frictional type of resistance is developed, then its value depends on the degree of confinement provided by the grout column. Due to important role of grout column, attention shall be given to yield of the borehole walls and crushing of the grout material, especially with high load capacity anchors. In the situation of crushing and yield of the grout, much greater displacements of the tendon relative to the grout occur.

But in actual practice above idealized behaviour does not apply, as shown by the extensive work carried out into bond stresses along concrete reinforcement (Tepfers, 1973; Lutz and Gergeley, 1967). These studies have established that in case of plain steel tendons, bond mainly depends upon adhesion prior to slip and upon friction after slip occurs. But in case of deformed steel tendons, bond depends primarily upon mechanical action as shown in Fig. 3.4. However, when the tendon steel is deformed or the arrangement of the tendon is such that the plain steel behaves in a similar manner to a deformed bar, slip of the tendon relative to the grout may occur by splitting of the grout column or crushing of the grout. Hence control over the thickness of grout cover is necessary for a particular load. During the pulling of tendon, the reaction forces are taken by shear stresses within the grout. At the locations where principal tensile stress exceeds the tensile strength of the grout, then cracks will develop. The occurrence of the cracks due to above phenomenon is shown at Fig. 3.5.

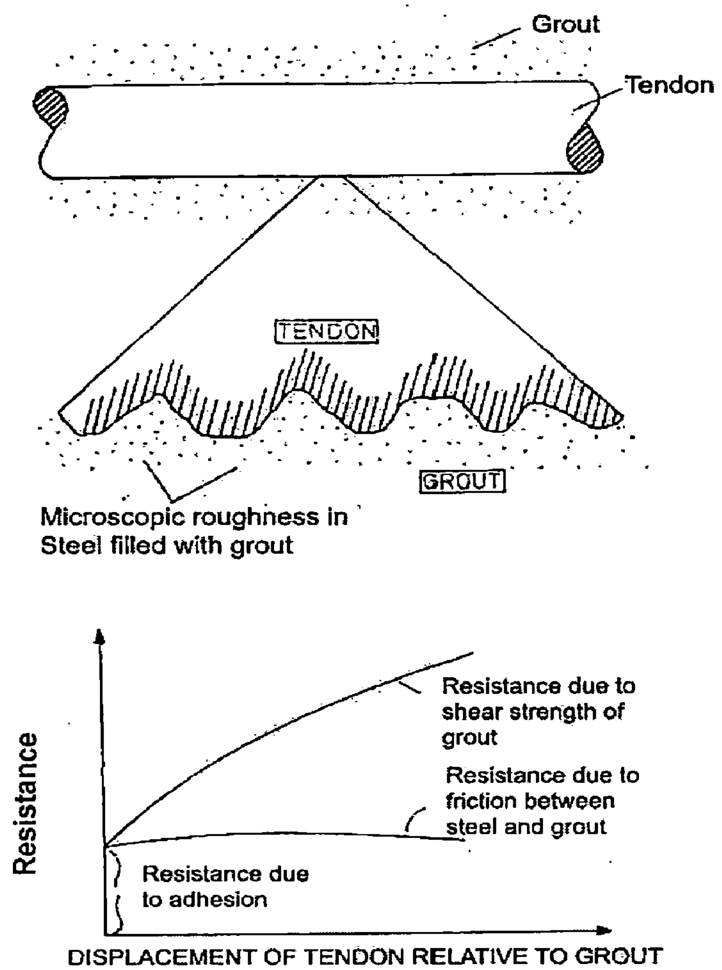


Fig. 3.3: Idealized behaviour of tendon/grout interface (Hanna, 1982)

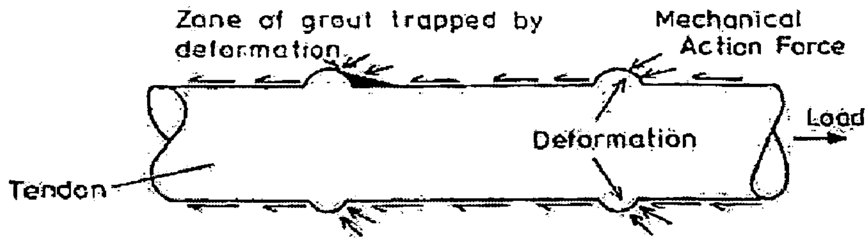


Fig. 3.4: Interaction between a deformed bar and grout

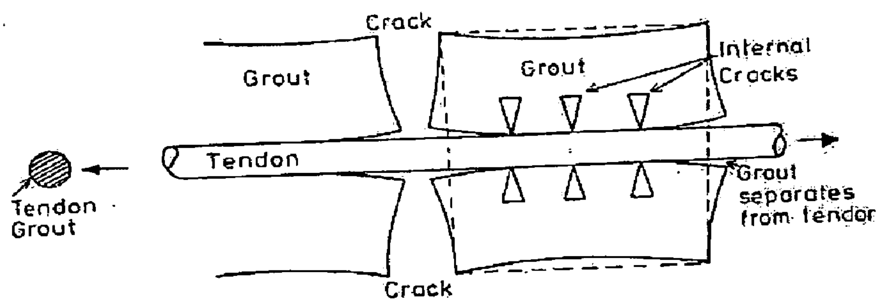
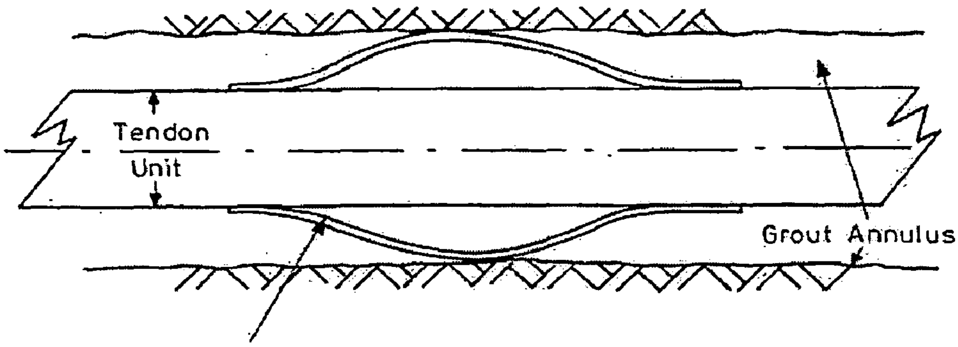


Fig. 3.5: Cracking of grout surrounding steel tendon

To minimise the effect of cracks minimum grout cover shall be maintained throughout the fixed length of anchor (usually>10mm) and provision of centralizer and spacer system shall be provided in the case of multi-strand tendon having longer lengths.

The main function of tendon centralizer is to maintain the steel centrally within the grout column and to ensure the minimum grout cover throughout the fixed length. The spacing of centralizer is decided on the basis of self weight of the tendon and the inclination of the hole. The function of the spacers are to separate the individual units of the tendon e.g. strands shall be spaced in such a way that proper grout cover is maintained among all strands. General arrangement of centraliser and spacer has been shown in fig. 3.6 and fig. 3.7.



Centralizer securely fastened to tendon unit

Fig. 3.6: Detail of centralizer system

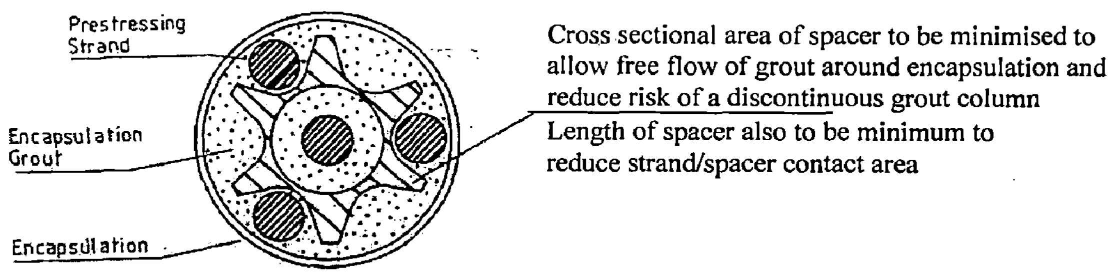


Fig. 3.7: Detail of a spacer system for a multi-strand system

3.2.3.2 Mechanics of bond between grout and borehole wall

The anchor load P is given by

$$P = \pi . d . L . \tau$$

Where L = fixed length considering a cylindrical borehole d = diameter of borehole

τ = average working bond stress

In the above equation it is assumed that there is no local debonding at the grout/ground interface and failure takes place by shear at the grout/ground interface. Also over the whole of the fixed anchor interface, bond stress has been assumed to be distributed uniformly. In reality, such idealizations do not exist. However, a number of approaches have been used to ascertain the proper solution.

After the anchor is loaded, the adjacent ground experiences a very complex loading sequence.

The formation of failure surface depends on roughness of the anchor hole, the strength of the ground & changes due to nearby construction works. Due to drilling in the hole, mechanical disturbance is created and drilled hole causes stress relief, which is very difficult to quantity. Also during setting of the grout, there are movements of fluids from the grout to the surrounding ground and from the ground to the hardened grout. The result of this is that the interface between the grout & ground is far from being well understood because various ranges of ground types exist in the nature. On the loading of an anchor tendon, stresses are transmitted from the grout column to the ground in the form of radial stresses and shear stresses. At which location failure will occur, this is not clear. One assumption is that failure may be at some distance from borehole wall and second assumption is that it may be on the interface. This depends on the relative strength of the interface and the adjacent ground. Now further, the ground condition may decide probable location of failure. In the case of hard rock, the failure may be at the grout/rock interface or even in the grout itself. Whereas for soft rock or a soil like a stiff clay, failure may occur at short distance (probably a few mm) into the ground.

The bond creates problem when failure occurs at an interface, because the interface is brittle in behaviour. Hence debonding is most likely to occur where the load levels approach the limiting value. The problem becomes more significant in case fixed anchor length is long.

In the case of rock anchors the bond resistance has been tried to relate with strength of rock. According to Littlejohn and Bruce (1976) average bond stress value shall not exceed the half the minimum shear strength of the rock. This is applicable to rocks of uniaxial compressive strength less than $7 N/mm^2$. In the absence of shear strength test data, the ultimate bond resistance can be taken as 10% of uniaxial compressive strength of massive rocks with a maximum limiting value of $4N/mm^2$.

The range of fixed anchor length for rock anchors which have been used in practice varies between 3m-10m considering the effect of quality of the rock, the presence of weakness, significance of errors in construction dimensions and quality (Littlejohn and Bruce, 1976). Also distribution of bond length along the length of the fixed anchor mainly depends upon the stiffness of the tendon relative to that of the rock; the higher the ratio of E_{grout}/E_{rock}, the more uniform the bond distribution.

It is very important to note that with high capacity anchors, some debonding over the fixed anchor length is possible. As the load increases, the debonding progresses towards the lower end of the anchor. Therefore, in design, average bond values shall be used.

3.2.3.3 Recommended bond stress

The values of ultimate bond stress given in the Table 3.1 are guide values only. The design assumptions adopted prior to installation of anchors shall also include the following.

i) Core drilling to explore the rock quality.

ii) Core testing together with pull-out tests of test anchors.

In order to obtain the safe working load, suitable factor of safety shall be used. Normally safety factor should range from 1.5 to 2.5. Safety factor applied to the ultimate bond stress shall be obtained from either pull-out tests or bond stress table.

Table 3.1: Typical Bond Stresses for Rock Anchor

	Ultimate Bond Stresses Between Rock and Anchor-Grout Plug
Type	**Sound, Non-decayed**
Granite & Basalt	250 - 450 psi (17.6-31.7 kgf/cm^2)
Dolomitic Limestone	200 - 300 psi (14.1-21.1 kgf/cm^2)
Soft Limestone*	150 - 220 psi (10.6-15.5 kgf/cm^2)
Slates & Hard Shales	120 - 200 psi (8.4-14.1 kgf/cm^2)
Soft Shales*	30 - 120 psi (2.1-8.4 kgf/cm^2)
Sandstone	120 - 250 psi (8.4-17.6 kgf/cm^2)
Concrete	200 - 400 psi (14.1-28.2 kgf/cm^2)

(EM 1110-1-2907, Feb., 1980)

*Bond strength must be confirmed by pullout tests which include time creep tests.

Proof loading of every anchor should not be less than 110 percent of its transfer (lock-off) force. The prestressing force shall not be more than 80 percent of the guaranteed ultimate tensile strength (GUTS) of the high strength steel during the proof loading operation. The duration of the proof loading is usually up to 15 minutes, in case the prestressing force is held by the jack. In case proof loading is done for longer duration, then it is recommended to transfer the force to the anchorage and remove the jack. Generally transfer (lock-off) force shall be taken between 50 and 70 percent of its guaranteed ultimate tensile strength. The difference between transfer load and design load shall include allowance for time dependent losses.

For small load strand anchors (such as single strand) the predominance of bond lies between grout and strand. The bond capacity between grout and strand is about 450 psi.

The bond values also have been given in IS: 10270-1982 for computing fixed length of Anchors. The following values of allowable rock grout bond from IS: 10270 are shown in the Table 3.2 below. Also as per IS: 10270, the fixed length (bond length) should be provided considering the following aspects:

a) From the data available on typical rocks (Table 3.2)
b) Co- relation between unconfined compressive strength of rock and bond value (Fig.3.8), and
c) Experience with similar type of rocks in adjacent areas.

Table 3.2: Allowable Rock Grout bond values

Sr. No.	Type of Rock	Allowable Bond Stress
(1)	(2)	(3)
i)	Basalt	5 kgf/cm^2 to 7 kgf/cm^2
ii)	Khaondolite/Charnokite	3 kgf/cm^2 to 5 kgf/cm^2
iii)	Granite	5 kgf/cm^2
iv)	Shale	3 kgf/cm^2
v)	Weathered sandstone/Quartzite	2.5kgf/cm^2
vi)	Jointed quartzitic	3.5kgf/cm^2
vii)	Grey chioritic schist	3.5kgf/cm^2
viii)	Sandstone/ Quartzite	3 kgf/cm^2

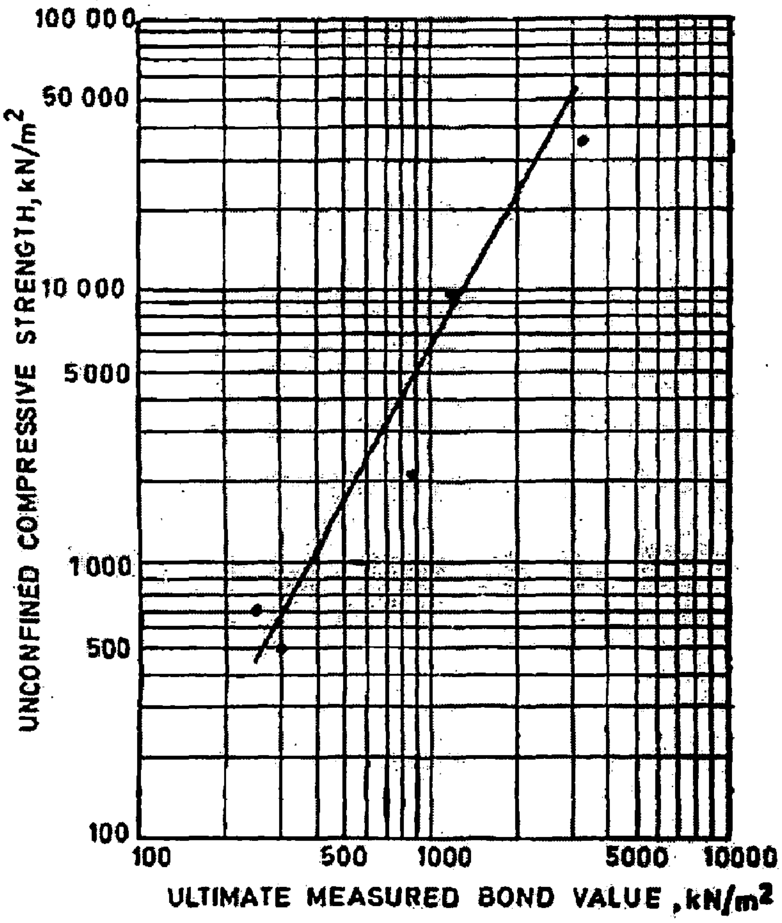


Fig. 3.8: Tentative relation between bond and unconfined compressive strength

Table 3.3: Comparison of allowable bond values (in kg/sqcm) as per IS code, EM & others

Sr. No.	Type of Rock	As per IS code	As per EM (FOS 1.5)	As per EM (FOS 2.0)	As per EM (FOS 2.5)	As per Littlejohn & Bruce,1976
1	Basalt	5-7	11.73-21.11	8.8-15.83	7.04-12.67	6.3 - 19.3
2	Granite	5	11.73-21.12	8.8-15.84	7.04-12.68	6.3 - 15.6
3	Shales	3	5.63-9.38	4.22-7.04	3.38-5.63	3 - 6.2
4	Sandstone	3	5.63-11.73	2.22-8.8	3.38-7.04	3.1 - 11.7

After comparing bond values (Table 3.3), it is found, the values given in IS code are on lower side. The values given in EM 1110-1-2907, Feb.,1980 (EM stands for Engineer Manual) after applying suitable factor of safety (FOS) can be used in the design of anchors subjected to core testing together with pull-out tests of test rock anchors.

3.2.4 Corrosion Protection in Anchors

The life of structural component is controlled by three main factors: corrosion, fatigue and wear. For the anchors in rock/soil, first factor i.e. corrosion has its predominance, while effect of fatigue & wear can be neglected. The metals in the presence of water or moisture react with oxygen and metal hydroxyl is formed as below:

$$\text{Metal} + O_2 \xrightarrow{(H_2O)} \text{Metal}\ (OH)_x$$

The formation of metal $(OH)_x$ is mainly responsible for corrosion.

The overall objective for the anchorage system is to ensure that during the planned design life of the anchor unit, unacceptable corrosion does not occur. The corrosion is most likely to start where there is crack in contact with the tendon steel.

In the fixed anchor zone the tendon steel may be in contact with a cement grout. All concretes are permeable to a certain degree and thus gas & their solutions can permeate. When steel is left unprotected in the atmosphere, it gets rusted. Rusting can be prevented by providing an alkaline environment with a pH in the range 9 to 13. Hydrated cements have a pH value of about 12.5. The penetration of carbon dioxide and sulphur dioxide gases react with the alkali and hence alkalinity is significantly reduced. With the further permeation at the tendon steel surface, corrosion is likely especially if both oxygen and water are available. The penetration is usually small when the concrete is sound. There is difficulty in penetration, if concrete has good cover (> 10 to 15mm).

Due to tensile loading, all cementations materials crack and when this crack reaches the steel, corrosion attack is possible. Under loading, the steel tends to debond itself from the surrounding concrete.

The magnitude of corrosion depends on

(i) Crack width

(ii) Exposure to pollutants in the ground

(iii) The nature of loading (static or pulsating)

The presence of chloride ions, reduce the alkalinity which can create problem. Therefore, major precaution shall be taken to ensure that chlorides are not deliberately added to cement grouts.

Corrosion risk near anchor head

At the interface between the anchor head and start of anchor borehole, the potential hazard for corrosion is maximum due to following reasons.

(i) In this area, there may be movements (in the form of thermal and/or ground rebounds) and settlement of the structure.

(ii) In this region, ground water seepage is comparatively high.

(iii) Near to the ground surface, the soil may be partly disturbed and aerated above the permanent ground water table.

The tendon in this region is passed through metal or plastic pipe or casing through the structure and into the ground. The tendon is passed through this pipe to the anchor head. The surrounding void shall be carefully filled with a cement grout or resin. Care has to be taken to vent off the air during the filling operation.

Requirements of a corrosion protection system

There is no reliable and proven method of predicting rates of corrosion. Now it has become normal practice to provide double protection system for permanent anchors. Here double protection means, the provision of at least two physical barriers against corrosion attack. Certain requirements are needed to be fulfilled for this system.

(i) The effective life to the anchor protection should be equal to that of the anchor, provided the efficiency of anchor is not affected.

(ii) The anchor tendon must be free to move over the free anchor length and during stressing, load shall be transferred to fixed anchor zone.

(iii) The protection shall be flexible and robust, so that it does not fail during stressing. In case it gets damaged, then it can not be replaced or repaired.

(iv) The protection shall not be damaged during manufacturing, handing and installation period.

Overall aim is to achieve a corrosion protection system which covers full specifications of material to be used and should

a) Remain crack free and not become brittle or fluid over the anticipated service temperature range.

b) Remain chemically stable, non-reactive with adjacent materials and impervious to moisture (Hanna, 1982).

Protection Systems

Generally two methods of protection system are used. Either a pre-protection in employed prior to insertion of the tendon into the hole or a post-construction protection system is used. Plastic taps may be used for temporary protection. They are wrapped over the free anchor length with a 50% overlap. Greasing is done over the tendon to prevent moisture being trapped below the tapes. In addition, free movement of tendon is allowed relative to outer column of protective grout. The best protective sheath is obtained by extrusion of the coating on the tendon in the factory. A method has also been developed where it can be applied to the tendons on site, (Littlejohn and Truman-Davies, 1974). The sheaths made of poly propylene are used in general as compared to PVC coatings, as later becomes brittle during long-term use. The atmosphere included between steel & sheath shall be removed by grease.

Apart from use of sheaths & tapes for protection of steel, the considerations have to be given to account for deformations which occur when the tendon is stressed. Normally cement grouts are used as fixing medium to transmit the fixed anchor loads to the walls of the anchor hole. Cementations grouts are brittle and cracks develop when the tendon is loaded. In order to ensure reasonable grout protection in this cracked condition, it is usual to keep a minimum grout cover of 20 mm with the use of centralizers to maintain the required grout cover. More expensive epoxy resin grouts can be used to overcome the above mentioned cracking condition.

Post-protection systems are also used and installed after the anchor has been constructed. In this case, system involves filling the annulus around the tendon in the free anchor length after anchor stressing.

Anchor Head Protection

The bare tendon steel at the anchor head is required to be protected against corrosion. Also bearing plate needs protection. The basic principle followed is to enclose the exposed head but simultaneously it should allow freedom to pre-stress the tendon and to allow load changes to occur in the anchor while in service.

Cement grouts are not used for inner head protection, because no flow of water is allowed in inner head area. The protection of outer parts can be done as per requirement of strands to be stressed i.e. restressable or non-restressable. In the case of restressable multistrand, grease filler is normally used within plastic or steel caps and when restressing is not required then resin and other sealants can be used because there is no need to remove the cap. Anchor head details for restressable multi-strand tendon and non-restressable multi-strand tendon are shown at Fig. 3.9 & 3.10 respectively.

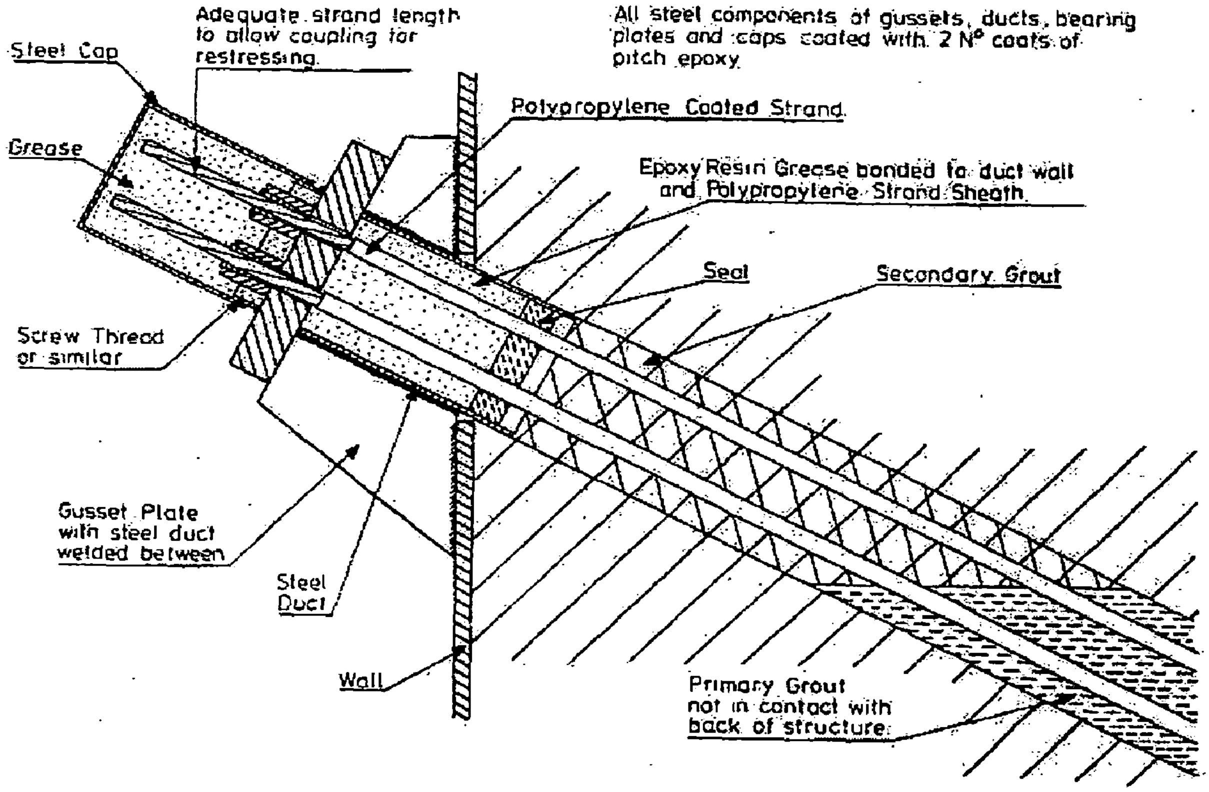


Fig. 3.9: Anchor head details for restressable multi-strand tendon

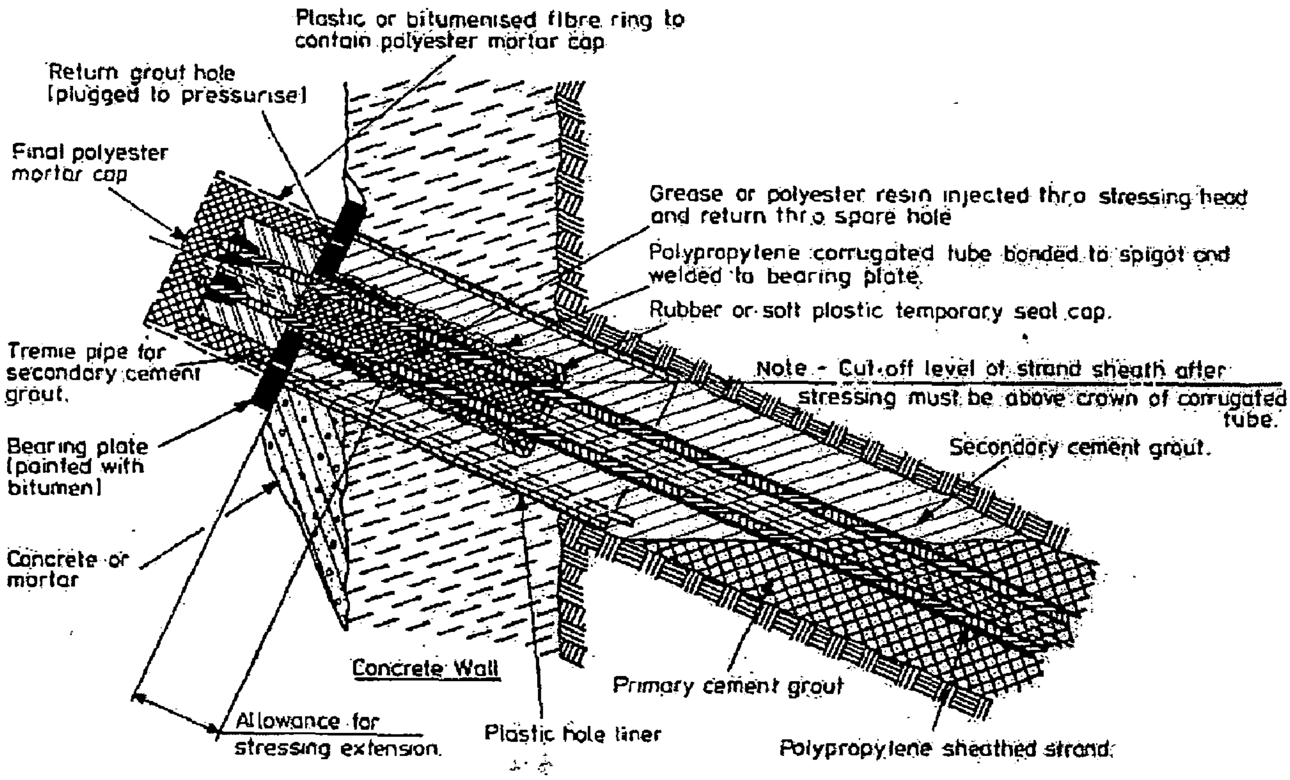


Fig. 3.10: Anchor head details for non-restressable multi-strand tendon

3.2.5 Determination of Anchor Embedment Depth

The ground adjacent to an anchorage must have sufficient resistance to withstand the bursting forces generated by the stressing of the anchorage. For large anchor loads in rock, care is necessary in the selection of the anchor depth to ensure that failure does not occur in the rock mass. Anchors may function in isolation or in groups. Both cases shall be considered to determine anchor depth. Commonly, it is assumed that an anchorage mobilises the resistance of a conical or wedge-shaped body of material surrounding it. Over the years many approaches have been devised to prevent failure within the rock and the most common is to consider an inverted cone of rock to be pulled out as shown in Fig. 3.11.

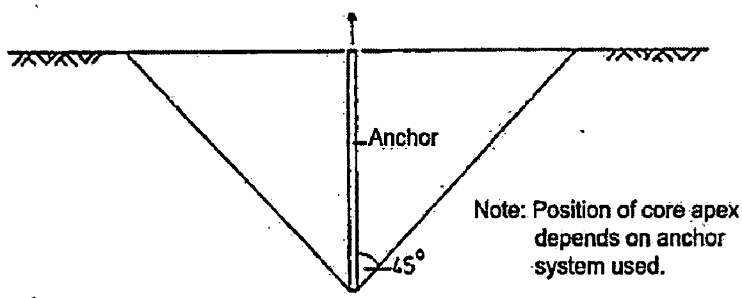


Fig. 3.11: Theoretical cone of rock pulled out by anchor in isotropic rock (Hobst and Zajic, 1977)

Different opinions prevail over the location of the base of cone w.r.t location of fixed anchor length. In one case, location of the base of cone has been selected from the middle of the fixed anchor length (Morris and Garrett, 1956 Eberhardt and Veltrop, 1965). In another case, location of the base of cone has been selected from the top of the fixed anchor (Rawlings and Rescher, 1968). For the design purpose the free length may be taken from ground surface to top position of fixed anchor length to remain on safer side (as proposed by Rawlings and Rescher, 1968). The occurrence of intact and homogeneous rocks is rare. Rocks in general are jointed and fractured and it is for this reason that use of the cone method requires rock mechanics experience and realistic factors of safety (Hobst and Zajic, 1977). Very high shear strength exits in homogeneous rocks, while in fissured and altered rocks the strengths are relatively low. For the estimation of anchor depth (free anchor length), rock shear strength has been considered in homogenous material but neglected in the fissured rocks as shown in the expressions in Table 3.4. In case anchor spacing is less, then cones interact as shown in Fig. 3.12 and the anchor embedment depth estimation for different type of rocks is given in Table 3.5 (Hobst, 1965).

Table 3.4: Empirical method of determining anchor embedment depth in rock using the cone method (Hobst, 1965)

Type of Rock	Depth of cone
Homogeneous	$\sqrt{\dfrac{F.P}{4.4\tau}}$
Irregular fissured	$\sqrt[3]{\dfrac{3F.P.}{\gamma\pi\tan\phi}}$
Irregular fissured and submerged	$\sqrt[3]{\dfrac{3F.P.}{(\gamma-1)\pi\tan\phi}}$

Where

τ = rock shear strength

F = factor of safety (2 to 3)

ϕ = angle of friction across fractures in rock

γ = unit weight of rock

P= anchor load

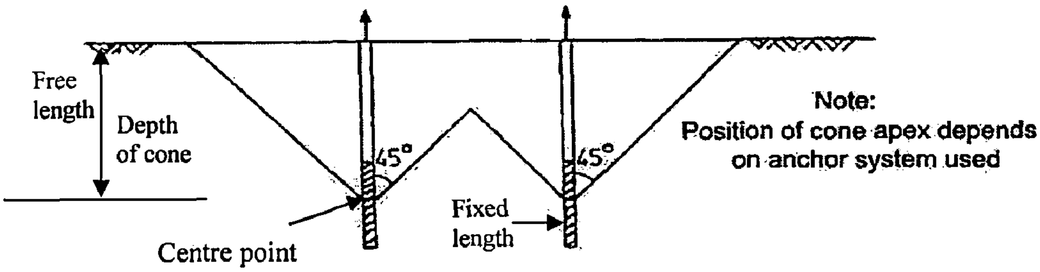


Fig. 3.12: Interaction of theoretical failure cones of rock from two adjacent anchors

Table 3.5: Empirical method of determining the anchor embedment depth in rock using the cone method-groups of anchors (Hobst, 1965)

Type of Rock	Depth of cone
Homogeneous	$\dfrac{F.P.}{2.8.\tau.s}$
Irregular fissured	$\sqrt{\dfrac{F.P.}{\gamma.s\tan\phi}}$
Irregular fissured and submerged	$\sqrt{\dfrac{F.P.}{(\gamma-1)s\tan\phi}}$

s = spacing of the anchors.

3.2.6 Determination of Prestressed Anchorage Force for Block of Rock

In order to determine prestressed anchor load, consider a block of rock as shown at Fig. 3.13, which is tending to slip on the discontinuity A-A. For the example purpose let the hydrostatic uplift acting on the base of the block be U_1 and the hydrostatic load from the water filled tension crack be U_2. Let a prestressed anchor is inclined at an angle of α with A-A. Let T be the required anchorage force and W be the weight of block of rock.

Resolving forces along the plane and normal to plane, we get

Resisting force = $c.l + (W\cos\theta - U_1 + T\sin\alpha)\tan\phi$

Disturbing force = $W\sin\theta + U_2 - T\cos\alpha$

For the equilibrium condition,

$$W\sin\theta + U_2 - T\cos\alpha = c.l + (W\cos\theta - U_1 + T\sin\alpha)\tan\phi$$

Where c and ϕ are cohesion and angle of internal friction of rock respectively.

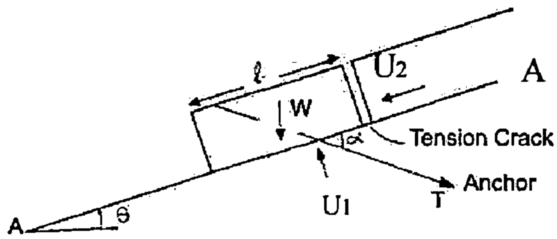


Fig. 3.13: An anchor supporting block of rock

From above equation, it is found that the anchor reduces the disturbing force. Also, the anchor increases the normal stress on the plane A-A.

Rearranging the above equation, an expression for T can be found as

$$T = \frac{W\sin\theta + U_2 - cl - W\cos\theta\tan\phi + U_1\tan\phi}{(\cos\alpha + \tan\phi\sin\alpha)}$$

The minimum value of T results when $\alpha = \phi$.

The factor of safety of this block of rock against sliding is

$$F = \frac{cl + [W\cos\theta - U_1 + T\sin\alpha]\tan\phi}{W\sin\theta + U_2 - T\cos\alpha}$$

This equation shows that the stability of the block of rock may be increased by a reduction in U_1 and / or U_2. This means drainage improve the stability. Further by increasing T, i.e. anchor load, factor of safety may be increased.

Also, it cannot be assumed that c,ϕ, U_1, U_2 and T are fully mobilized simultaneously. Therefore, some factor of safety is assigned to each of these variables. The values of the factors of safety depend upon the degree of accuracy in the data. If the factors of safety for c, ϕ, pore water pressures and self-weight are F_c, F_ϕ, F_u and F_w, respectively, then above equation is modified as under:

$$T = \frac{W.F_w \sin\theta + F_u.U_2 - F_c.c.l - F_w W \cos\theta \dfrac{\tan\phi}{F_\phi} + F_u.U_1 \dfrac{\tan\phi}{F_\phi}}{\left(\cos\alpha + \dfrac{\tan\phi}{F_\phi}\sin\alpha\right)}$$

The value of F_w can be taken as unity for self-weight forces. The values of F_c and F_ϕ will usually range up to 1.5 while F_u may be as large as 2 (Londe et al. 1969, 1970).

3.3 CONSTRUCTION CONSIDERATIONS

3.3.1 Different categories of ground anchors

Ground anchors are categorised in various types as briefly described below.

(i) By the anchorage zone:

Ground anchors can be categorised as soil or rock anchors corresponding to the ground in which the fixed length is anchored. The same anchors may be used for different ground conditions, but the bore hole preparation, grouting procedure and the corresponding parts of the anchors may differ. In case of cohesive soils, the anchor may be equipped with post grouting pipes through which additional grout can be injected at high pressure which cracks the primary grout and consolidates the soil surrounding it (BBR, India Ltd.).

(ii) By classes:

The anchors can be classified as temporary anchors or permanent anchors according to their service life, corrosion protection requirements, risk assessment, cost etc.

(iii) By Material of Tendon:

The prestressing steel used in anchors is known as tendon. The tendon for anchor is generally made from one of the following three steel materials- bar, wire and strand. Now to decide what material shall be used is mainly influenced by following factors i.e. cost, stress level permitted, fabrication, transport, stressing problem, corrosion protection requirement etc. In general, wire and strand have advantages with respect to strength, transport, tendon manufacture and storage in comparison to using bar type tendons. The selection of wire/strand is normally useful with large load capacity anchors whereas for small loads the solid bar is easier to use and cheaper to install (Hanna, 1982).

Further details for the selection of tendon based on use of steel material are given below:-

(a) Wire type tendon

The 'Wires' are being used in anchors are generally supplied in the form of coil having solid section. The range of diameter of wires may vary from 2 mm to 8 mm and are available either "as drawn" or "pre-straightened". The pre-straightening process involves either a stress relieving heat treatment, which improves the elastic properties and gives the steel normal relaxation behaviour, or a hot stretching treatment, which also gives high elastic properties but produces low relaxation behaviour.

Relaxation is defined as loss in stress after a period of time when a tendon is stressed to a given load under constant length and temperature conditions. The rate of relaxation depends on the initial level of stressing and the duration of its application. These relaxation terms are also applicable to strand. The ultimate tensile strength increases with decrease in wire diameter and it has little influence on manufacturing method. The size of the tendon depends on the load to be carried and selection of wires may range from 10 to 100.

(b) Strand type tendon

In the case of 'Strand' a group of wires are spun in helical form around a common longitudinal axis whereas central wire remains as straight wire. The strand is made from cold-drawn low-carbon steel wire. The 7-wire strand is mostly used. To make 7-wire strand, six wires are helically wound to form a single layer about the straight inner core wire. A section of a 7- wire compacted strand is shown in Fig. 3.14. Normally, after forming the strand is subjected to a low temperature heat treatment. The number of strands in tendon usually varies from 4 to 20 and in general each strand comprises of diameter as 12.7 or 15.2 mm.

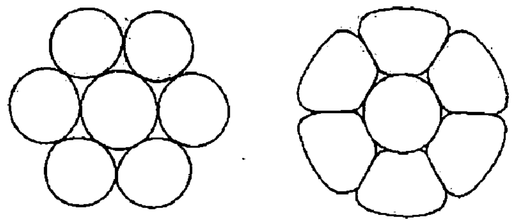


Fig. 3.14: Section of seven-wire strand before and after compaction (a) Normal, showing diameters of outer wires slightly smaller than that of core wire (b) Compacted (Hanna, 1982)

(C) Bar type tendon

The dia of bars normally varies from 12 to 40 mm and bars are usually made of high tensile alloy steel. Bars are either available in plain or deformed shape. In case of plain bars, threads are provided at the ends for anchorage and coupling purposes. The relaxation of steel bars as compared to strand/wire type tendon is between low & normal. Normally these are used for low capacity and when the tendons are relatively short. They are used in clusters of 2, 3 and 4 bars and on the basis of higher load requirement more bars can be used (Hanna, 1982).

(d) Carbon Fibre Reinforced Polymer

Carbon fibres tendons made of carbon fibre reinforced polymer (CFRP) wires anchored in a composite of ceramics and epoxy. A new technology which is under development has its advantages in the high capacity and the non corroding material. These are best suited where corrosion attack is high and research & development in this type of tendons is underway (Zhang Burong et al, 2001).

3.3.2 Different Anchor Types (based on hole arrangement)

There are in general three basic types of ground anchor systems based on hole arrangement in the fixed anchor length. These are shown in Fig.3.15 In the type 1 anchor on the basis of load to be mobilized, the hole dia is same in both free & fixed anchor length zone. It comprises a cylindrical hole filled with grout or other fixing agent.

In the type 2 anchor, the fixed anchor length zone which is also cylindrical is enlarged by grout injected into the sides of the borehole under high but controlled pressures. In this type main function of grout to from a bulb of strengthened ground, which acts as the anchorage.

In the type 3 anchor, the cylindrical portion in fixed anchor length zone is generally enlarged at one or more positions along its length by means of a special cutting device (Hanna, 1982).

3.3.3 Anchor Head

The stressing head assembly is required to transfer the load from the anchor tendon to the structure. This assembly normally comprises a stressing head, in which the tendon is fixed, and a distribution plate is provided for transferring the load. In case of large anchor forces, concrete pad/wall or steel structure are required, so that rock mass in contact may not burst. In the anchor head system provision shall also be there for grout pipes needed for secondary grouting.

A distribution plate is placed under the anchor head so as to prevent the overloading of the structure. Where a pad of concrete or continuous concrete wall is used, the design shall take care of relevant reinforced concrete Codes of Practice and design manuals.

In anchor systems the hardware generally is from the prestressed concrete industry. This comprises tendon steels, stressing head and grips, stressing equipment and grouting equipment.

The anchor head system in case of solid bar comprises a bearing plate and washer resting on a concrete pad. A detail assembly having bar type tendon is shown in fig.3.16.

In the case of anchors having multi-wire strands, the head of the tendon is secured by wedges or truncated cones. The wedges are pressed against the strands and forced by the prestress into tapered holes in the steel bearing plates. Later on when tendon stressing is over, the individual strands are locked in position by the wedges using either a mono- or multi-jacking method as shown in Fig. 3.17.

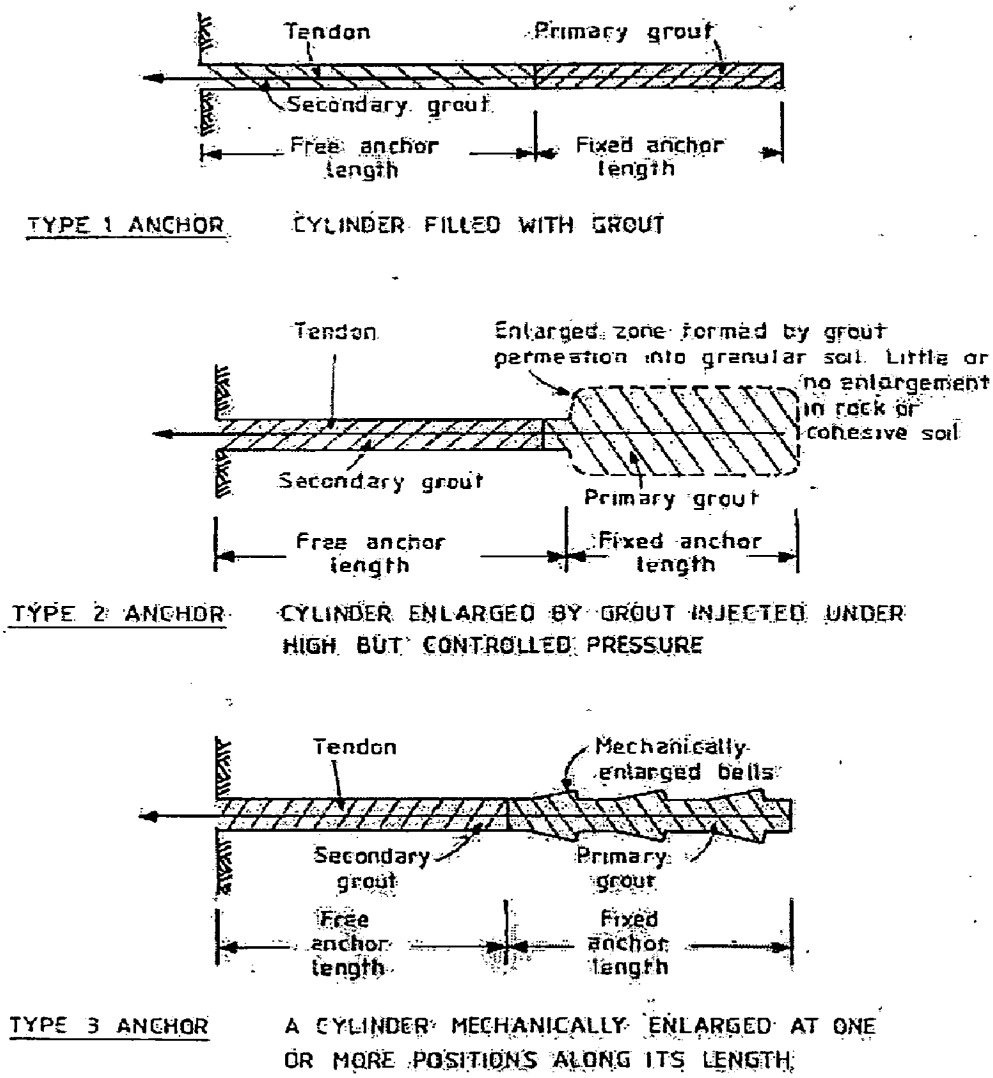


Fig. 3.15: Different anchor types in use (Hanna, 1982)

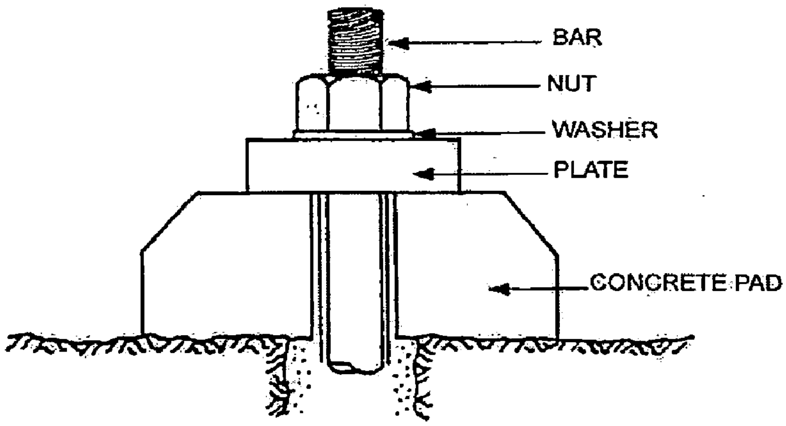


Fig. 3.16: Anchor head system having bar type tendon

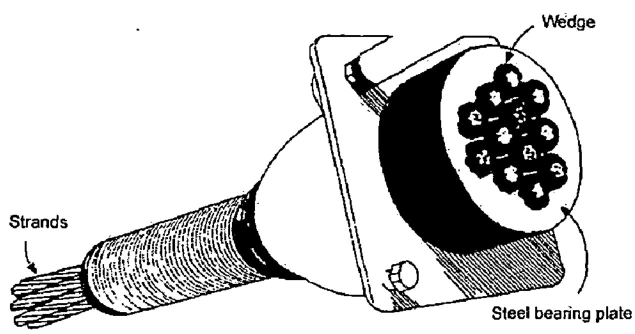


Fig. 3.17: Multi-strand anchor head system

3.3.3.1 Different type of Anchors (based on anchor head arrangement)

Standard anchor:-

This anchor uses the standard anchor head. It permanently locks the strands. The strand ends are cut off and the anchor can therefore not be adjusted nor re-or destressed (Fig. 3.18a).

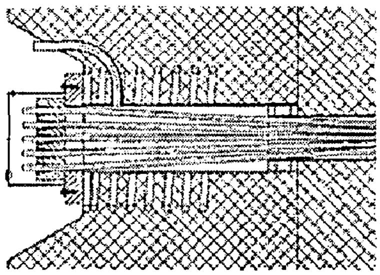


Fig. 3.18(a): Standard anchor

Control anchor:-

This anchor has a threaded anchor head to which a stressing sleeve can be attached and is used to check the anchor force and to restress the anchor (Fig. 3.18b).

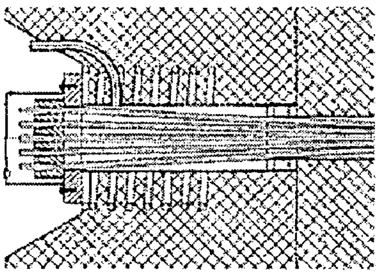


Fig. 3.18(b): Control anchor

Adjustable anchor:-

This anchor is also equipped with a threaded anchor head and has additional shims underneath. This type is used to check or restress as well as to destress the anchor completely. (Fig. 3.18c).

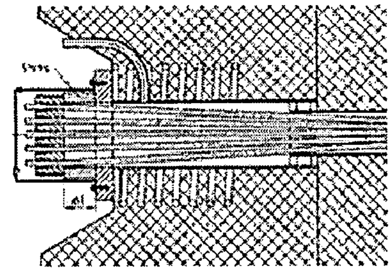


Fig. 3.18c: Adjustable anchor

Force monitoring anchor:-

This anchor is basically a control anchor but is additionally equipped with a force measuring device. This device can be linked to a remote control station for permanent monitoring (Fig. 3.18d).

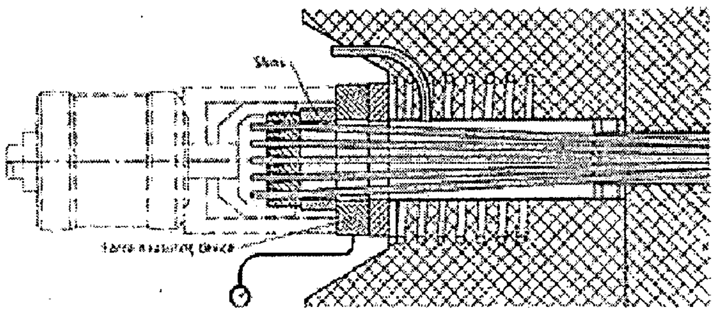


Fig. 3.18(d): Force monitoring anchor

(BBR India Limited, Banglore)

3.3.4 Details of different types of Tendons

Tendons may be formed from one of these i.e. bar, strand or wire. In case of solid bar tendon physical limitations to the size of tendon prevail mainly for the drilling and site handling conditions. Difficulty experienced is mainly with the damage to threads, and proper coupling of lengths of bars.

Strand steel for anchors is usually supplied to site in a greased and sheathed form. Fabrication is then performed in a workshop or under cover at site. Firstly, after deciding the overall length of the tendon, the individual strands are cut to that length. Next, the protective sheath in the fixed anchor length is removed and the grease is removed by means of a solvent or steam. Usually the individual wires of the strand have to be unravelled to assist the degreasing operation. In case tendon consists of several strands then it is necessary to use and fix securely spacers along the length of the tendon to keep the strands parallel. The spacers shall be strong enough to withstand handling and homing stresses. In general, over the fixed anchor length as far as possible the tendon steel is maintained in a near circular arrangement. With such spacers it is also possible to give the tendon a noded appearance, which improves the magnitude of the bond which can be developed between the grout and the tendon. The distance between spacers is generally kept at about 2m centres or less in the fixed anchor zone and along the free anchor length this spacing can be doubled. In order to keep the tendon near central position in the anchor hole, centralisers are used. They ensure minimum grout cover requirement over the tendon. Their spacing depends to some extent on the anchor inclination. Both spacers and centralisers are used in combination in some designs. Care has to be taken to ensure that the materials used for centraliser / spacer has no deleterious effect on the tendon steel.

Centralisers serve the dual function of ensuring a minimum grout cover to the tendon and also prevent sag of the anchorage between points of contact. The dimensions of the centraliser depend on the unit weight of the tendon and the softness of the anchor hole walls.

By using the spacers penetration of grout between the individual parts of a multi-strand tendon can be ensured. A spacing of at least 5 mm is provided between tendon units, in general.

The effect on load transfer and bond mechanisms by using both centralisers and spacers is not known with any certainty. Therefore it becomes necessary that proper field evaluation test trials are performed to assess the limitations, if any.

Further comment is made in Chapter 3 under corrosion protection of anchor systems. At the bottom end of the tendon a protective device in the form of a nose cone is generally provided. This serves a double purpose; firstly it reduces the possible damage to the tendon during handling and homing in the anchor hole and secondly borehole wall gets minimum damage during homing of the tendon.

Fig. 3.19 shows details of the arrangement of a multi-strand tendon whereas Fig.3.20 shows a layout for a bar tendon.

Generally, inspection is carried out after completion of tendon fabrication and assembly to check dimensions, adequacy of spacers and centralisers, damage to the protective systems along the free and fixed anchor lengths and the grout pipes which may be within or outside the tendon bundle.

Special carriage arrangements shall be made for transport in case distances between fabrication and installation are large. If special arrangements are not possible, then complete tendon is carried manually for insertion into the anchor hole. Prior to homing of the completed tendon, care and protection are necessary to avoid either physical damage or corrosion.

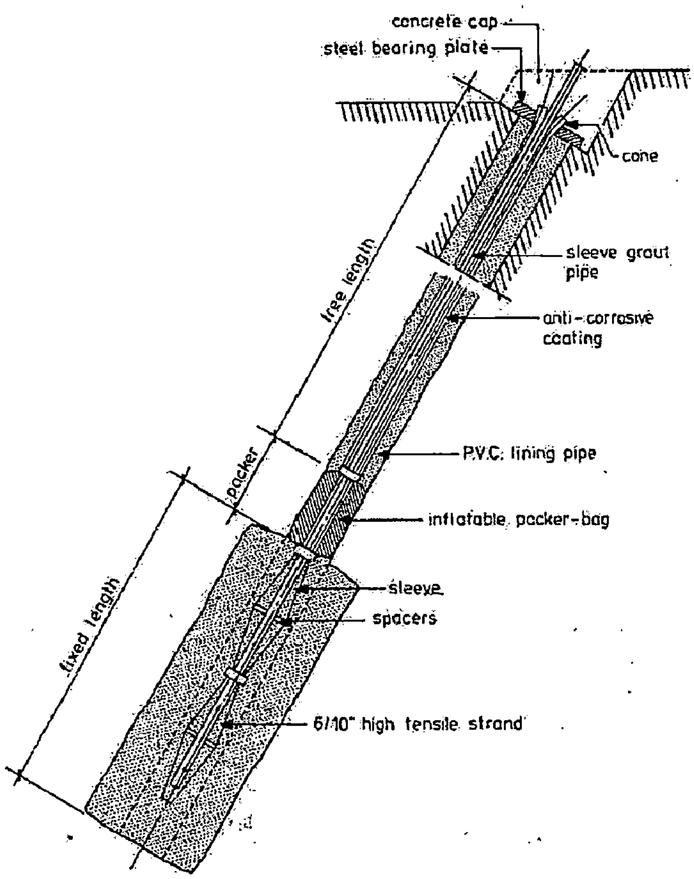


Fig. 3.19: General arrangement of a multi-strand tendon

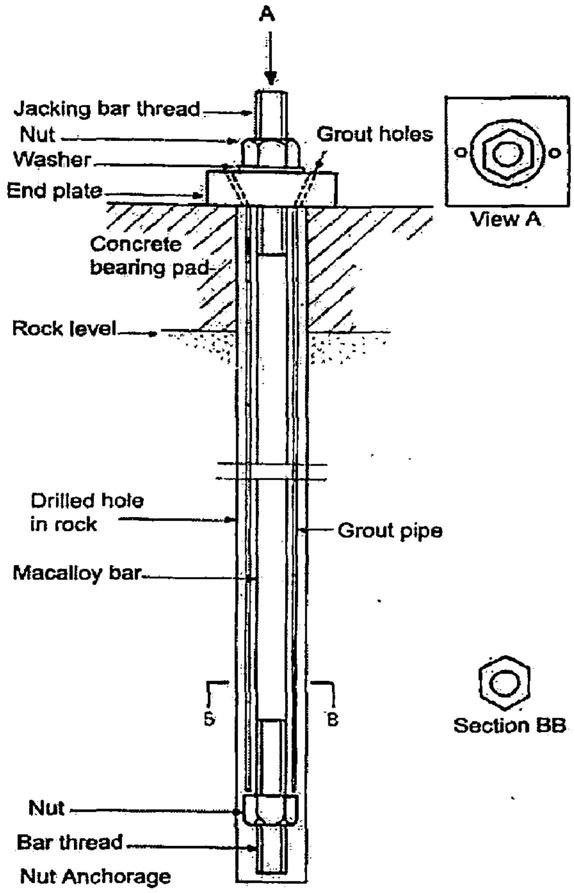


Fig. 3.20: General arrangement for a bar tendon

3.3.5 Stressing of Anchor

Main purpose of anchor stressing is to load each anchor to a known load level in a standard manner. Apart from this load/ deformation behaviour of particular anchor shall be recorded so that it may be compared with the known behaviour of test anchors or with other established criteria. Also, it may be necessary to test the tendon components. To test the tendon components some standard test methods are available and these are normally provided by the suppliers in the-form of test certificates for load/ extension data and ultimate strength values.

Normally direct pull method is most suitable for anchor stressing. But torque can be used to stress low capacity anchors and rock bolts. Anchors are usually stressed by direct pull using jacks capable of pulling all the strands or wire together. Anchor bar stressing is normally done by use of a hydraulic jack as shown in Fig. 3.21.

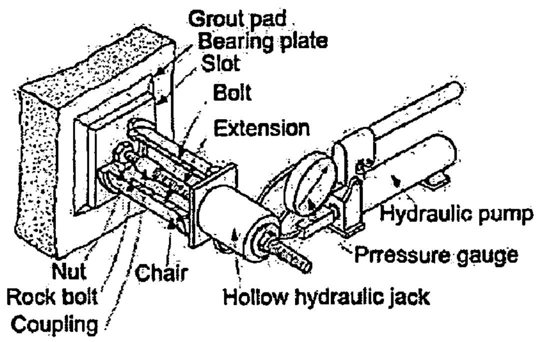


Fig. 3.21: Anchor bar stressing by use of hydraulic jack

In the absence of rigid structural member, a bearing plate is necessary to be placed on the ground surface to transfer the load and this plate shall be placed normal to the direction of loading. Applied load and anchor tendon extension are to be measured during the stressing operation. When high accuracy is needed or where chances of structure deformation are there during stressing operation, it is necessary to use a movement datum which is independent of the structure. Before starting stressing operation it shall be ensured that strands or wires are not crossed or fouled, in the free anchor length. By the use of spacers and centralizers this can be ensured. To maintain the uniform distribution of load in all strands of a tendon is difficult task. This aspect shall be taken seriously for very short anchors, where a small variation in tendon stretch can represent a large variation in load in an individual strand. The basic purpose of anchor stressing is to confirm the load carrying competence of each anchor constructed and in particular the ability of the fixed anchor length to carry its design load with a known factor of safety (Hanna, 1982). In order to transfer the load completely in fixed anchor length it has to be ensured that complete debonding of the tendon steel occurs along the free anchor length. After the satisfaction of this condition it is possible to assess the credibility of the load/extension values obtained. The differences between gross and net extensions of the tendon shall be known. The measured extension of the tendon after stressing but without locking tendons into wedge grips is the gross value. After locking the tendons into wedge grips there is small loss in the extended tendon length due to pull for maintaining equilibrium. Fig.3.22 shows method of gripping multi-strand tendon at anchor head. Apart from this, other losses may occur due to movements of the structure, the anchor bearing plate and slip of the tendon within the fixed anchor length. The movement measurement is mostly done by recording the stretch of the ram, but this is not reliable because slip of the strands relative to wedge grips may occur and give an overestimate of the tendon extension. It is better to provide marks on individual strands or wires and measure their positions relative to the load bearing plate or with respect to an independent datum by survey methods. The movement in the fixed anchor length can be measured by using an extensometer fixed at the top of the fixed anchor zone. On locking the load in the tendon, the wedges or grips pull into the tendon and slip movement occurs. This contributes to the phenomenon known as prestress loss during stressing. Such losses may be significant for short anchor lengths.

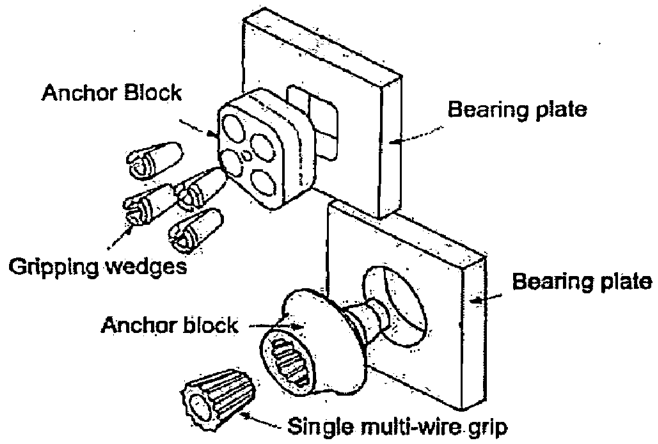


Fig. 3.22: Method of gripping multi-strand tendon at anchor head

CASE STUDIES

The study deals with design of prestressed anchors for stabilization of Himalayan slopes. The use of prestressed anchors has been examined in this chapter through three hydro projects located in Himalayas.

In the case studies, overall objective is to present analysis, design and construction aspects of different types of prestressed anchors used at three projects viz. Nathpa Jhakri Hydro-Electric Project, Yamuna Hydel Scheme Stage II covering Ichari Dam and Maneri Bhalli Hydro Electric Project Stage-II covering Dharasu Power House.

4.1 CASE STUDY NO. 1 (NATHPA JHAKRI HYDRO-ELECTRIC PROJECT)

In this case study, Design of Prestressed Cable Anchors for Stabilization of slopes in the vicinity of Nathpa Dam, Power Intake and Plunge Pool has been discussed. Detail design for one site i.e. Stabilization of slopes in the vicinity of Nathpa Dam covering stretch of about 120 meters on left bank has been given. For other sites as mentioned above design has been carried out on same principles. Design details cover method of analysis, adoption of different parameters, computation of anchorage forces at different locations, installation procedure technical specifications etc.

4.1.1 Salient Features of Project

The 1500 MW Nathpa Jhakri Hydro-Electric Project (NJHEP) is the first project commissioned by Satluj Jal Vidyut Nigam Limited (formerly Nathpa Jhakri Power Corporation Limited). The project is located in Shimla & Kinnaur Districts of Himachal Pradesh, along the National Highway No.22 about 150 km from Shimla. The Salient features are summarized below:

- A 62.50m high concrete Dam on Satluj River at Nathpa to divert 405 cumecs of water for power generation through four intakes.
- An underground Desilting Complex, comprising of four chambers, each 525m long, 16.31m wide and 27.5m deep, which is the largest underground complex of its kind in the world.
- A 10.15m dia and 27.39 km long Head Race Tunnel, which is one of the longest power tunnels in the world, terminating in a 21.6m/ 10.2m dia and

301m deep surge shaft.

- Three circular steel-lined pressure shafts, each of 4.9m dia and 571m to 622m in length, bifurcating near the power house to feed six generating units.
- An underground Power House with a cavern of size 222m x 20m x 49m having six Francis Turbine Units of 250MW each to utilize a design discharge of 405 cumecs and a design head of 428m.
- A 10.15m dia and 982m long Tail Race Tunnel to discharge the water back into the river Satluj.

All six units have been commissioned during the period Oct., 2003 – May, 2004. General layout of project has been shown in the Fig 4.1 and photograph showing view of Nathpa Dam from downstream side at Fig. 4.2 below:

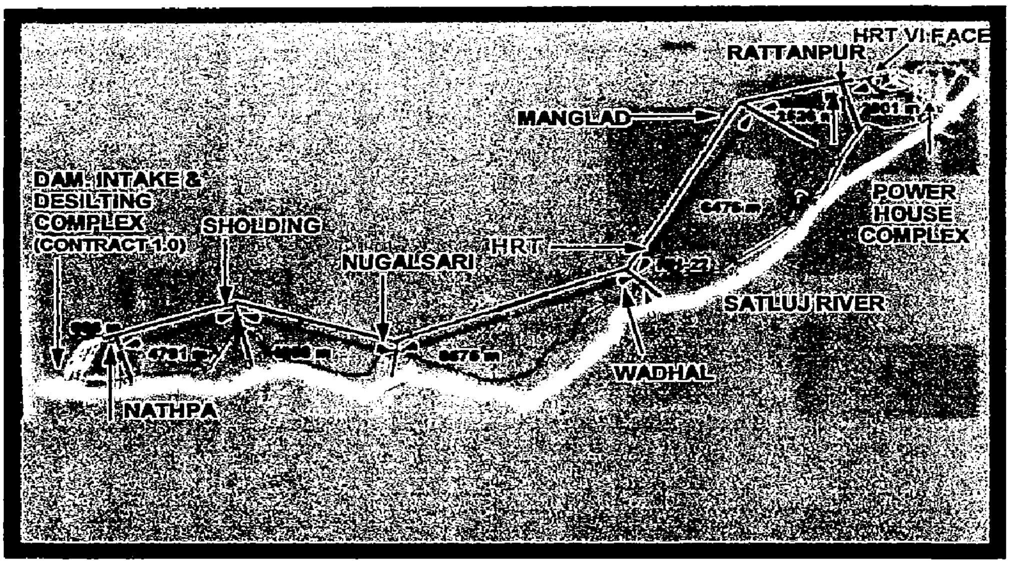


Fig. 4.1: General layout of Nathpa Jhakri hydroelectric project

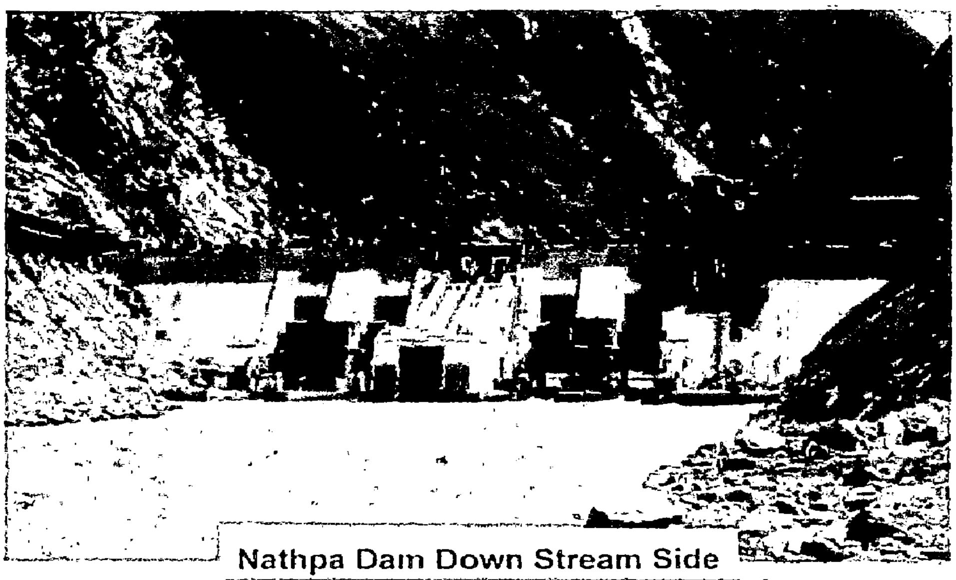


Fig. 4.2: Photograph showing view of Nathpa Dam from downstream side

4.1.1.1 Dam

The Nathpa Dam about 185 m long, is a diversion structure of the project, comprises of 12 blocks, each of width 15 to 18 m, inclusive of five low level sluices each of size 7.5m x 8.5 m operated by radial gates for passing a design discharge of 5660 cumecs. One overflow spillway with a capacity of about 100 cumecs is also provided. Three nos. grouting and drainage galleries of size 2.0m x 3.0 m, about 25-30 m long, on each bank abutment have been provided at different elevations of dam and connected with foundation gallery. The energy dissipation arrangement both for sluice and overflow spillway is ski jump type and jet falling on river bed is expected to develop a plunge pool about 40-50m downstream of bucket lip by scouring action.

The reservoir with a provision of storage to the extent of 343 ham stretches over a length of about 1.5 km and has an average width of 90-100m.

4.1.1.2 Power Intake

The Power intake structure of NJHEP consists of four Nos. independent intake blocks separated by contraction joints. The width of each block is 19.5 meters & each block has bell mouth opening of size 8.857m X 8.75m converging into rectangular shape of 6.0m X 5.25m which ultimately takes a shape of 6.0m dia horse-shoe tunnel. These have been designed to handle a design discharge of 486 cumecs, which includes 81 cumecs for flushing. Trashracks inclined at an angle of 12.5 degrees to the vertical, with clear opening of 68 mm to prevent the entry of large sediments inside the water conductor system, have been provided. Fixed wheel vertical lift gates at the end of each bell mouth have been provided. Each opening is further connected to an independent desilting chamber.

The hill slopes in area of the dam/intake and allied structures are steep with adverse geological features, necessitating specific measures for stabilization of slopes in the area for long term safety.

4.1.2 Identification of slopes instability problems on the banks of Nathpa Dam and adjoining areas

A massive rock slide took place on 8th July, 1993 on the right bank of the river just about 100 m upstream of the dam site. The slide to the tune of about 10 lacs cum completely blocked the river resulting in formation of lake upstream of slide having a length of about 1.5 km and depth of water was raised to about 20-25 m just upstream of slide. This slide also created danger for existing SVP (Bhaba) Power House (120 MW) having Pelton Turbine, located about 1 km upstream of Nathpa

Dam. Due to slide river bed level u/s of dam axis was considerably raised and the site of the inlet of originally proposed diversion tunnel was buried deep under the slide debris. This necessitated modification in layout of the diversion tunnel. Now inlet portal was shifted to u/s of its proposed original location with raised elevation, hence length of diversion tunnel increased from 410m to 738m.

This slide occurred about 6 months prior to the start of construction of Dam. The main contracts for major civil works stands awarded when this slide took place. In the main contract the provision for protection of slopes to tackle slide of such large magnitude in the Dam & Intake vicinity was not envisaged. On the left bank, slides have also been observed along the National Highway from time to time. Because of these slides further caution was necessary particularly in view of stabilization problems due to steep left bank slopes having foliation planes dipping towards the river bed. On left bank, construction of road at dam top level was unavoidable and minimum rock excavation was necessary for construction of dam and intake. Therefore, any rock cutting on left bank in the dam & intake area was expected to disturb the rock slope stability and could have resulted into a planar failure i.e. failure along the foliation planes.

The Right Bank slide was a planar slide caused due to presence of joints dipping at 50^0 towards the valley. The open nature of steeply dipping joints acted as avenues for the water seepage which lubricated the plane and thus activated the slide. Closer examination of the area also indicated stability problems on left bank during excavation of the access road and the dam foundation where under cutting of slopes into weak low shear strength biotite schist bands, present there, could occur.

Sheet jointed large rock blocks with thickness from 2 to 7m and having been separated at 5 to 20m widths are present in many places on the steep slope on the left bank. These blocks tend to slide down towards the river bottom. Steep slopes may result in loosening of surface rock due to stress relief and bank slopes to be under cut or side cut needed to be carefully treated so as to avoid stress relieving when excavation in undertaken. Because of steep slopes having number of foliation planes dipping towards valley & biotite schist bands located about 15-25m inside the hill almost parallel to exposed sloping surface dipping into valley, it was necessary to protect the slopes from sliding and necessary stabilization measures had to be taken accordingly.

On the right bank, joints are mainly dipping into the hill, but there are certain joints dipping towards river and in the location of gentle slopes in the dam vicinity, certain areas have been identified as talus, accumulated due to broken rock pieces and soil from the top of bank. Therefore, nominal stabilization measures were necessary on this bank also.

The ski jump type energy dissipation arrangement has been provided for both sluice spillway and overflow spillway in the dam. The ski jump jet is likely to develop deep scours in the river bed resulting in formation of a plunge pool. This scour will destabilize hill slopes.

4.1.3 Topography and Geology in the vicinity of Nathpa Dam

The bank slopes in the dam area and vicinity of dam varies from 40° to 50°. On left bank, slopes rise for a height of more than 140 m from the river bed at El. 1445 m to National Highway No.22 at El. 1585m. It is covered by overburden from road at El. 1585m to El. 1524m. Below El.1524 m, the rock comprises of gneisses with pegmatite intrusions, bands of quartz mica schist, biotite schist, amphibolites etc. The dip of the foliation planes of gneisses varies from 40° to 55° towards river side having strike almost parallel to the river flow. The dam site is located mainly in the area underlain by a gneiss group named Jeori-Wangtoo complex aged Pre-Cambrian. The foundation rocks in the dam complex are comprised of gneiss with bands of quartz mica schist, biotite schist on the left bank and of augen-gneisses with the same bands on the right bank. Topography of the area shows geologically young features characterized by rugged and steep slopes continuing to great high elevations on both banks resulting from remarkable dissecting.

The rock exposed on the right bank consists of mainly augen-gneisses with pegmatite veins. In general, the rock slopes are very steep and where the slopes are little gentler, the broken rock pieces and soil from the top have accumulated as talus. On the right bank, rock is relatively stable as bedding planes are dipping into the abutment. There are, however, certain joints that dip steeply towards the valley.

4.1.4 Types of Stabilization Measures Adopted

After identifying instability of existing slopes in the dam area, it was studied what type of stabilization measures are required in order to line up with overall construction schedule and keeping in view importance of structures like Dam & Power Intake. Consequently, careful analysis and investigations were carried out and a comprehensive plan for stabilizing these slopes was worked out and implemented. This includes drainage & dressing of overburden material, surface and subsurface drainage, construction of retaining walls and most important is anchoring of the potential slide rock mass with rock bolts and prestressed cable anchors of different capacity viz. 40,100 & 200 tonnes at different locations depending on magnitude of rock mass to be stabilized. From the computations based on potential failure plane & rock mass to be stabilized, it was found that destabilizing force comes to be very high, which can be tackled only by providing prestressed cable anchors of higher capacities. It was found that by providing only rock bolts or cable bolts & dressing of slopes at feasible locations, it was rather difficult to stabilize the slopes having huge destabilizing rock masses with unfavourable dipping & orientation of joints. Therefore, it became necessary to provide cable anchors of higher capacities alongwith rock bolts plus other measures like drainage & dressing of overburden material, surface & subsurface drainage, construction of retaining walls etc. In the proceeding paras, main stress has been laid on use of prestressed cable anchors of different capacities for the stabilization of slopes.

4.1.5 Criteria for deciding stabilization measures adopting prestressed cable anchors on left bank dam area

On the left bank, dam area has been analyzed for stabilization from 15 m u/s to 105 m d/s of dam axis in order to cover the entire length of dam & apron plus some margin on upstream and downstream of dam. The sections are analyzed at an interval of 15 m and detailed geological sections are used for stabilization computations. The proposed road cut alongwith proposed excavation profile of dam has been marked on these geological sections. Geological section at dam axis alongwith position of road on left bank, assumed failure plane, position of biotite schist band etc has been shown at Fig. 4.3.

The angle of potential failure plane has been taken as 45^{o} from 15m u/s to 60 m d/s of dam axis and further 40^{o} from 60 m d/s to 105 m d/s based on observed foliation angle of rock mass at different geological sections. The location of failure plane has been marked on each section based on deepest cut i.e. it's location w.r.t. exposed surface. In some sections, road cut is governing the deepest cut and in some sections dam foundation cut is governing the deepest cut. The sliding mass based on failure plane has been analyzed considering the effect of resisting forces, disturbing forces, earthquake forces, uplift pressure, angle of internal friction and cohesion along the failure plane. The anchorage forces at different sections are computed for a factor of safety of 1.1 for non-seismic case and 1.0 for seismic case. The critical of two cases are finally adopted in the design and capacity, number of rows and spacing of anchors was decided accordingly.

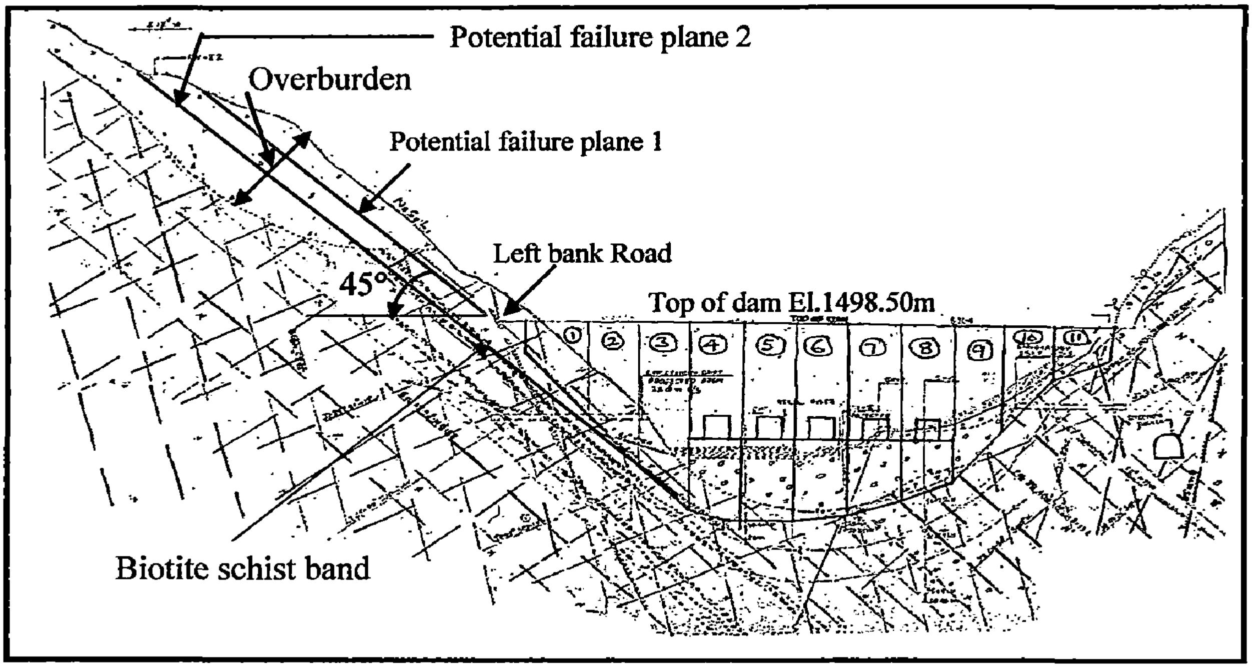


Fig. 4.3: Geological section at dam axis

The angle of internal friction (ϕ) and cohesion(C) for rock and overburden have been established on the basis of test results. The test values of cohesion found varying from 0 to 1.9 t/m^2 and of angle of shearing resistance from 28.3° to 32.9° for material encountered in the drift on left bank. It has been found that samples tested were not true representative of area under study. Back analysis of existing stable natural slopes has also been carried out for the angle of internal friction & cohesion for rock and overburden.

On this basis the values of angle of internal friction & cohesion for rock and overburden adopted for the design are 41.9° & 1 t/m^2 and 41° & 0 t/m^2 respectively. The uplift force is assumed to act on 25% of sliding rock mass and 12.5% for overburden material. The value of horizontal seismic co-efficient adopted is 0.08 (site is located at Zone IV of seismic map specified in IS: 1893) and vertical component is assumed to be 50% of that of horizontal component. Unit weight of rock and overburden are taken as 2.7 t/m^3 and 2.3 t/m^3 respectively based on test results in the dam area.

4.1.6 Design Computations

Figure 4.4 shows the various forces acting upon a sliding mass and combination of Resisting and Disturbing forces adopted in the design. Various parameters involved in the analysis are as under.

W_r	=	Weight of Rock
W_0	=	Weight of Overburden
l_r	=	Contact length of failure plane in rock
l_o	=	Contact length of failure plane in overburden
C_r	=	Cohesion in rock
C_0	=	Cohesion in overburden
ϕ_r	=	Angle of internal friction in rock
ϕ_0	=	Angle of internal friction in overburden
U_r	=	Uplift along the failure plane, in rock
U_0	=	Uplift along the failure plane, in overburden
α_h	=	Horizontal seismic coefficient
β	=	Angle of failure plane with horizontal
θ	=	Inclination of anchor with horizontal (adopted as 15 °)
T	=	Anchorage force

4.1.6.1 Non – Seismic case

$$\text{Resisting Force (RF)} = \{W_r \cos\beta - U_r + T \sin(\beta+\theta)\} \tan \phi_r + \{W_0 \cos\beta - U_0\} \tan\phi_0 + C_r l_r + C_0 l_0 \quad \text{--- (i)}$$

$$\text{Disturbing force (DF)} = W_r \sin\beta + W_0 \sin\beta - T \cos(\beta+\theta) \quad \text{--- (ii)}$$

Factor of Safety = RF/DF = 1.1 (Adopted for non-seismic case) --- (iii)

By substituting different values, anchorage force (T) required per metre width of sliding mass was computed using equations (i) to (iii) as mentioned above.

Numbers of rows (N) of cable anchors of capacity Tc at each section are worked out from the following equation.

$$N = T.\ b/ T_c \quad \text{--- (iv)}$$

where b is the horizontal spacing of anchors.

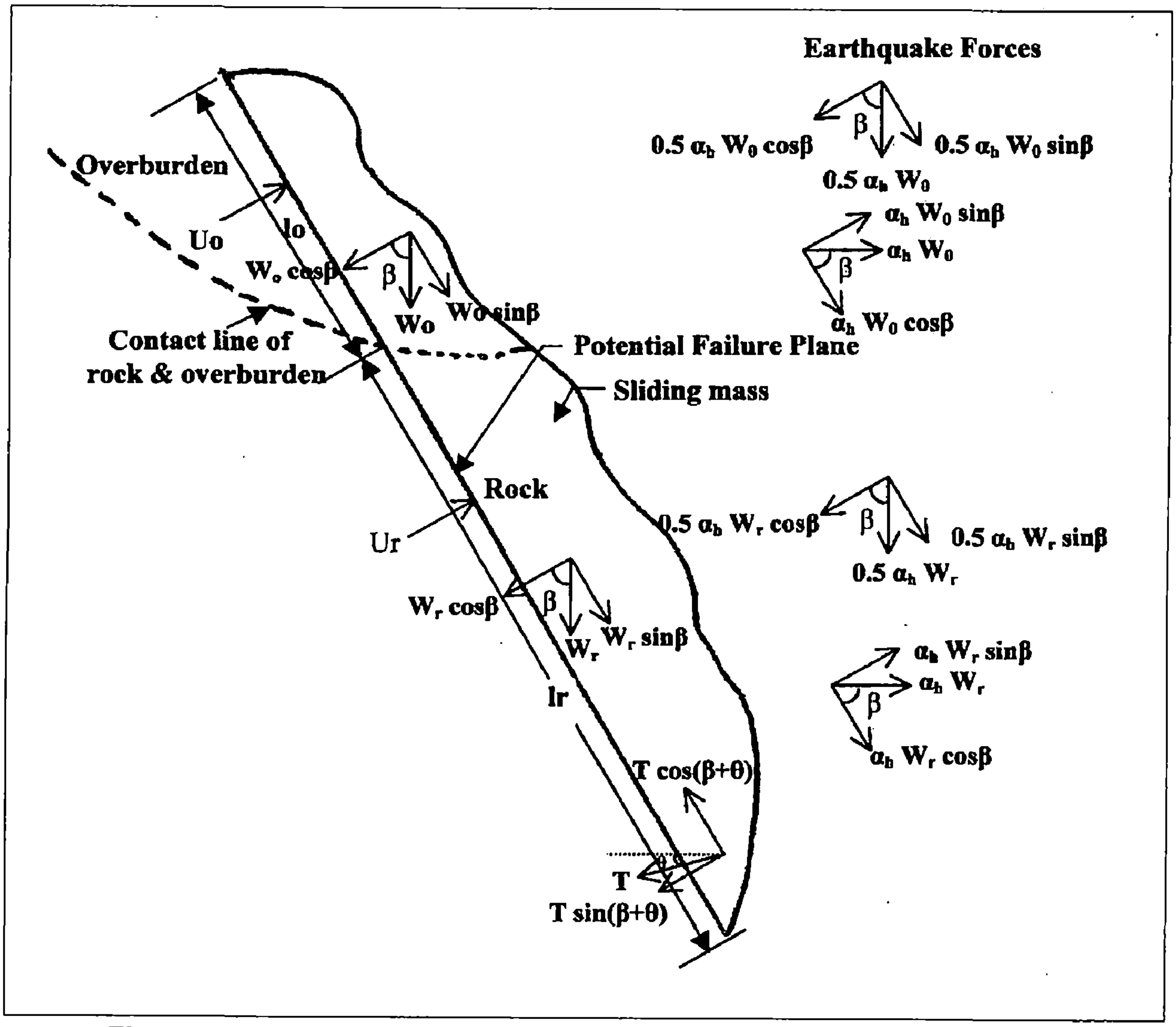


Fig. 4.4: Schematic sketch showing forces acting on a sliding mass

4.1.6.2 Seismic case

Resisting Force (RF) = $\{W_r \cos\beta - U_r - \alpha_h W_r \sin\beta + 0.5\alpha_h W_r \cos\beta + T \sin(\beta+\theta)\} \tan\phi_r$

$+ \{W_0 \cos\beta - U_0 - \alpha_h W_0 \sin\beta + 0.5\, \alpha_h W_0 \cos\beta) \tan\phi_0 + C_r l_r + C_0 l_0$

--- (v)

Disturbing force (DF) = $W_r \sin\beta + \alpha_h W_r \cos\beta + 0.5\alpha_h W_r \sin\beta + W_0 \sin\beta + \alpha_h W_0 \cos\beta$

$+0.5\alpha_h W_0 \sin\beta - T \cos(\beta+\theta)$ --- (vi)

Factor of Safety = RF/DF = 1.0 (Adopted for seismic case) --- (vii)

By substituting different values, anchorage force (T) required per metre width of sliding mass was computed using equations (v) to (vii) as mentioned above.

Similarly, number of rows (N) of cable anchors of capacity T_c at each section was worked out from the equation (iv) as mentioned above.

Finally; the critical result obtained from above two cases i.e. seismic & non-seismic was adopted for designs.

4.1.6.3 Computational methodology and results for case study

The anchorage force (T) required per meter width of sliding mass has been computed by using equations (i) to (iii) for non-seismic case and equations (v) to (vii) as mentioned above.

The sample calculation for computing anchorage force (T) & no. of anchor rows at one of the cross-sections at 75 m d/s of dam axis (seismic case) are shown below.

In this case failure plane has been marked based on foliation angle as 40° (i.e. β=40° based on geological information). At the location of 75 m d/s of dam axis, the deepest cut lies at El. 1498.50m i.e. left bank road level. The approach road to top of dam on left bank is 8.5 m wide including the concrete wall of 70 cm thick required for prestressed anchors. So, deepest cut at this section is governed by road. Section at 75 m d/s of dam axis showing different segments of overburden & rock is shown at Fig.4.5.

Based on road level and proposed cut for the road 8.5 m deep into hill, the point 'A' is marked in the figure. The proposed failure plane on the basis of foliation angle of 40° has been marked as 'AB' as shown. The loose unstable rock mass and overburden has been trimmed out as shown. The contact line of rock and overburden also has been shown. The areas of rock and overburden have been computed separately. The different areas have been divided in trapezium and triangle segments as shown. After calculating areas of rock portion and overburden, respective weights W_r and W_0 have been computed with known unit weights. The contact length of overburden (l_0) and contact length of rock (l_r) along failure plane measured from the drawing respectively.

Computer Programme using Excel

The symbols appearing in the computation tables not covered earlier are explained as below:-

The symbols 'p, q & x' (Table 4.1) are different dimensions of particular segment for computing the area. The 'p, q & x' are part of trapezium as shown in sketch at Fig. 4.6.

'x' being perpendicular distance between two parallels 'p& q '.The area of different segments as shown in Fig. 4.5 has been marked as a, b, c.... 1, 2, 3... etc. can be computed as below:

Area = 0.5 (p+q)*x

In case segment is triangle, then for area calculation p or q can be substituted as zero.

Symbol 'to' stands for average thickness of overburden ordinates above potential failure plane excluding the extreme corner ordinates (i.e. zero values).

Similarly, symbol 'tr' stands for average thickness of rock ordinates above potential failure plane excluding the extreme corner ordinates (i.e. zero values).

Symbol 'Ao' stands for area of overburden portion above potential failure plane.

Similarly, symbol 'Ar' stands for area of rock portion above potential failure plane.

Symbol 'So' stands for saturation depth of overburden portion; in this study assumed as 0.25 (i.e. 25 % of average depth).

Similarly, symbol 'Sr' stands for saturation depth of rock portion; in this study assumed as 0.25 (i.e. 25 % of average depth).

Ur stands for uplift in rock portion, value taken as ($Ur= Sr*tr* l_r$)

Uo stands for uplift in overburden portion, value taken as ($Uo= 0.5*So*to* l_0$)

Symbol '∂o' stands for unit weight of overburden, value taken as (∂o = 2.3 tonnes/m^3)

Similarly, symbol '∂r' stands for unit weight of rock, value taken as (∂o = 2.3 tonnes/m^3).

Further, for the ease of computations, following parameters/coefficients have been utilized as below:

Let k = FOS (factor of safety)

& let $y = \sin(\beta+\theta) \tan\phi_r + k \cos(\beta+\theta)$

$a1= (1/y) \{ \sin\beta (k+0.5k\alpha_h + \alpha_h \tan\phi_r) + \cos\beta (k .\alpha_h - \tan\phi_r -0.5 \alpha_h \tan\phi_r)\}$

$a2 = (1/y) \tan\phi_r$

$a3 = (1/y) C_r$

$b1 = (1/y) \{\sin\beta (k+0.5k\alpha_h + \alpha_h \tan\phi_o) + \cos\beta (k.\alpha_h - \tan\phi_o -0.5 \alpha_h \tan\phi_o)\}$

$b2 = (1/y) \tan\phi_o$

$b3 = (1/y) C_o$

Where a1, a2, a3 ---- Coefficients of rock w.r.t. weight, uplift & contact length respectively (Table 4.1).

Similarly b1, b2, b3 ---- Coefficients of overburden w.r.t. weight, uplift & contact length respectively (Table 4.1).

Finally, T (i.e. anchorage force) found by following formula.

$$T = a1*Wr + a2*Ur - a3* l_r + b1*Wo + b2*Uo - b3* l_0$$

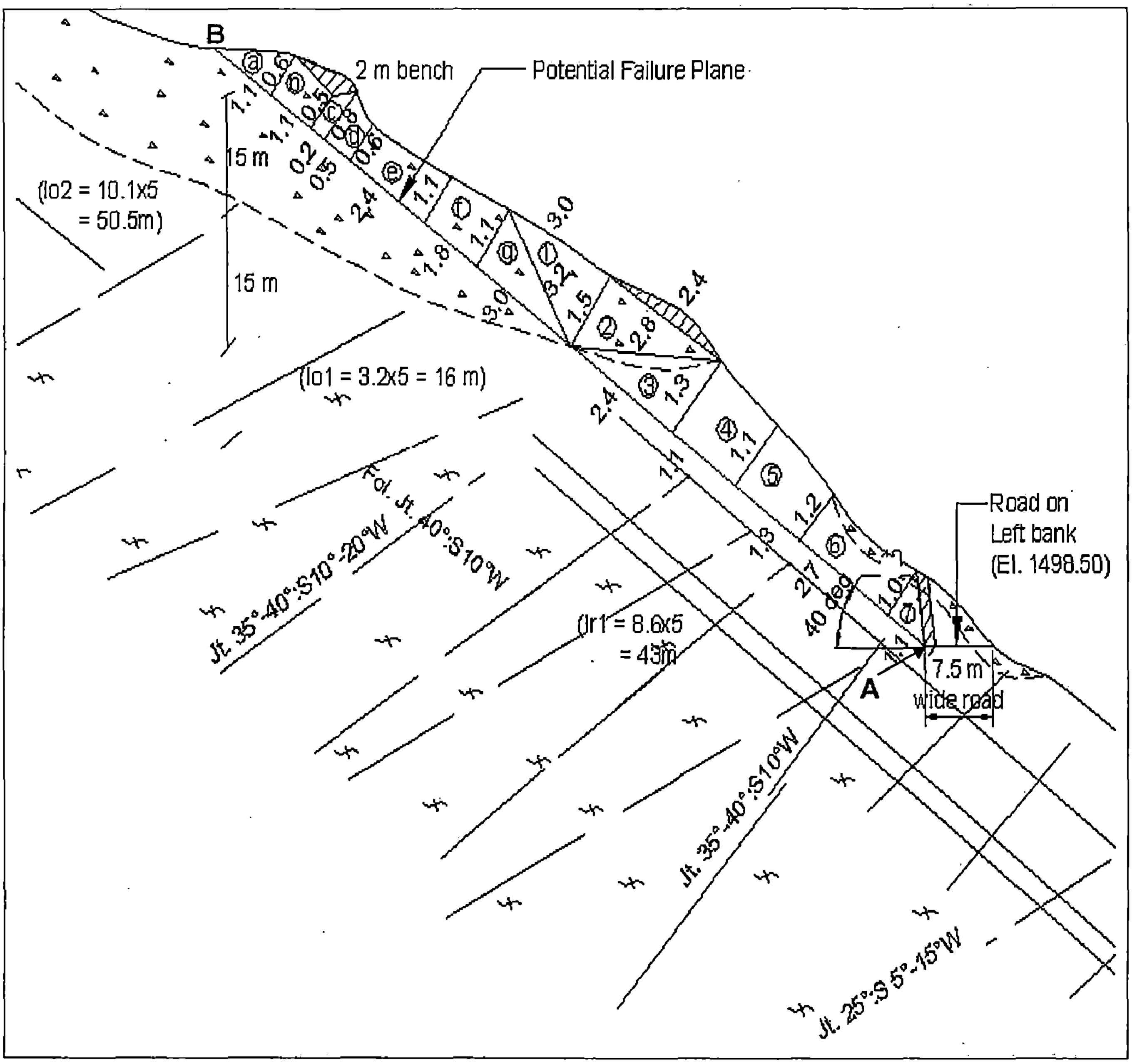


Fig. 4.5: Section at 75m d/s of dam axis showing different segments of overburden & rock for computational purpose (Left bank)

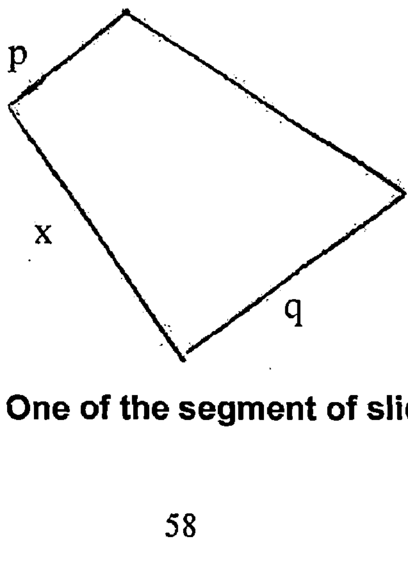


Fig. 4.6: One of the segment of sliding mass

Similarly, other computations have been done on different sections at the interval of 15m from 15m u/s to 105m d/s of dam axis. The location of failure plane is governed by deepest cut point. This cut point depends upon road and dam layout. At locations, where dam cut does not exist, only road cut exist (Fig.4.5 showing section at 75m d/s of dam axis). In some sections both road and dam cuts exist (Figs. 4.3 and 4.7). But in one situation failure plane due to dam foundation cut may be deeper w.r.t failure plane due to road cut (Fig. 4.3), whereas in another situation failure plane due to road cut may be deeper w.r.t failure plane due to dam foundation cut (Fig. 4.7). Keeping all situations in mind, required anchorage forces at different sections have been computed and shown in Table 4.2.

After computing anchorage force (T) required per meter width of sliding mass at each section the corresponding nos. of rows at each section has been calculated by using formula (iv) above. Total anchorage forces (in t/m) & nos. of rows of 200 tonnes at spacing of 3 m c/c capacity provided in the dam area at different sections for both seismic & non-seismic case has been computed and shown at Table 4.2 & Table 4.3. The critical values have been arrived through seismic case. Finally, 166 nos. of cable anchors of 200 tonnes capacity have been worked out against seismic case for left bank slopes at Nathpa Dam covering stretch of 120m from 15m u/s to 105m d/s of Dam Axis.

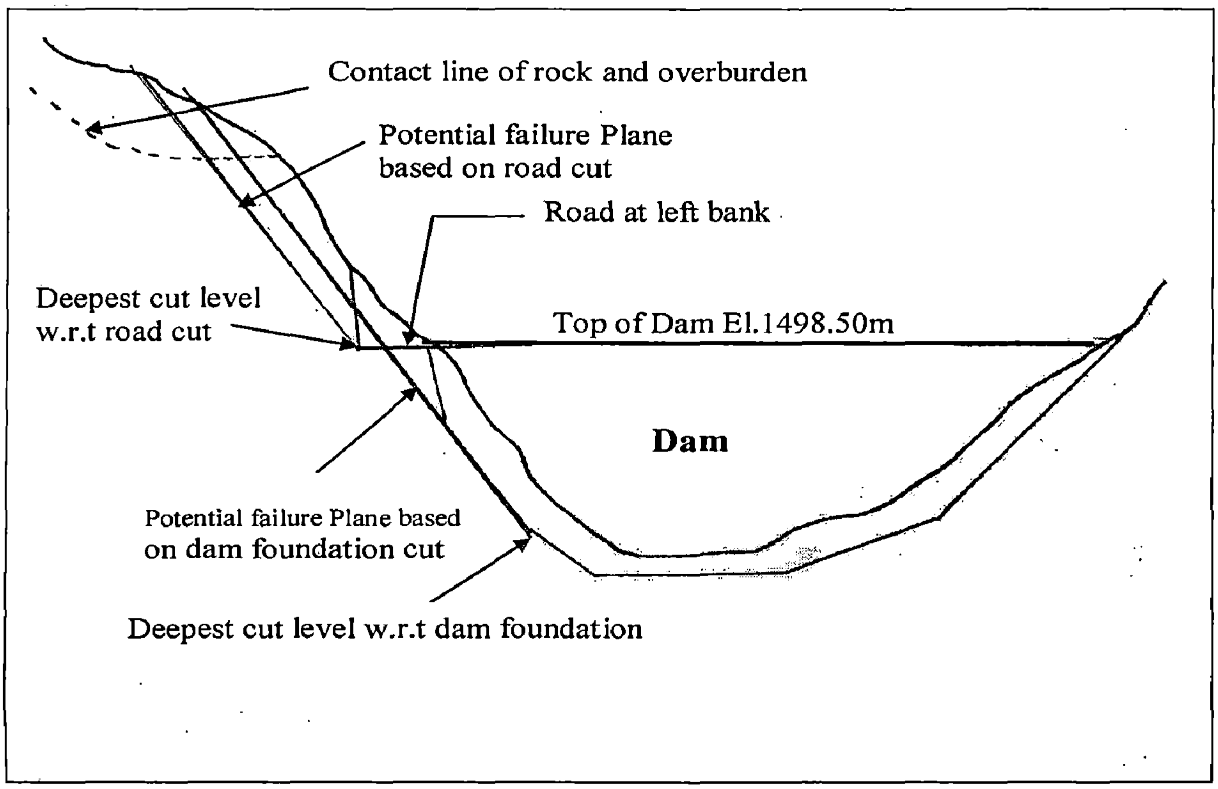


Fig. 4.7: Sketch showing location of different failure planes w.r.t cuts

Table 4.1: Section at 75 m d/s (Analysis w.r.t road cut) - Seismic case

Area= 0.5(p+q)*x

Overburden portion

Seg. No.a	p= 0.0	q= 0.6	x= 1.1	Area= 8.25
Seg. No.b	p= 0.6	q= 0.5	x= 1.1	Area= 15.13
Seg. No.c	p= 0.5	q= 0.8	x= 0.2	Area= 3.25
Seg. No.d	p= 0.8	q= 0.6	x= 0.5	Area= 8.75
Seg. No.e	p= 0.6	q= 1.1	x= 2.4	Area= 51.00
Seg. No.f	p= 1.1	q= 1.1	x= 1.8	Area= 49.50
Seg. No.g	p= 1.1	q= 0.0	x= 3.0	Area= 41.25
Seg. No.1	p= 0.0	q= 1.5	x= 3.0	Area= 56.25
Seg. No.2	p= 1.5	q= 0.0	x= 2.4	Area= 45.00
		to(m)= 4.43	**lo(m)= 50.50**	**Ao= 278.38**

Rock portion

Seg. No.3	p= 0.0	q= 1.3	x= 2.4	Area= 39.00
Seg. No.4	p= 1.3	q= 1.1	x= 1.1	Area= 33.00
Seg. No.5	p= 1.1	q= 1.2	x= 1.3	Area= 37.38
Seg. No.6	p= 1.2	q= 1.0	x= 2.7	Area= 74.25
Seg. No.7	p= 1.0	q= 0.0	x= 1.1	Area= 13.75
		tr(m)= 5.75	**lr(m)= 43**	**Ar= 197.38**

Let FOS=k

k= 1	∂r(t/cum)= 2.7	∂o(t/cum)= 2.3	αh= 0.08	
β(deg)= 40	β(rad)= 0.6984	Θ(deg)= 15	Θ(rad)= 0.262	
Ar(sqm)= 197.4	Wr(t/m)= 532.91	Φr(deg)= 41.9	Φr(rad)= 0.732	Cr(t/sqm)= 1
Sr= 0.25	tr(m)= 5.75	lr(m)= 43	Ur(t/m)= 61.81	
Ao(sqm)= 278.4	Wo(t/m)= 640.26	Φo(deg)= 41	Φo(rad)= 0.716	Co(t/sqm)= 0
So= 0.25	to(m)= 4.43	lo(m)= 50.5	Uo(t/m)= 27.96	

T(t/m) = a1*Wr+a2*Ur - a3*lr+b1*Wo+b2*Uo - b3*lo

Let y = sin(β+Θ).tanΦr+kcos(β+Θ)

p= 1.309
a1= 0.047
a2= 0.686
a3= 0.764
b1= 0.063
b2= 0.665
b3= 0.000

T(t/m)= 93.0828 t/m

No. of rows of 200T capacity @3m c/c = 1.4
= 2 (say)

Table 4.2: Total anchorage forces (in t/m) and No. of rows of 200 Tonnes at spacing of 3mc/c capacity at different sections

Sr No	Location of Section	Seismic case						Non-Seismic case					
		Analysis of section through Dam abutment		Analysis w.r.t road cut		Analysis between Dam foundation & Road cut		Analysis of section through Dam abutment		Analysis w.r.t road cut		Analysis between Dam foundation & Road cut	
		Anchorage force	No. of Rows	Anchorage force	No. of Rows	Anchorage force	No. of Rows	Anchorage force	No. of Rows	Anchorage force	No. of Rows	Anchorage force	No. of Rows
1	7.5m u/s	-	-	253.91	3.8	-	-	-	-	203.89	3.1	-	-
2	Dam Axis	395.26	5.9	308.04	4.6	-	-	317.74	4.8	246.70	3.7	-	-
3	15m d/s	-	-	316.36	4.7	20.30	0.3	-	-	255.34	3.8	10.90	0.2
4	30m d/s	-	-	298.78	4.5	16.78	0.3	-	-	241.20	3.6	7.44	0.1
5	45m d/s	-	-	309.97	4.6	36.48	0.5	-	-	250.30	3.8	21.26	0.3
6	60m d/s	-	-	198.95	3.0	104.80	1.6	-	-	159.61	2.4	75.77	1.1
7	75m d/s	-	-	93.08	1.4	-	-	-	-	53.93	0.8	-	-
8	90m d/s	-	-	78.88	1.2	-	-	-	-	46.58	0.7	-	-
9	105m d/s	-	-	85.14	1.3	-	-	-	-	49.02	0.7	-	-

Table 4.3: Total no. of prestressed cable anchors of 200 T capacity (Dam Area) (Seismic Case & Non-Seismic Case) - Results of the case study

Sr. No.	Location	Seismic Case No. of Anchors Above Road Level	Seismic Case No. of Anchors Below Road Level	Non-Seismic Case No. of Anchors Above Road Level	Non-Seismic Case No. of Anchors Below Road Level
1	**15 m u/s**	2		2	
2	12 m u/s	3		2	
3	9 m u/s	4		3	
4	6 m u/s	5		4	
5	3 m u/s	5	1	4	1
6	**0 Dam Axis**	5	1	4	1
7	3 m d/s	5	1	4	1
8	6 m d/s	5		4	
9	9 m d/s	5	1	4	1
10	12 m d/s	5		4	
11	**15 m d/s**	5	1	4	
12	18 m d/s	5		4	1
13	21 m d/s	5	1	4	
14	24 m d/s	5		4	
15	27 m d/s	5	1	4	1
16	**30 m d/s**	5		4	
17	33 m d/s	5	1	4	
18	36 m d/s	5		4	1
19	39 m d/s	5	1	4	
20	42 m d/s	5		4	
21	**45 m d/s**	5	1	4	1
22	48 m d/s	5		4	
23	51 m d/s	4	1	4	
24	54 m d/s	4	1	3	1
25	57 m d/s	3	1	3	
26	**60 m d/s**	3	2	3	2
27	63 m d/s	3	1	3	1
28	66 m d/s	3		2	
29	69 m d/s	2		2	
30	72 m d/s	2		1	
31	**75 m d/s**	2		1	
32	78 m d/s	2		1	
33	81 m d/s	2		1	
34	84 m d/s	2		1	
35	87 m d/s	2		1	
36	**90 m d/s**	2		1	
37	93 m d/s	2		1	
38	96 m d/s	2		1	
39	99 m d/s	2		1	
40	102 m d/s	2		1	
41	**105 m d/s**	2		1	
	Total	**150**	**16**	**115**	**12**
	G. Total		**166**		**127**

4.1.6.4 Fixed Length

The following equation is adopted for the purpose of determining fixed length.

Fixed length = $T_p.f_s/\pi d.C_s$ --- (viii)

Where T_p = Anchor proof load

f_s = Factor of safety

d = Diameter of hole

C_s = Allowable bond stress between grout & rock

<u>Computation of fixed length for 200 tonnes capacity anchor</u>

Working load of anchor (t) =200 t

Anchor proof load (T_p) = 1.1*200

=220 t

Diameter of hole (d) = 15 cm

Factor of safety (fs) = 2

Allowable bond stress between grout & rock = 10 kg/cm²

Substituting different values in equation (viii),

Fixed length = 934 cm

= 10m (say)

4.1.6.5 Free length

The minimum free length is based on pullout criteria for a group of anchors. However, a minimum free length of 5m or as worked out by pull out criteria, which ever is more, is considered desirable as specified in IS: 10270.

Minimum free length = $\sqrt{f_r.T_p/\gamma.b.\ \tan\omega}$ --- (ix)

Where

γ = unit weight of rock (2.7 t/m³)

b = Horizontal spacing of anchors (3m)

f_s = Factor of safety (2)

ω = angle of friction across fractures in rock (30°)

<u>Computation of free length for 200 tonnes capacity anchor</u>

Unit weight of rock	=	2.7 t/m³
Horizontal spacing of anchors	=	3m
Factor of safety	=	2
Angle of friction across fractures in rock	=	30°
Anchor proof load (T_p)	=	1.1*200
	=	220 t

Substituting different values in equation (ix),

Free length = 970 cm

= 10m (say)

4.1.7 Stabilization Measures

4.1.7.1 Right Bank

The slopes on right bank are more or less stable as bedding planes are dipping into the abutment. However, to stabilize certain joints dipping steeply towards the valley and for stabilization of talus deposited in the lower reaches of bank above road level, the portion from 50 m u/s to 80 m d/s of dam axis has been analyzed. The value of cohesion adopted as 2.94 t/m^2 and angle of internal friction angle as 40°. The slope of failure plane has been kept as 50° w.r.t. horizontal. Similar computations on the pattern of left bank have been done. Total 33 Nos. of 100 tonnes capacity prestressed cable anchors and 37 nos. of 40 tonnes capacity cable anchors alongwith grouted rock bolts have been provided on the right bank.

4.1.7.2 Left bank

On left bank in dam area portion from 15 m u/s to 105 m d/s of dam axis, about 166 nos. of prestressed cable anchors of 200 tonnes capacity have been provided based on anchorage forces computed w.r.t. different sections at 15 m interval (also indicated in the computational part). These have been installed at a minimum horizontal spacing of 3 meters in maximum five rows above road level and vertical spacing of 2 meters. Staggering of anchors has been done between adjacent rows. The installation was done in stages from top to bottom by constructing different benches and also incorporating dressing of overburden material, installing rock bolts of 32 dia, 8 m long in 3 rows before the excavation of first bench.

In the intake area, about 274 nos. of cable anchors of 200 T capacity have been installed in the stretch of 125 m u/s to 15 m u/s of dam axis.

Sketch showing elevation of prestressed cable anchors including rock bolts & drainage arrangement covering left bank dam area has been shown at Fig. 4.8.

Photographs showing ongoing installation works of 200 T capacity cable anchors & further view of prestressed cable anchors on left bank alongwith intake excavation in progress are shown at Fig. 4.9 and Fig. 4.10 respectively.

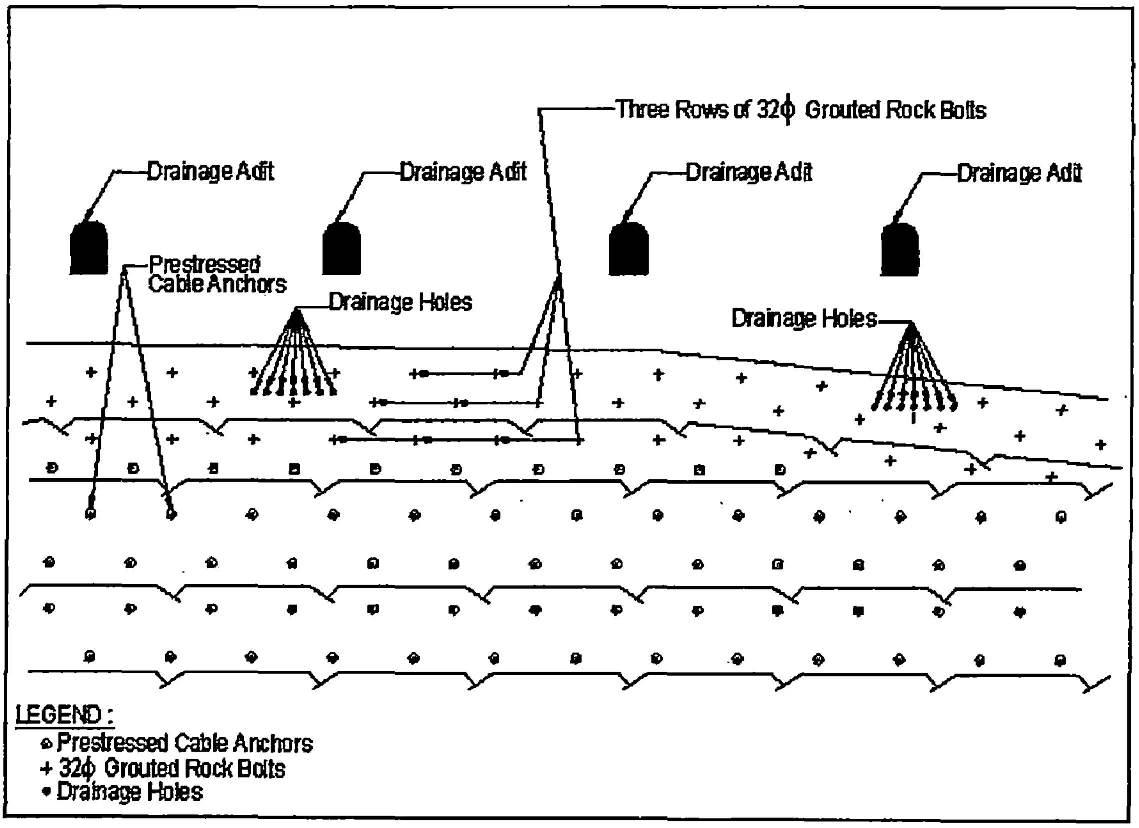


Fig 4.8: Sketch showing elevation of Prestressed Cable Anchors of 200 tonnes capacity including rock bolts and drainage arrangement (left bank dam area)

Fig 4.9: Photograph showing installation works of 200 T capacity cable anchors in progress

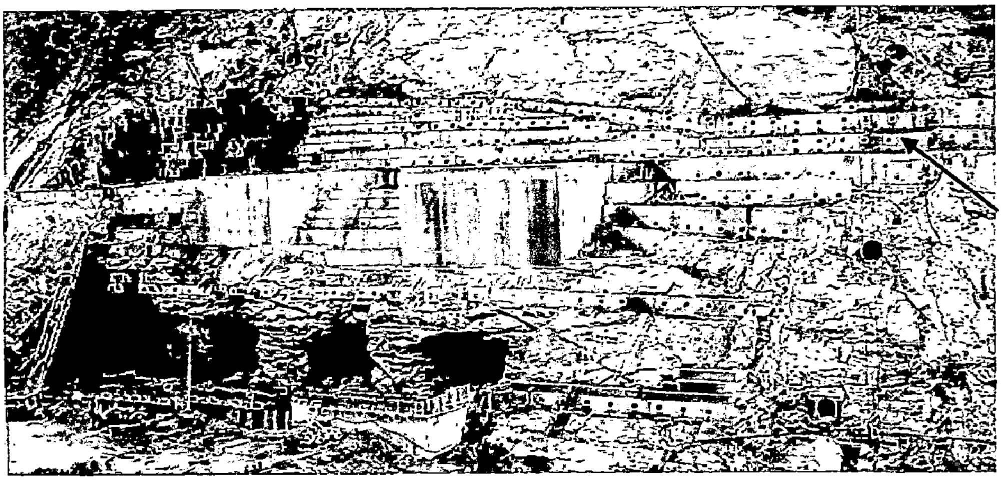


Fig 4.10: Photograph showing view of prestressed cable anchors with intake concreting in progress

4.1.7.3 Plunge Pool Area

In the plunge pool area, the slope stabilization computations have been done for both the banks from 90 m d/s to 150 m d/s of dam axis corresponding to a scour depth of 5 m in the rock as per model test results. Here the same values of C and Ø have been adopted as that adopted for left and right bank dam area computations respectively as mentioned above. The location of deepest cut for the failure plane on both the banks has been fixed on extreme corners of sluice blocks i.e. at the outermost location of jet, likely to strike on the river bed.

Before installation of cable anchors, the rock portion above cable anchors area has been stabilized by 3 rows of grouted rock bolts of 32 mm dia, 8m long @1.5m c/c alongwith 2 rows of drainage holes inclined 5° upwards, each hole having dia 77 mm, about 10m long, @ 3m c/c. In the plunge pool area, 55 nos. and 76 nos. of 200 T capacity cable anchors have been installed for the stabilization measures of right bank and left bank respectively covering area from 90 m to d/s of dam axis to 150 m d/s of dam axis. Also the bottom portion below cable anchors area on right bank has been supported by providing 3 rows of grouted rock bolts of 32 mm dia, 8m long @ 3 m c/c. Schematic sketch showing slope stabilization in plunge pool area has been shown at Fig 4.11.

4.1.8 Equipment

4.1.8.1 For Bench Excavation

For the installation of anchors, the heights of benches varying from 2m to 4m have been adopted. Track drills & jack hammers have been deployed for this purpose. The loosened rock mass has been removed with excavator and hauled to dump site with dumpers.

4.1.8.2 For Drilling Cable Anchor Holes

The three nos. Anchor Drill Rigs (KR 80302) with down the hole hammer have been used for the drilling of 150 mm diameter holes and these have been imported from IR-KLEMM Germany along with 3 casing handling cranes, 3 hydraulic pulling devices for pulling the casing, with sufficient number of 178mm dia casings, drill strings and other parts. The drilling has been carried out with hydraulic mechanism whereas flushing of holes has been done pneumatically alongwith water. The selection of drills was such that drilling through all types of rocks through rotary or rotary percussive method was possible. They have been used at the project site to drill holes at an inclination of 15^0 -20^0 with horizontal, upto 47m depth in rock and upto 30m depth in the soil and upto 4m height from base of machine by rotary percussive method. The RPM of drill string is selected after trials on a particular type of rock.

4.1.8.3 For Grouting

Grouting for consolidation of rock mass in the vicinity of bore hole and secondary grouting of anchor after stressing is carried out with the help of grout pump. A grout pump namely CHEM -GROUT (US make) was used. It is run by an electric motor of 7.5 BHP. It consists of two cylindrical drums viz. mixing drum and injection drum having 600mm dia each. Mixing drum is fitted with vane type blades for mixing and injection drum is provided with an agitator to keep the grout agitated while it is being pumped.

4.1.8.4 For Stressing of Anchors

For stressing the cable anchors, hydraulic jack namely ISMAL-4000MG provided by M/S Usha Martin Industries which is capable of stressing all the 12 cables of an anchor simultaneously has been used. The ram area and stroke length of the jack are 1025.70 cm^2 and 210 mm respectively. The load transferred by the jack is read on a dial gauge. After the strands are stressed, the individual strands are held by the cleaved wedges on top of stressing plate. These wedges are driven/pushed hydraulically once the desired load is imparted by the jack.

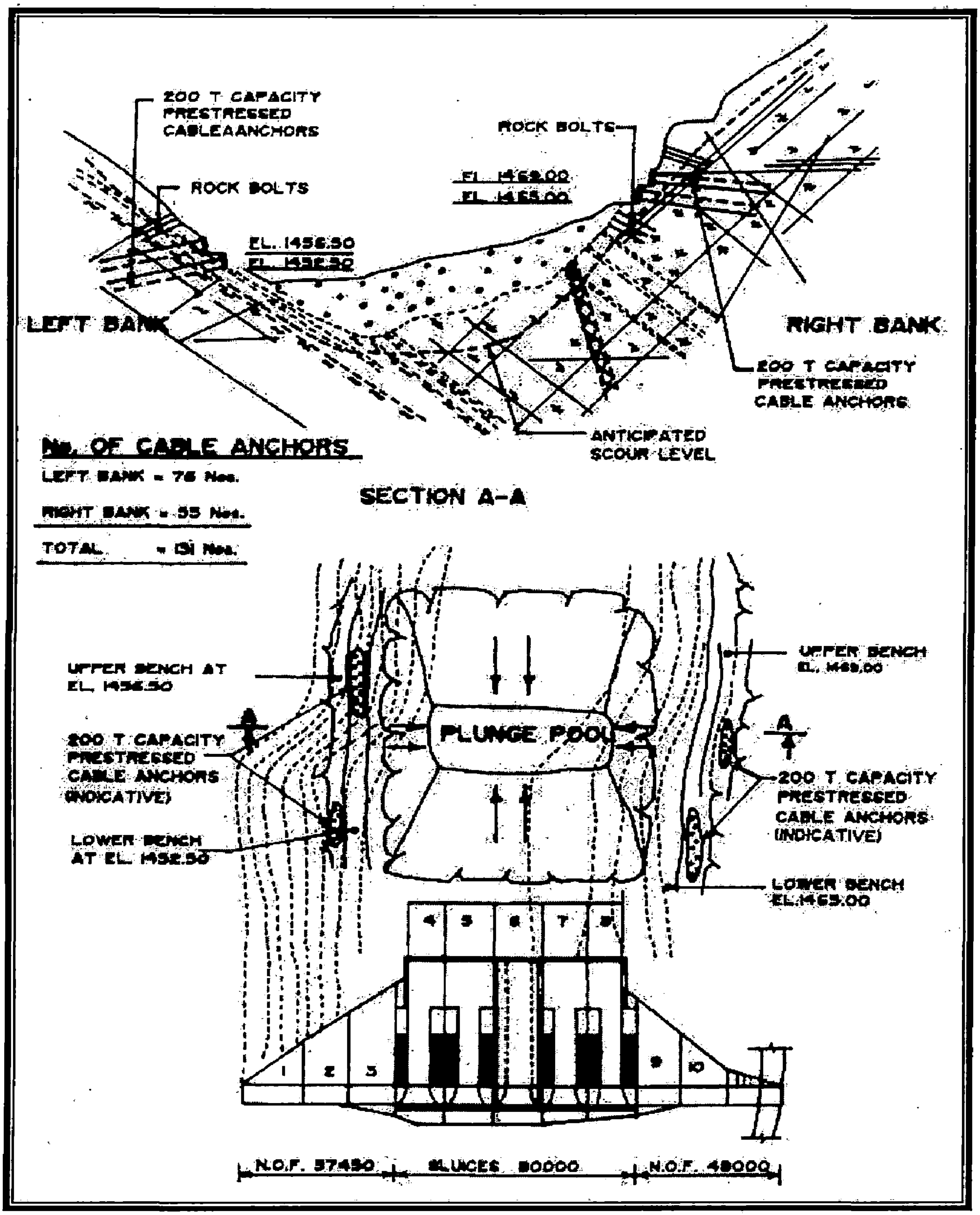


Fig 4.11: Schematic sketch showing slope stabilization in plunge pool area

4.1.9 Installation aspects of Cable Anchors and Allied Works

For the installation of cable anchors on left bank the following works have been proposed to be carried from top to bottom as shown in Fig. 4.12 and sequence as indicated below.

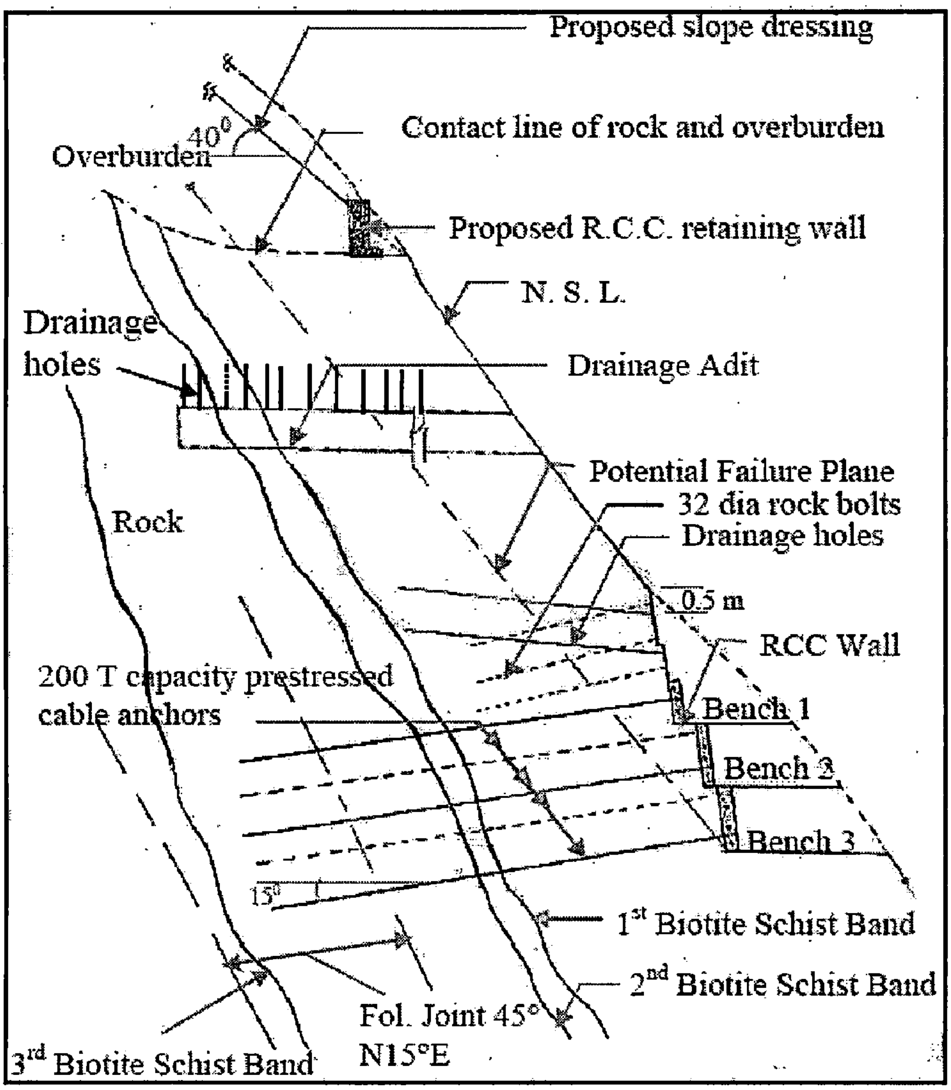


Fig 4.12: Sketch showing stabilization measures including installation of cable anchors

(i) Completion of slope dressing of overburden material.

(ii) Provision of R.C.C. retaining wall at the contact line of rock and overburden.

(iii) Provision of drainage adits (2.0m x 2.75m) at 25m c/c with drainage holes fanning on the top portion.

(iv) Installation of 3 rows of rock bolts of 32 mm dia, 8m long @ 2m c/c and 2 rows of drainage holes in upward direction on the top portion of first excavated bench.

(v) Excavation for first bench to a height of 3-4 meters.

(vi) Drilling of holes for cable anchors above this bench.

(vii) Preparation of R.C.C wall for placing anchor head.

(viii) Installation of cable anchors (details covered in succeeding paras).

(ix) After placement of first row of cable anchors, drilling series of drainage holes about 0.5m below the excavated bench.

(x) Similarly repeat steps (vi) to (viii) for next benches.

4.1.9.1 Technical specifications of cable anchors

The 200 T capacity prestressed cable anchors proposed to have 12 strands of high yield strength conforming to IS: 6006, including accessories such as anchors, studs, spacers, grout tubes, cement admixtures for grout, tensioning jacks etc. and materials required for primary grouting, stressing, secondary grouting , permanent corrosion protection of anchors. The IS:6006 covers the requirements for manufacture, supply and testing of uncoated, stress relieved high tensile steel strands for use in prestressed concrete. Each strand is of 15.2 mm diameter and comprised of 7 steel wires of 4-5 mm dia. It gives an ultimate breaking force of 314 T and working load of 200 T corresponds to 66% of ultimate load. For corrosion protection, 3 coats of epoxy formulations to high tensile steel wires as specified in IS: 10270 have been applied.

4.1.9.2 Steps involved in the instillation of anchors

The size of drilled hole adopted is 150 mm and length of 200 tonnes capacity anchors on left bank on dam area varied from 35 to 47 m deep into the rock. Before drilling for individual anchors, geology of strata & exact location of biotitic schist band has been ascertained with core recovery at every 30m interval both in vertical & horizontal direction. The total length of anchor covers free plus fixed length. The fixed length of 10m is kept beyond the second biotite schist band, where applicable. The holes to the required depth are drilled at 15^{O} inclination with horizontal & cleaned with water for atleast 10 minutes. The depth of each hole exceeds the total anchor length by 50 cm. The use of bentonite/grease or other lubricants on drill rods shall not be allowed. After drilling, consolidation grouting of hole is done till permeability values of 3 lugeons or less is achieved thus creating the water tightness of hole, which further will prevent the corrosion attack on anchor.

After drilling and grouting of holes all 12 strands are cleaned from dirt and dust and applying the three coats of epoxy treatment. Spacers, sheath pipe for grouting and clamping rings are provided at required spacing. Thereafter strands are inserted manually by a crew of about 25 persons, so that epoxy coating is not damaged. After homing of anchor, primary grouting for anchorage of the bottom 10m fixed length is carried out, with grout having water cement ratio as 0.4.

The cable anchors have been installed at 15^0 downwards from horizontal mainly due to grouting considerations. RCC wall about 70 cm width has been provided for transferring load from anchor head to the rock.

Details regarding 'Stressing of anchors' and of 'Secondary grouting of anchors and capping of anchor heads' adopted at Nathpa Jhakri Project are given in Appendix A. The main features are as below:

Stressing is carried out with the help of multiple jacks through which all 12 strands of anchors can be pulled simultaneously. The stressing is done when either the grout had achieved minimum cylinder strength of 25 MPa or 21 days had elapsed, whichever is earlier. But minimum strength of 25 MPa shall be ensured. Stressing is done in stages. Initially 20 tonnes load (i.e. 10% of working load) is applied and pressure released to zero to remove the slackness of strands. Thereafter load is applied again as 20 tonnes (i.e. 10% of working load), then 80 tonnes (40 % of working load), then 200 tonnes (100%working load), thereafter 80% of GUTS (Guaranteed Ultimate Tensile Strength) works out to be about 250 tonnes is applied with a gap of 5 minutes every time. Finally anchors are locked to proof load i.e. 220 T (this includes 10% long term losses).

The residual load in the anchor is checked after 3-5 days by lift-off test. Now, anchor is again pulled by stressing device and the load at which the stressing block has just lifted off is observed. Anchor is accepted only when residual load is more than 98% of proof load, otherwise stressing and lift off test is repeated till final acceptance.

In the free length zone of anchor, annular space between hole and strands is filled with cement grout i.e. secondary grouting is carried with water cement ratio as 0.4 for grout, required mainly for the corrosion protection of anchor. The bearing plate, anchor head and protruding strands are painted with bituminous paint. The painted anchor head is covered with centrally placed steel cap of size 3 mm thick, 310 mm dia & 110 mm deep welded on bearing plate (Fig. 4.13). Anchor head is also grouted with grout of w/c ratio as 0.4. Finally, cap is also painted with bituminous black paint. Details of prestressed cable anchor are shown in Fig 4.13.

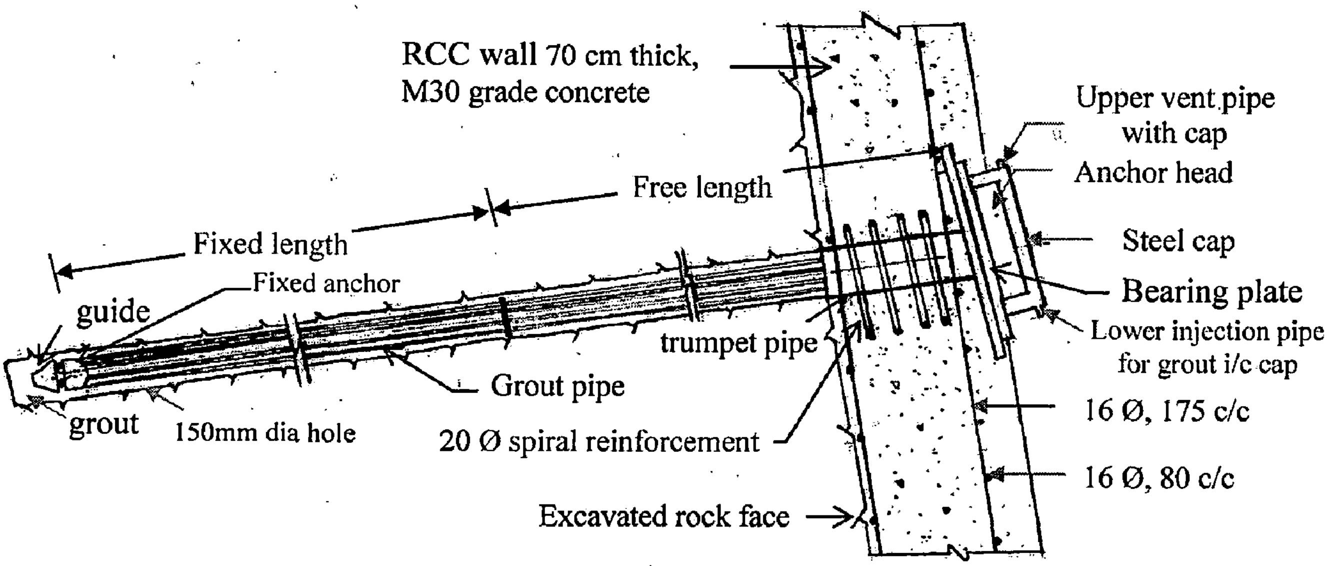


Fig 4.13: Sketch showing prestressed cable anchor including anchor head details

4.1.10 Stressing observations pertaining to anchors

The procedure adopted for 'Stressing and monitoring of anchors' for the slope stabilization works at Nathpa Jhakri Hydro-Electric Project has been given in Appendix A. The Specimen Tables covering details of 'Load Chart for Anchors', 'Calculations of Elongation', 'Anchor Stressing Record' and 'Restressing & Monitoring Record' are shown at Tables A-1 to A-4. After the perusal of record pertaining to anchor stressing maintained at project site, the variations in the elongation values have been noticed between computed and actual observed.

The most of the deviations lies within 10% and anchors in general are overstressed. The variations in elongation values for few anchors have exceeded more than 10%. The some part of deviation may be accounted to observational errors, variation in actual area of individual strands and the variation due to actual modulus of elasticity values of steel material used. But overall performance can be rated as satisfactory.

The lift off test has been done for all the anchors after 3-5 days of stressing the anchors. If the residual anchor load is more than 98% of the proof load the anchor is accepted. In case loss in stress is encountered during the test then the anchor is restressed to proof load till final acceptance.

The stress gauge readings (in kg/cm²) observed at the time of locking of anchors (i.e. observed at proof load) are noted and compared with the readings taken at the time of restressing of anchors. Some loss of stress observed in few anchors has become stable after 1st restressing of anchors to proof load. Also in some cases 2nd restressing has also been done just to note down whether any loss in stress has occurred or not. At 2nd restressing stage the results are all right i.e. no losses in stresses have been observed.

4.1.11 Construction aspects

Corrosion protection, stressing and grouting of anchors are the important aspects on which success of anchors depend. Their technique and monitoring during construction are important. These are descried in detail in Appendix B.

4.1.12 Slopes behaviour during construction and operation stage

The excavation of slopes on left bank in the reaches of the dam and Power intake was main concern for the safety and construction of these components due to foliation joints dipping towards the valley. The excavation works of dam and intake foundations on left bank have commenced only after stabilization measures using prestressed anchors have been provided. During construction stage no untoward incidence has been reported pertaining to slopes.

The anchors have been installed during October, 1996 – March, 1999. The dam and intake structures after their construction have further added stability to existing slopes. Also the slopes adjoining to plunge pool area and right bank are stable till date.

All units have been commissioned during the period October, 2003 – May, 2004. The behaviour of slopes during operation stage is also satisfactory. Hence installations of anchors with other measures adopted for slope stabilization in Nathpa Jhakri Project have ensured adequate safety to slopes and adjoining structures.

4.2 CASE STUDY NO. 2 (ICHARI DAM)

4.2.1 General

In this case study, details of prestressed anchors for stability of left training wall comprising of thin section and stabilization of Ichari Dam left bank slopes in the upstream of dam axis have been covered.

The area in the vicinity of left training wall consists of inter-bedded quartzitic–slates and slates. Further slates are soft and thinly bedded and dipping towards the river. Due to adverse geological features it was decided to provide a thin section of RCC training wall with anchors because other alternative comprising full gravity section needed a lot of excavation, which would have destabilized the slopes.

The left bank slopes in the stretch of 140 m u/s of dam axis comprises of thick overburden of hill wash material and debris and further underlying rock has a number of shear zones and glide cracks dipping gently towards the river. These adverse features would have further created problem in the stability of slopes in the operational stage due to the impact of impounding of the reservoir and its daily drawdown because of peaking at Chibro Power house. Hence this necessitated to take the extensive stabilization measures by providing anchors.

4.2.2 Salient features and layout of project

The 59.25 m high straight gravity concrete dam is a diversion dam, located at Ichari on river Tons (a tributary of river Yamuna) in Uttaranchal State. The dam is located about 72 km upstream of Kalsi alongwith intake and sediment exclusion arrangements to divert the water into a 6.2 km long power tunnel which feeds an underground power house at Chibro utilizing a drop of 124 m. The water coming out from the power house can either be discharged into river Tons or fed through tunnel into the part II of the scheme. The total length of dam at top is about 155m with seven spilling bays, each of 9.5 m clear openings separated by 3m thick piers. The Salient Features of Project are as below:-

a) Concrete Diversion Dam at Ichari

River	Tons
Location	Ichari
Height	59.25 m
Live storage	5.1 million m^3
Tainter gates	7 Nos. 9.5 X 16.5 m

b) Power Tunnel

Length	6.2 km
Diameter	7.0 m
Capacity	235 cumecs

c) Pressure shafts

Nos. and diameter	4 nos. each of dia 3.81 m

d) Surge Tank (Underground)

Type	Restricted orifice type
Height	100 m
Diameter	20 m

e) Powerhouse (Underground)

Installed Capacity	240 MW
Size of powerhouse cavity	113 X 18.2 X 32.5 m
Turbine	4 X 60 MW, Francis
Avg. estimated generation in the year	900 million units

The layout of Yamuna Hydel Scheme is shown at Fig. 4.14 and Photograph showing view of Ichari Dam showing different components is shown at Fig. 4.15. The project was completed in 1975.

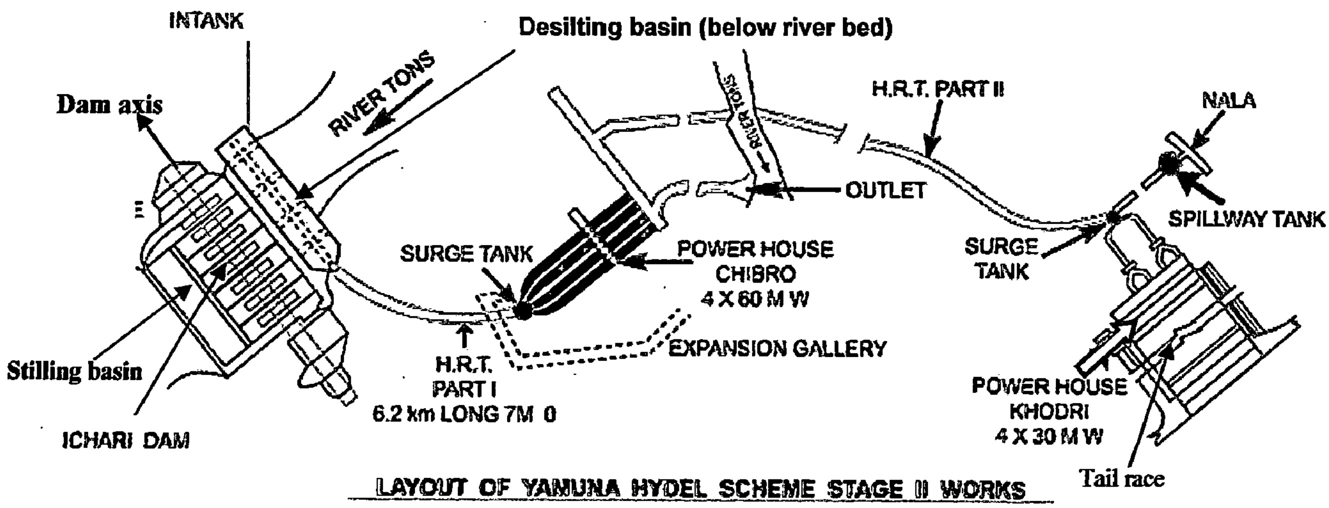


Fig. 4.14: Yamuna Hydel Scheme Stage II

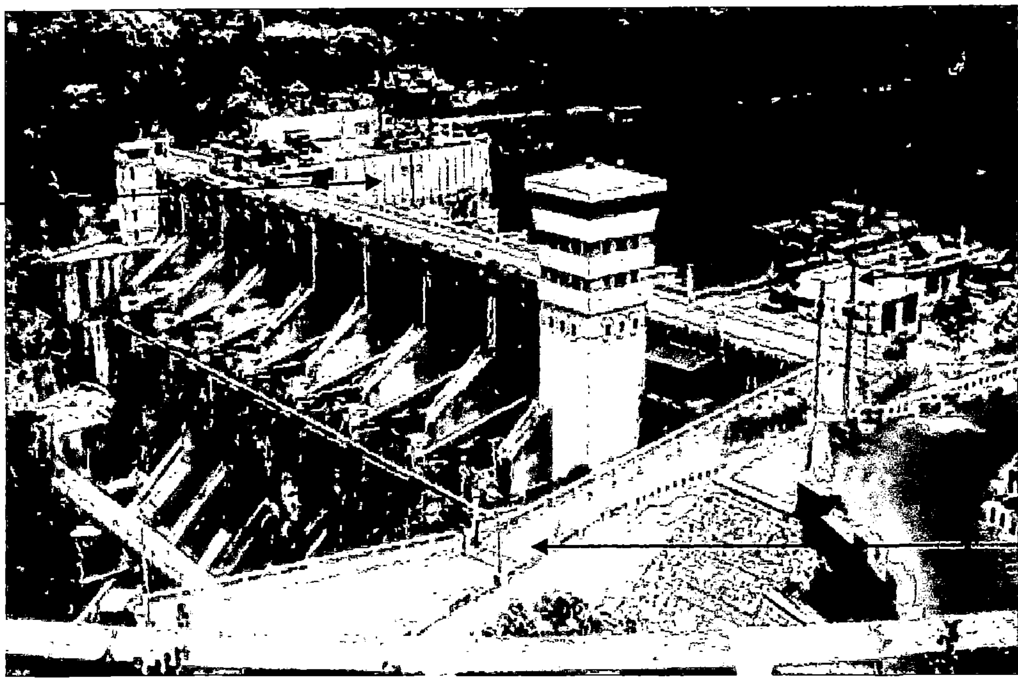


Fig. 4.15: Ichari Dam showing different components

4.2.3 Left Training Wall

The training wall along the left flank of the spillway at Ichari comprise a relatively thin concrete section held in position against over-turning and sliding with the help of positive prestressed anchors. The radius of curvature of training wall has been kept as 140 m extending 35 m upstream of the toe of the roller bucket. The general layout plan of Ichari Dam showing training wall has been shown at Fig. 4.16.

4.2.3.1 Geology

The bed and left bank rock consists of inter-bedded quartzitic – slates and slates. The slates are soft and thinly bedded having the bedding parting at an interval of less than even 1 cm. The rocks are dipping at an angle of 10^0 to 15^0 towards the river. Above El. 618.00 approx. the rock consists of many shear zones and glide cracks dipping towards the river. The geological details of the left bank slopes are shown in Fig. 4.17.

Right bank is of good rock with stable slopes requiring no stabilization and treatment. Hence the intake is located on right bank whereas HRT takes off from left bank.

4.2.3.2 Earlier Proposal

In the early study before finalizing the positive anchors in left training wall, the following possibilities were exploded. In one case it was proposed to provide a wall with full gravity section & in another case thin section with anchors was proposed. By providing full gravity section, a lot of excavation was involved. By providing full gravity section further instability problem was anticipated due to deep cuts in the rock slopes which would have involved very heavy cost.

Therefore, it was decided to provide a thin section of RCC training wall with anchors. Hence the proposal of a retaining wall with a bottom width of 4.0 metres only anchored to the rock with pretensioned anchors was examined. The rock face was to be cut in a slope of 1 in 8. The top of the retaining wall was kept at El. 617.00 m which corresponded to a flood discharge of about 2200 cumecs, same as for the right training wall. Also a parapet retaining wall has been proposed upto El.620.00 which corresponds to a flood discharge of 3500 cumecs and which is the normal high flood encountered at the dam site. This top level has been considered satisfactory for guiding the normal high flood. For higher floods the rock face has been proposed to be protected by 0.5m thick concrete panels 3m wide and 1.5m high secured with anchor bolt arrangement upto El. 636.00. The bottom of the training wall has been kept at the general foundation level of spillway i.e. El. 603.50.

The adopted section was made safe in overturning and sliding by anchoring the training wall to the rock mass with four pretensioned anchors of 50 tonnes capacity each spaced at 2 meters intervals at an inclination of approximately 60^0 to the back face of wall. The length of the anchors has been kept as 12.5 metres with 5.0 metres positive anchorage at the end. Ordinary anchors of 36 mm @ 2 metres c/c in bottom have also been provided. Two rows of ordinary anchors of 40 mm @ 2 metres and 8 metres deep along and across the flow have also been provided in sides to provide further factor of safety.

In brief, the earlier arrangement on left training wall consists of following as shown in drawing at Fig. 4.18.

i) Construction of left training wall upto El. 617.00 with four rows of pretensioned anchors of 50 tonnes capacity, 12.5m long, spaced @ 2m c/c both ways.

ii) Ordinary anchors of 36 mm @ 2 metres c/c in the bottom portion.

iii) Two rows of ordinary anchors of 40 mm @ 2metres and 8 metres deep as shown.

iv) Provision of parapet retaining wall from El. 617.00 to El. 620.00 m.

v) Protection of rock face treatment against maximum design flood in downstream side upto El. 636.00 by providing 0.5m thick face concrete in panels of 3 metre width and 1.5m height with gaps of 75 mm filled with porous concrete to ensure free drainage.

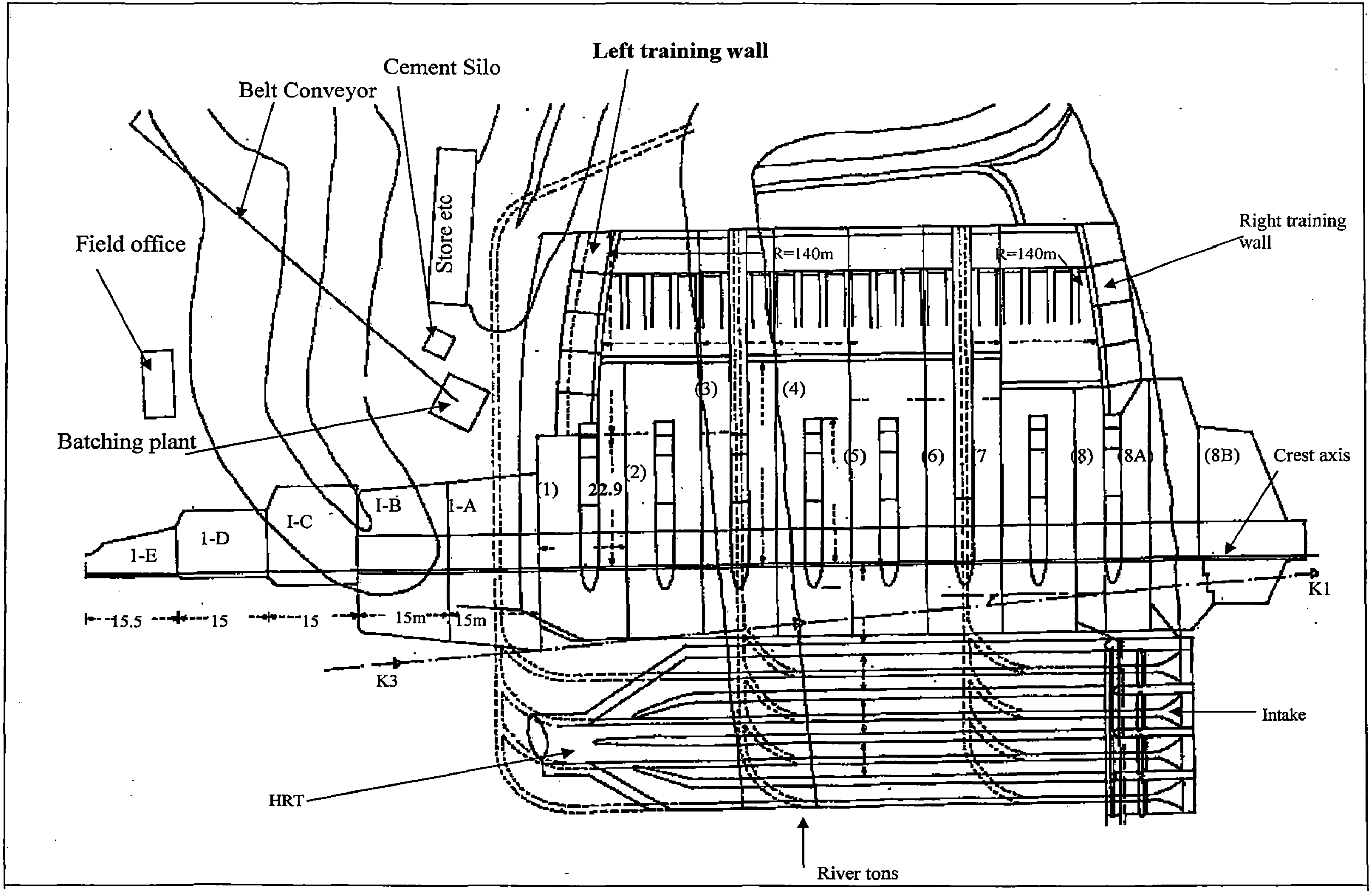


Fig. 4.16: General layout plan showing training wall (Ichari Dam)

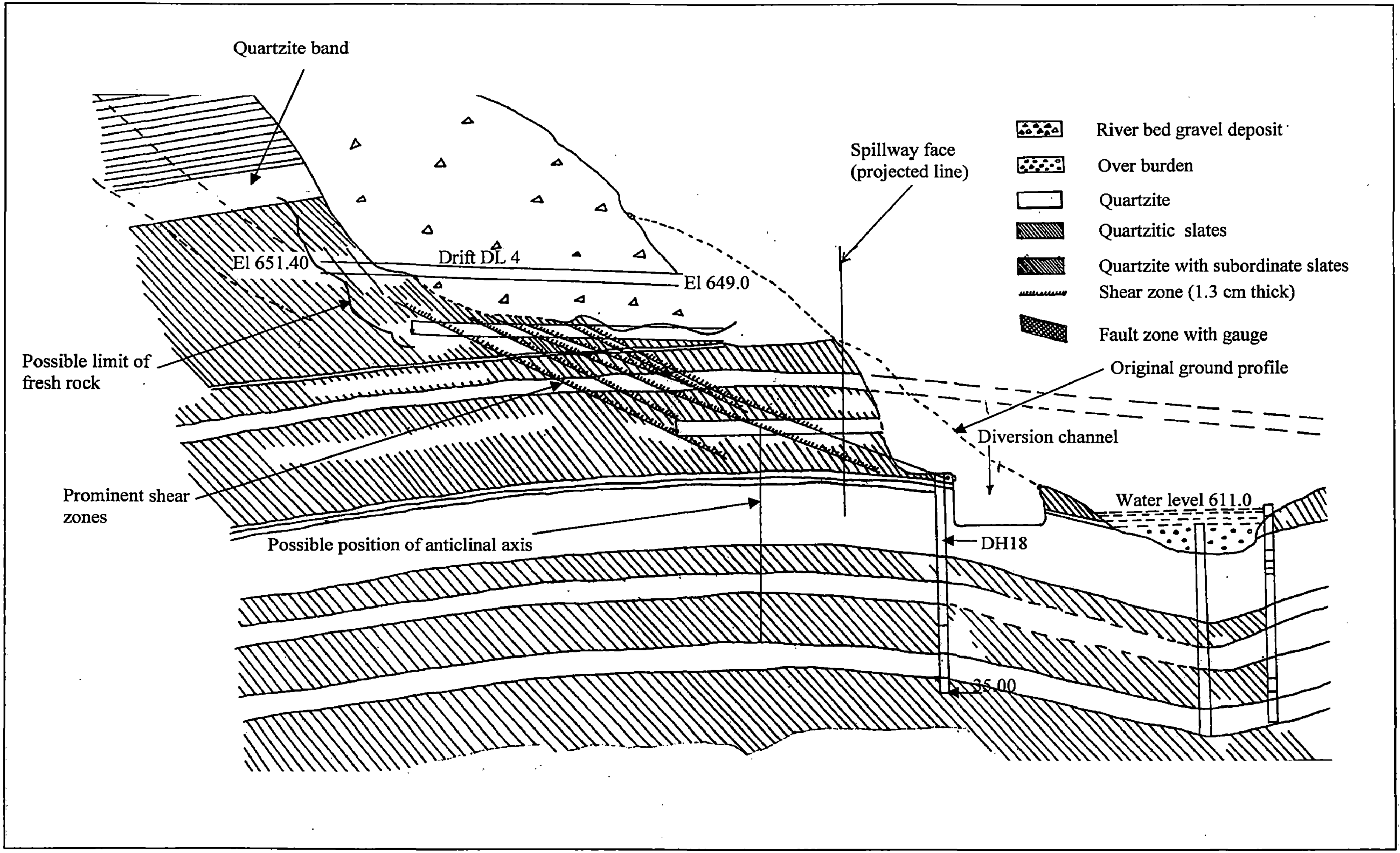


Fig. 4.17: Geological details of the left bank slopes

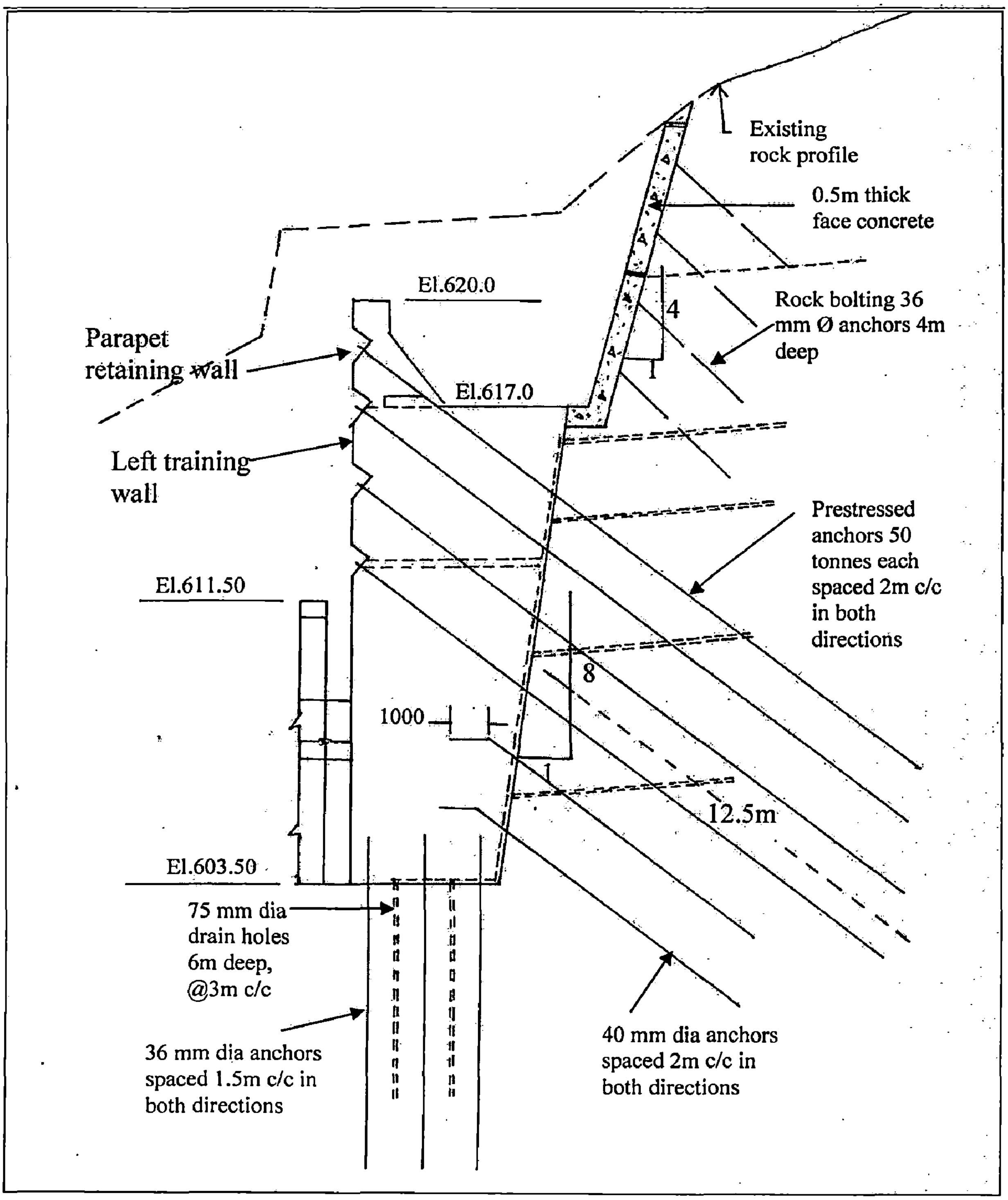


Fig. 4.18: Section showing pretensioned anchors of 50 tonnes capacity (Earlier proposal)

4.2.3.3 Revised Proposal Incorporating Positive Pre-Tensioned Anchors

In earlier proposal, for the 50 tonnes anchors it was decided to grout the inner end (fixed length) of anchors. It was proposed that 5 meter length out of 12.5m length will be grouted. The anchors were inclined approximately 60^0 to the back face of the wall.

Later on, due to certain site constraints it was decided to replace grouted pre-tensioned anchors with positive prestressed anchors similar to those installed in the underground power house and other cavities at Chibro. In the case of positive prestressed anchors, both ends of anchors are accessible and stressing can be done from both ends.

Whereas in case of pre-tensioned anchors, only one end is accessible and farthest end lies deep into the rock mass. Here stressing is done from the end lying on the slope surface after grouting of the fixed length.

The following factors were considered for the revised proposal:

i) When new proposal came into picture, the face was available within rock mass (i.e. excavation of the tunnel for the third flushing conduit was in progress) where positive anchorage can be conveniently provided.

ii) By providing positive anchors, there is no element of uncertainty in getting good performance, as both the terminals of the anchors are approachable and anchor plates can be fixed as required.

iii) Complete know how and resources were available at work site in case of positive anchors, as about 500 such anchors in the power house cavity at Chibro were already installed.

iv) In the case of grouted anchors, the expertise for installation is required which might not be available during that period i.e. during seventies.

The revised proposal for the positive anchorages to be installed through the tunnel of the third flushing conduit is shown in the drawing at Fig. 4.19.

Therefore, finally positive pre-tensioned anchors have been provided instead of the grouted pre-tensioned anchors proposed earlier.

4.2.4 Stabilization of Ichari Dam left bank slopes – upstream of dam axis

The study deals with the problem of stabilizing the hill slopes on the left bank upstream of Ichari Dam. The left bank slopes in the stretch of 140 m u/s of dam axis were found instable due to the impact of impounding of the reservoir and its daily drawdown because of peaking at Chibro Power house. The problem was created due to existence of a thick mantle of overburden material lying over the rock. The sound rock face is available about 85 meters behind this mantle of overburden material. Overall aim has been to evaluate the type of treatment necessary for the stabilization of left bank hill slopes to ensure its stability after the filling of reservoir.

Due to presence of thick mantle of overburden, there has been problem of tying the dam to the left bank rock and hence not considered.

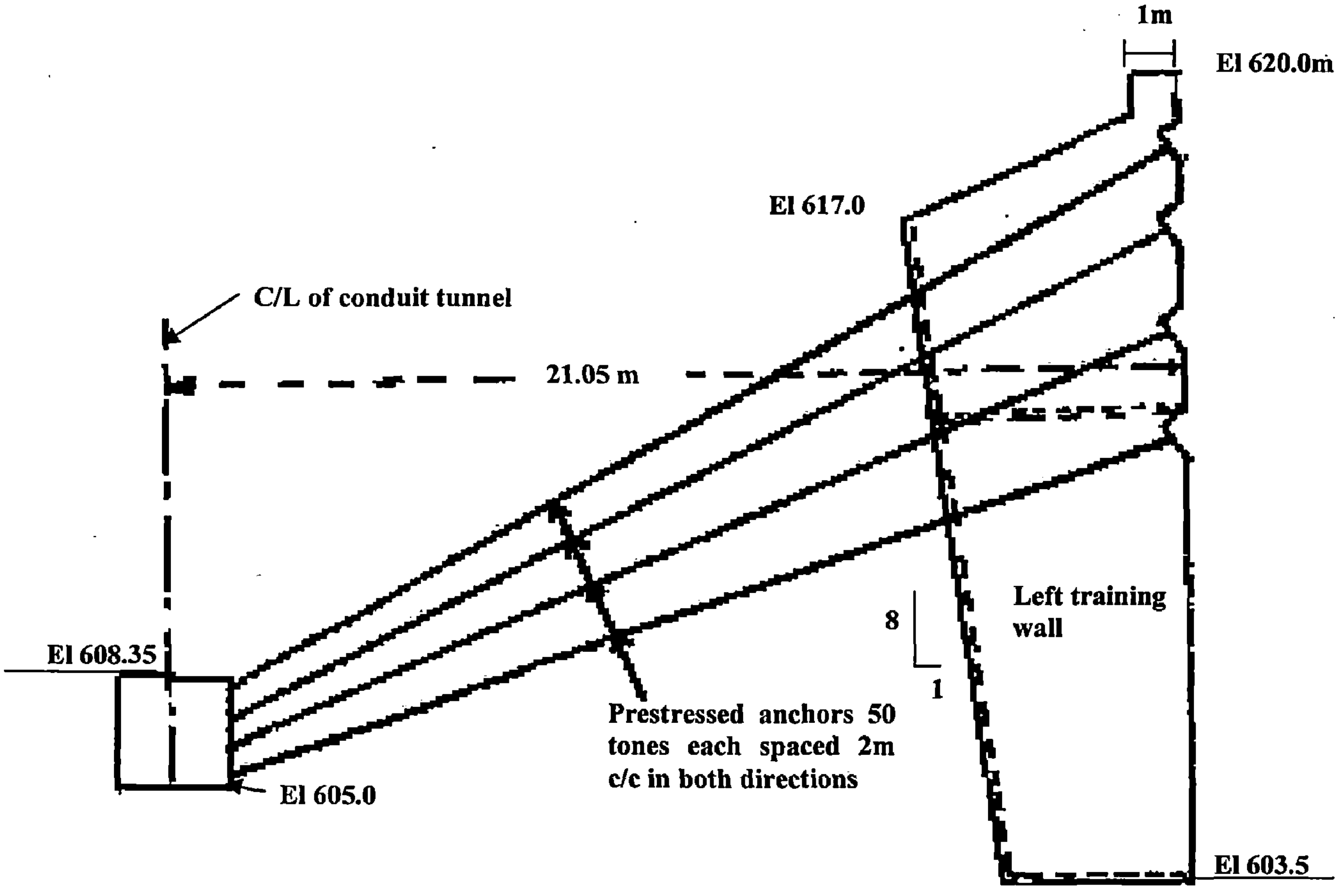


Fig. 4.19: Section showing positive pre-tensioned anchors (Revised proposal)

4.2.4.1 Geology

The geological section at 70 m u/s of dam axis also covering details of anchors has been shown at Fig.4.20. There is a thick overburden of hill wash material and debris standing approximately at an angle of 37^0. This overburden material is overlying a rock ledge at an approximate El. 636.00, which rises steeply behind it. The underlying rock has a number of shear zones and glide cracks dipping gently towards the river. The left bank formation has been roughly divided into the following zones as shown in Fig. 4.20.

Zone – I: Comprising of hill wash material and debris

Zone – II: Highly fragmented and weathered rock mass

Zones-III: Zone of fresh but fractured and jointed rocks

Zone – IV: Zone of sound and fresh rock

A crushed rock seem has also been observed at an approximate El. 618.00. The mantle of overburden extends up to about 140 m upstream of the axis where an exposed rock ledge is visible. For this reason slopes upto 140 m u/s of dam axis have been considered for treatment.

4.2.4.2 Measures for Left Bank Stability

In order to make the slopes more stable, the following was considered

(i) Reduction in actuating forces

(ii) Increase in resisting forces

(i) Reduction in actuating forces:

(a) Easing of slopes

The actuating (disturbing) forces can be reduced by reducing the mass of earth at higher elevations which contributes maximum towards the instability of slope. But this can be done to a very limited extent, because overburden material is providing the toe support to the rock mass lying behind having number of secondary joints and cracks. Any large scale removal of overburden will result in the instability of the steep hill slopes behind, which may result in progressive sliding. Therefore easing of slopes to the minimum possible extent was adopted.

(b) Drainage improvement:

The actuating forces can also be reduced by reducing the extent of build up of pore pressures in the hill mass. This can be achieved by providing:

(i) 75 mm thick asphaltic carpet on the berms at El. 680.00m and also covering small portion below it.

(ii) Sealing effectively all the open glide cracks and joints on the hill face at upper levels through which water may seep into the hill mass.

(iii) Pucca surface drains along the slopes and along the berms may be provided to afford quick flow of surface water without percolation in the soil below.

(iv) Drilling series of drainage holes at different locations through the drifts above El. 645.00 and filling these holes with gravel and other filter material and left as such to afford quick drainage.

(v) 1.0 meter thick filter may be provided behind the toe wall.

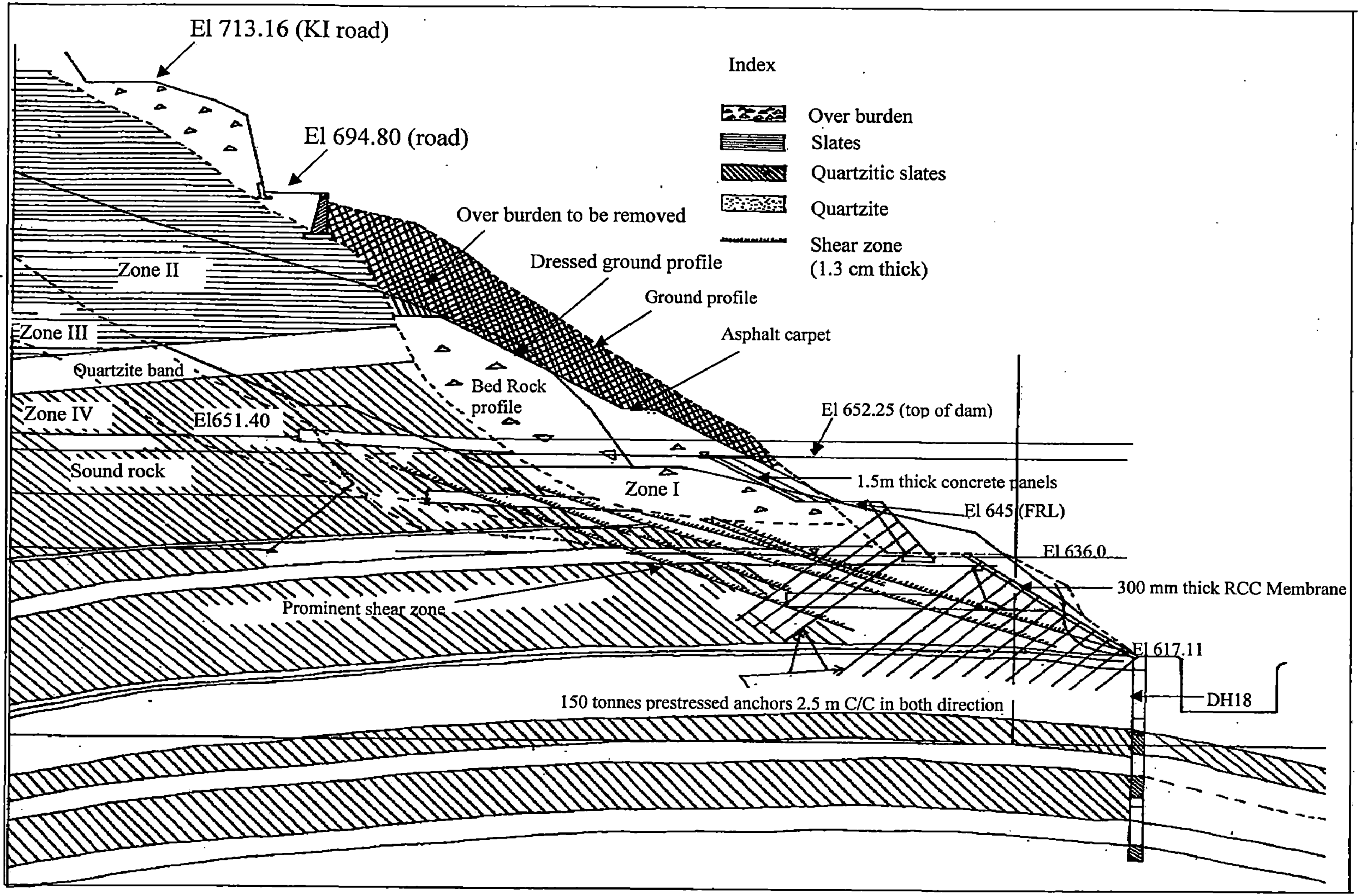


Fig. 4.20: Geological section also covering details of anchors at 70 m u/s of dam axis

(ii) Increase in resisting forces

In order to increase the resisting forces, two alternatives as discussed below were considered.

Alternative I – Prestressed Anchors

It comprises of providing a reinforced concrete toe wall from El 636.00 to El 645.0 varying in thickness from 3.0 m at the bottom to 1.5 m at the top, held down by prestressed anchors and providing requisite number of prestressed anchors from El. 636.0 to El. 618.0 through the rock mass and butting against a R.C.C. membrane 300 mm thick on the rock face (Fig. 4.20).

The prestressed anchors have been assumed to introduce a positive resisting force in the rock mass which reduces the actuating force. The following advantages can be achieved by using Prestressed Anchors:

(i) When aim is to stabilize rock mass, a better solution lies in preventing the movement rather than catering for the support to the rock mass in which sliding tendency has already been initiated.

(ii) The prestressed anchors have been proposed to spread over about 40 meters at spacing of 2.5 meters along the slope and along the length perpendicular to dam axis. As consolidation grouting will be done during anchor installation, grouting will consolidate a big chunk of rock mass at the toe of the hill slope, which will act as a support for the hill mass.

(iii) The blasting can be prevented in already sheared rock mass.

The following disadvantages are also to be considered while selecting prestressed anchors.

(i) In the anchors, reliability of maintaining the prestress over long period and through overburden material is questionable.

(ii) There are chances for stress corrosion in the prestressing steel even after grouting.

Alternative II - Concrete Anchors

In this alternative, the following has been covered.

(i) A concrete toe wall based on the rock ledge at El 636.00 and varying in thickness of 10 meters at the bottom to 3.0 meters at top.

(ii) Reinforced cement concrete anchors constructed in situ. Plan and typical section showing this arrangement are shown at Fig. 4.21 and Fig. 4.22 respectively.

The concrete anchors have to be necessarily provided in a manner that they are as close to each other as possible so that the uncertain effect of variable yield rock in between the anchors is minimized or eliminated. The concrete anchors has also to be in the region of the toe of the rock formation and at the same time so disposed that sufficient rock cover is available behind the anchors towards the river side.

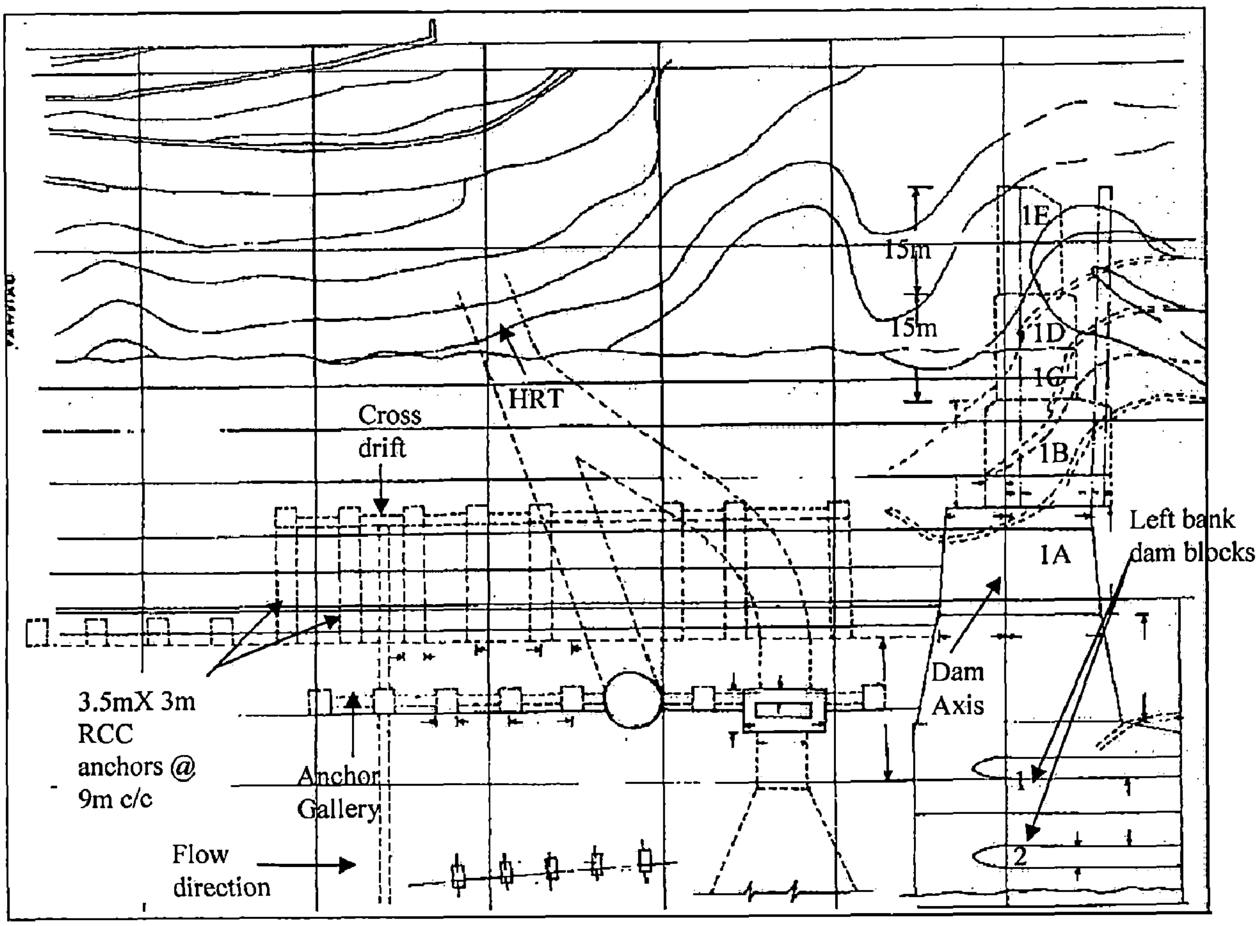


Fig. 4.21: Plan showing Concrete Anchors (Ichari Dam area)

Keeping these factors in view a system of anchors in a tandem has been proposed in which one anchor is inclined at an angle of 30^0 with the vertical while the other anchor is vertical.

In addition, provision of a mechanical anchorage has also been made by a cross drift (Fig.4.21) filled with reinforced concrete. This would also help in more uniform distribution of stress near the toe of anchors which is a very desirable requirement. The drift will be very useful during construction for drainage and speedy excavation of the shafts.

The size of the shafts has been kept as 3 m x 3.5 m each and they are provided at 9 meter centre to centre staggered. This arrangement has been made keeping in view the feasibility of construction, adequate rock cover in between the adjacent shafts and providing maximum coverage along the flank to resist the sliding forces as uniformly as possible.

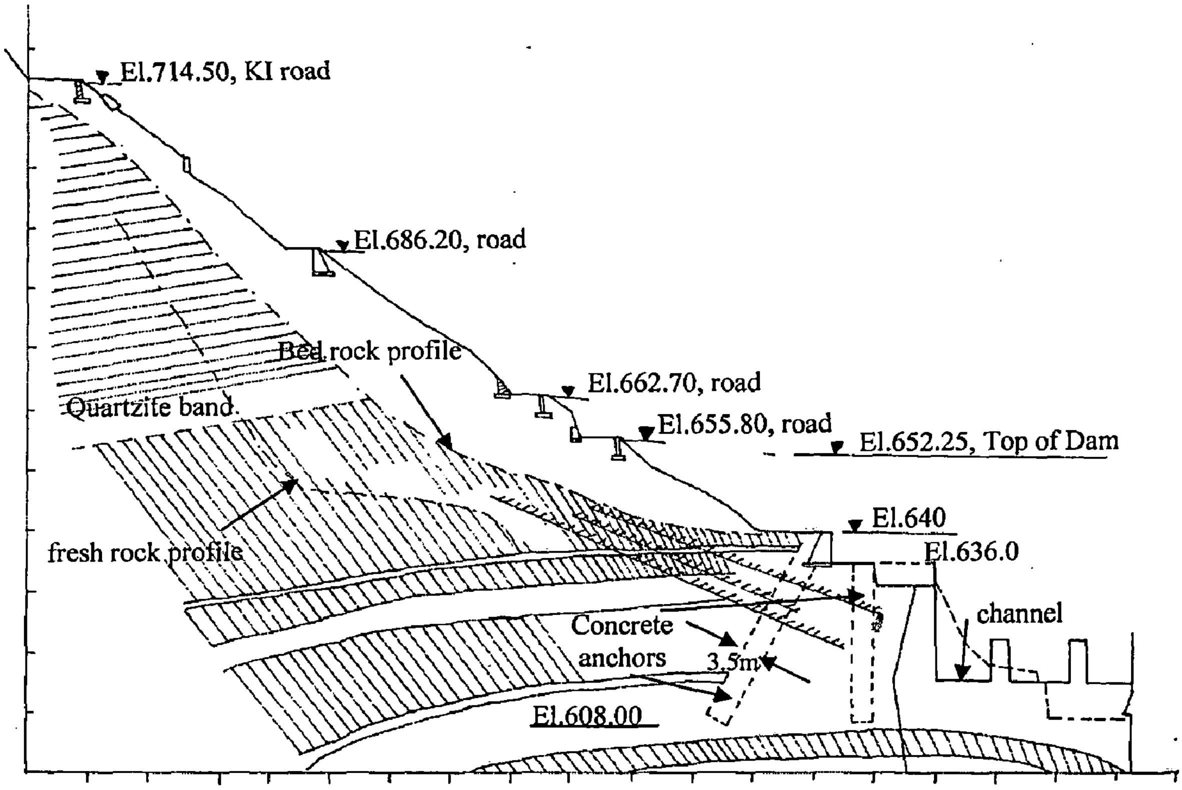


Fig. 4.22: Typical section showing Concrete Anchors

The shear resistance is provided partly by concrete and partly by reinforcement in the anchors. The reinforcement has been provided so as to limit the size of section for underground excavation and at the same time keeping the reinforcement limited to a degree that no practical difficulty is experienced in placing reinforcement and concrete in the excavation shafts.

This proposal of constructing R.C.C. anchors was adjudged in the light of the following points.

Advantages:

The strength of R.C.C. anchors is fully known and there is no factor of uncertainty in the same. These can be constructed at the site with know how available.

Disadvantages:

The following can be the possible disadvantages of this proposal:

(i) Construction of these anchors involved large scale but controlled blasting in the already fractured and sheared rocks of the left bank.

(ii) The concrete anchors would come into play only when the rock mass tends to slide where as the prestressed anchors provide a constant toe support.

Finally, for the protection of slopes, alternative – II comprising concrete anchors was adopted because probably during that period (1969-70), an adequate know how for prestressed anchors was not available. But now a days due to advancement in anchor technology and reliable techniques available for the corrosion protection of anchors has changed the scenario. By using prestressed anchors the resisting force is induced in advance in the rock mass, whereas in case of concrete anchors the same comes into play when there is tendency to slide.

4.2.5 Behaviour of Slopes

The geologically adverse slopes in the vicinity of Ichari Dam has been treated with anchors in conjunction other stabilization measures. The thin section of training wall in the downstream of dam axis has been stabilised with positive prestressed anchors and portion upstream of dam axis with concrete anchors during seventies. Till now, the slopes are functioning well and structures adjoining to slopes are safe.

4.3 CASE STUDY NO. 3 (DHARASU POWER HOUSE)

Protection of Slopes adjoining to u/s Wall of Dharasu Power House (Prestressed Anchors of 100 Tonnes Capacity)

4.3.1 General

Maneri Bhalli Hydro Electric Project Stage-II is a Hydel scheme (304 MW) for generating 1642 MU during average year by harnessing the water drop of 285 meters between Uttarkashi and back water of Tehri Dam Project near Dharasu. The river Bhagirathi which is called the Ganga in the lower reaches is a snow fed river originating from the glacial regions of the Himalayas. Plan showing Ganga Ghati Projects incorporating Maneri Bhalli Hydro Electric Project Stage-II is shown at Fig. 4.23.

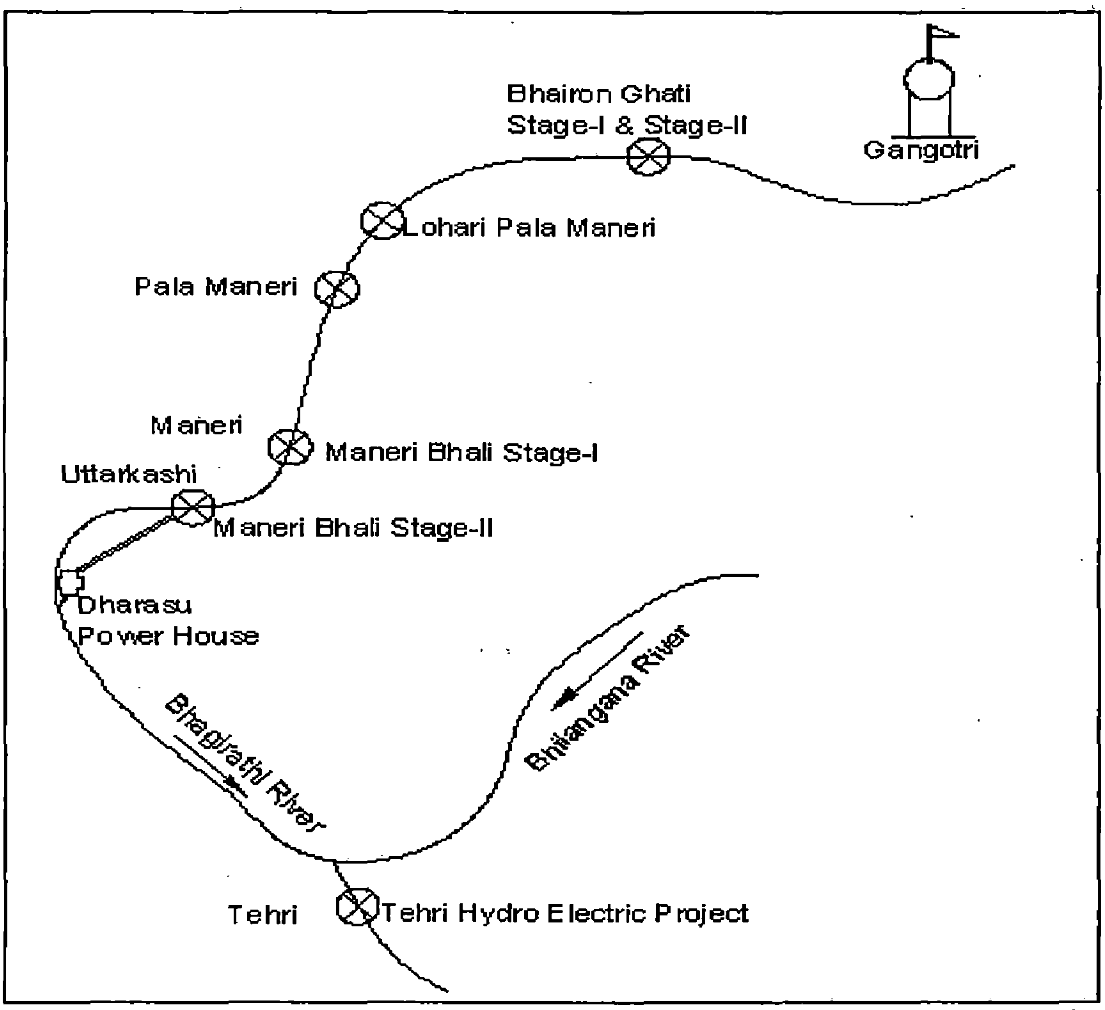


Fig. 4.23: Plan showing Ganga Ghati Projects

The salient features of Maneri Bhali Stage-II Project (Installed Capacity 304 MW) are as below:-

Geographical location	In Uttaranchal State (District Uttarkashi)
Location of Barrage	At 1 Km down stream of Joshiyara steel bridge and about 2.5 Kms D/s of confluence of tailrace channel of power - house of stage-I
Hydrology ---	
Catchment area at Barrage Site	4416 Sq Km
Snow Catchment area above 12000 ft.	3199 Sq Km
90 % Available discharge	21.4 cumec
70 % Available discharge	31.0 cumec
Design Flood ---	
For Hydraulic design	5000 cumec
For Over toppling	8000 cumec
Barrage	81m long
Intake	56m long
Sedimentation Chamber	93m x 182m
Forebay	D/S of Chamber -gated
Cut & Cover	Between forebay & Tunnel portal –43m long
Power Tunnel	16.0 Km long, 6.0m dia horse shoe
Surge Tank	13.72m dia, 172m high
Penstocks	3.0m dia –4 no. Each 800m long
Power House	Situated near Dharasu on left bank of river Bhagirthi
Gross Head	285m
Net Head at 142 cum	250m
Installed Capacity	304 MW (76MW x 4)
Type of Machines	Vertical Shaft Francis
Units generation target	1642 MU

Dharasu Power House is situated in a pit cut in rock. The general ground level was between E1. 860 to 892 m and the deepest foundation level of Power house is 809 m. Phyllite and Greywacke are the main rock types in the power house area upto El. 855 m in unit 4 and Unit 1, 2 & 3 respectively and above that is river borne material. During the excavation work for Dharasu Power House and Appurtenant works in its first stage consisted of removal of River Borne Material (RBM) from El 892.00 to El 855.00. Thereafter the rock excavation in the Power House pit was further done upto El 815.00. At this stage, because of increase in size of Power house shifting of upstream boundary of the power house towards the hill side was proposed. Because of this the earlier proposed berms at El 826.00 and EL 838.00 m were eliminated.

Certainly, the upstream slopes upto El 855.00 m became very steep due to increase in the size of the Power House. Further, at this stage of work, the dressing of the slopes was not possible for improving the stability of slopes. Also before treatment of slopes, heavy rock slide occurred on u/s side of unit 4 as well as on gabble end side in July, 1985 after heavy rains. At the initial stage of work, stone pitching in sufficient length in RBM portion was done along with 4-5 m deep 2-3m c/c grouted anchors with surface shotcreted with chain link fabric. But the same couldn't withstand.

Due to above circumstances, it was decided to provide prestressed anchors on the u/s side of power house pit to improve the stability of slopes.

The Plan showing different units of Dharasu Power House and view of Dharasu Power House & adjoining hill slopes with treatment are shown at Fig. 4.24 and Fig. 4.25 respectively.

4.3.2 Design Parameters & Criteria for prestressed anchors

4.3.2.1 Phyllite Rock

Before establishing design criteria, the geological details of Phyllite rock have been studied in detail. In unit no. 4 mainly Phyllite rock consists of thinly foliated rock mass of poorer grade quality, puckered, weak and totally shattered and properties matching with soil mass. So, in this reach it has been decided to adopt the failure criteria as slip circle, based on criteria similar to as adopted normally for soil mass. The excavation in this zone was not possible, as overlying river borne material already stands treated in the form of dressing of slopes covering top portion with boulder pitching. In view of this, it has been decided to provide prestressed anchors covering the fixed length zone beyond critical slip circle established after different trials.

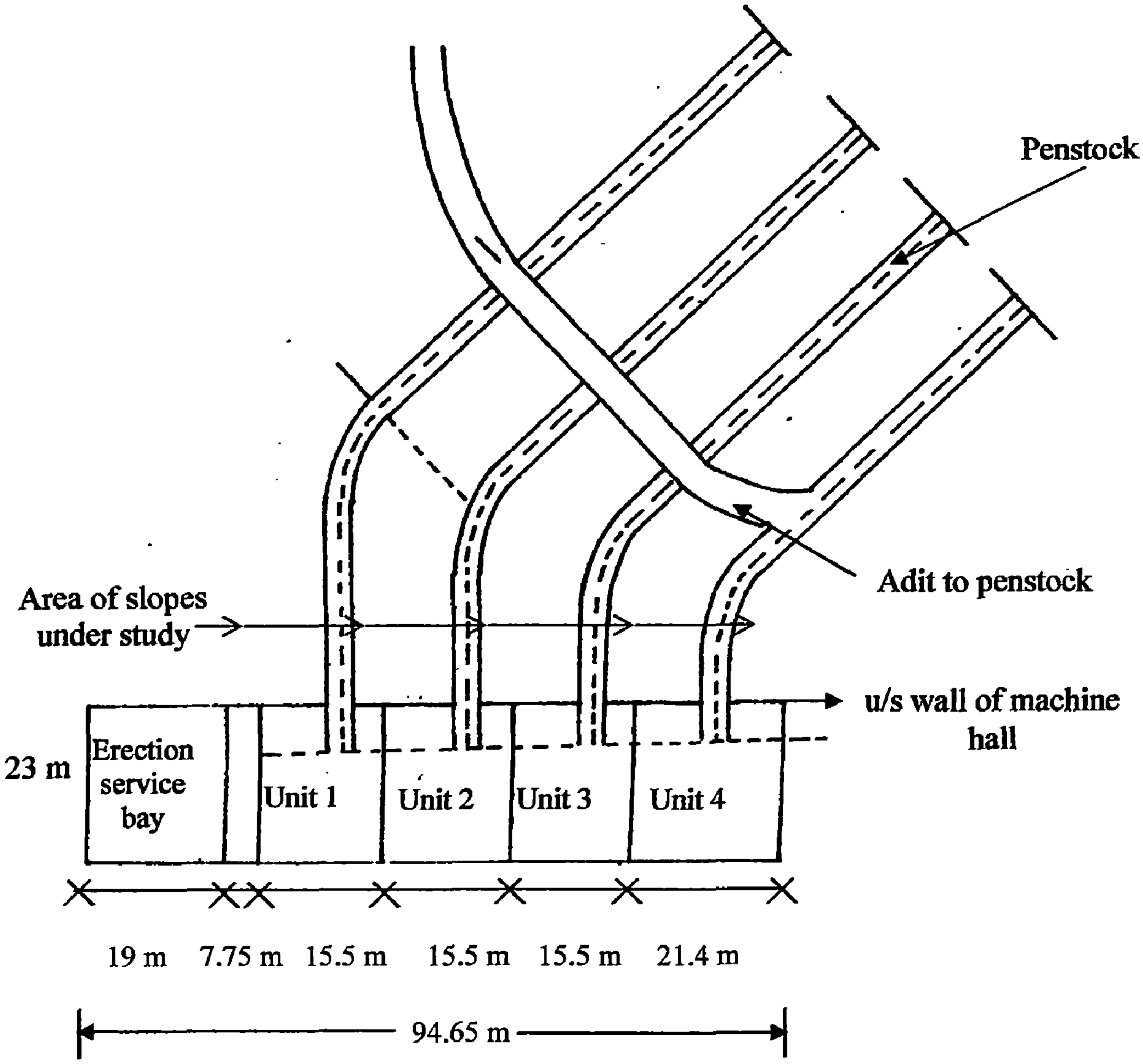


Fig. 4.24: Plan showing different units of Dharasu Power House

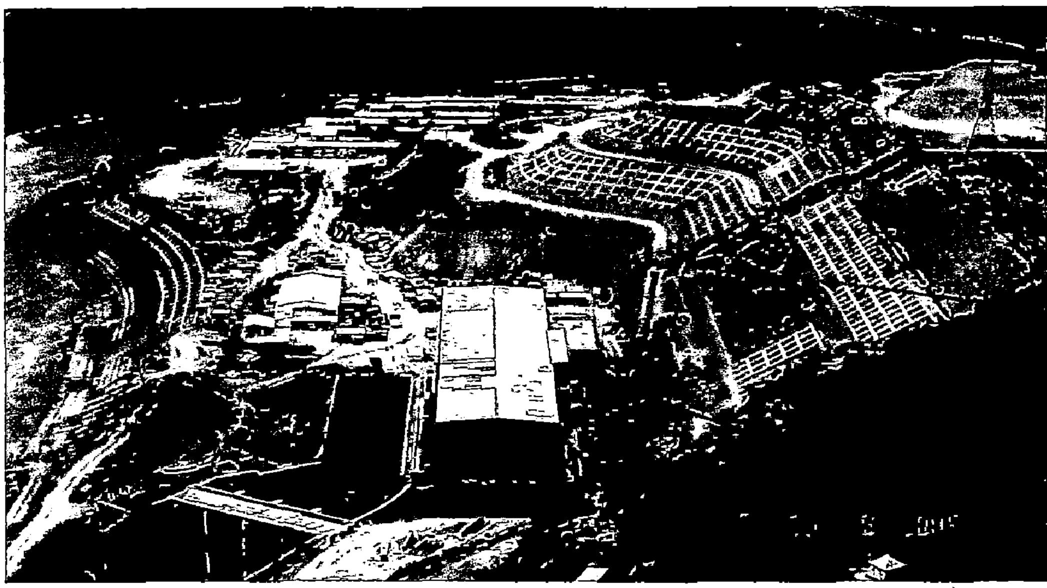


Fig. 4.25: View of Dharasu Power House and adjoining hill slopes with treatment

The following values of cohesion (c) and angle of internal friction of material have been found by conducting in situ rock to rock test under saturated conditions in unit 4.

$C = 0.60 \text{ kg/cm}^2$

$\phi = 36.0^0$

Also, density of rock

(i) Max = 1.91 gm/cm^3

(ii) Min = 1.49 gm/cm^3

The stability of slope is tested under two conditions so that no pressure comes on Power House.

(i) Max. water level of 836.5m at the toe of slope and the whole slope mass saturated.

(ii) Earthquake condition - Normal water level at toe of slope being 830.00 and whole slope mass is saturated.

Also pull out test results in Phyllite rock on u/s slope of Power House in unit 4 were conducted. The abstract of test result is as below:

Test No.	Bond strength kg/cm^2	Remarks
1.	1.86	Bond failure
2.	6.56	Bond failure
3.	7.40	Bond failure

Test No.2 and 3 seems to be similar. A safety factor of 2.5 has been taken for Phyllite rock since Phyllite, is soft, thinly foliated, puckered, sheared and breciated whereas Greywacke is hard and highly jointed, and gives an average bond strength of 2.80 kg/cm^2. A value of 2.75 kg/cm^2 has been considered.

Method of analysis: The slope stability analysis is carried out by Swedish or slip circle method (IS: 7894).

The safety factor against sliding for the assumed failure surface is computed by the equation.

$$\text{FOS} = \frac{\Sigma S}{\Sigma T} = \frac{\Sigma[C + (N - U)\tan\phi]}{\Sigma W \sin\alpha}$$

Where

FOS = factor of safety

S = resisting or stabilizing force

T = driving or actuating force.

C $= c \times \dfrac{b}{\cos\alpha}$

N = force normal to the arch of slice

U = pore water pressure

ϕ = angle of shearing resistance

W = weight of the slice

α = angle made by the radius of the failure surface with the vertical at the centre of slice

c = unit cohesion

b = Width of slice.

The factor of safety for earthquake condition shall be worked out from the following formula.

$$\text{FOS} = \frac{\Sigma C + (N - U)\tan\phi - \Sigma(W_1 \sin\alpha \tan\phi \times AH)}{\Sigma W \sin\alpha + \Sigma W_1 \cos\alpha \, AH}$$

W_1 = Saturated weight of slice

AH = Horizontal seismic coefficient (adopted as 0.15g)

Safety factors: The following minimum desired values of factor of safety shall be achieved in the analysis as per IS: 7894.

Case	Loading condition	Minimum desired factor of safety.
1.	Reservoir full with steady seepage	1.5
2.	Earthquake condition with reservoir full or steady seepage	1.0

Various trails made with different slope angles and finally at a slope angle of 29^0 with Phyllite and R.B.M, the safety factor were found out to be 1.61 in normal condition and 1.04 m in seismic condition towards u/s side slope. The safety factor with slope angle of 27^0 with Phyllite rock only towards gable end worked out to be 1.51 in normal condition and 1.06 in seismic condition which is within permissible limits.

4.3.2.2 Greywacke Rock

The C & ϕ values of Greywacke rock have been tested under saturated condition in the field and values are as below:

$C = 1.30 \text{ kg/cm}^2$

$\phi = 41^0$

Also density of rock as mentioned below have been found

Dry density = 2.713 gm/cm^3

Wet density = 2.737 gm/cm^3

Based on anchor pull out test conducted on erection bay site by 32 ϕ tor anchor bar by applying a load of 25 tonnes, bond strength of 1.98 kg/cm^2 has been computed. In this case 12.60 m depth of anchor was grouted.

Also, pull out tests have been conducted in Greywacke rock in horizontal position, which gave bond strength of 2.41 kg/cm^2 for 25 ϕ anchor bar and having length of 4.6m.

Factor of safety:

The factor of safety of wedge is defined assuming that sliding is resisted by friction only and ϕ is the friction angle for both the phases is given by

$$FOS = \frac{(R_A + R_B)\tan\varphi}{W\sin\psi_i}$$

Where R_A & R_B are the normal reaction provided by the planes A & B, W is the weight of the wedge. A factor of safety of 1.5 has been adopted.

Establishment of Wedge Failure Based on Graphical Plot

Based on rock slope characteristics and orientation of different joint sets, the steroplot is plotted which gave different combination of joint sets, responsible for wedge failure. Then from plot it is established how many combinations of joint sets out of total types of joint sets can cause wedge failure. Further, separate stereo plot is plotted with the critical joints established (seven nos. in this case), rock slope, strike and angle of friction to have clear picture for the formation wedge failure and finally factor of safety is established.

Specification and No. of wires for 100 tonnes capacity anchor:

The high tensile wires each of 7mm dia, 30 nos. in each cable confirming to BS: 2691-1968 (Great Britain) with proper ties have been used.

Diameter of wire = 7mm

Normal relaxation has been considered.

Minimum tensile strength = 160 kg /mm^2

Stress load taken = 55% of min. tensile stress

Fig. 4.26 shows typical details of Prestressed Anchor comprising of H.T wires.

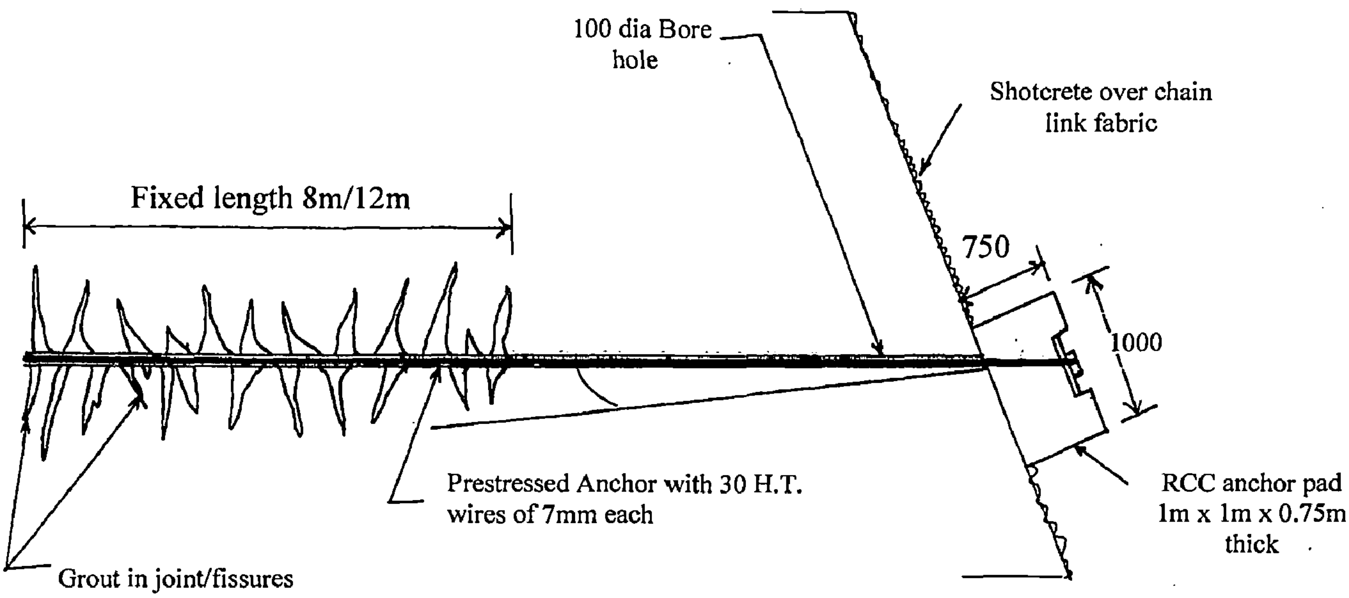


Fig. 4.26: Typical details of prestressed anchor

Computation for No. of wires:

Min. tensile strength = 160 kg/mm^2

Stress load = 160 x 0.55 = 88 kg/mm^2

Working load carrying capacity of each wire = 38.48 x 88 = 3,387 Kg

= 3.387 tonnes

No of 7mm dia H. T wires = 100/3.387

= 29.52

= 30 (say)

Fixed length:

As per Hobert & Zajic, the fixed length is calculated as:

$$l_1 = \frac{P}{\pi \times D \times Pb_1}$$

Where

D	=	Dia of ground hole
	=	100 mm (10 cm)
Pb_1	=	Bond stress between grout & rock
	=	4 kg/cm^2
P	=	Force on each anchor

$$l_1 = \frac{100 \times 10^3}{\pi \times 10 \times 4} = 796cm$$

= 8m (say)

Length required for bond between steel and grout:

$$l_2 = \frac{P}{n \times \pi \times d \times Pb_2}$$

Where

n	=	no. of wires in each cable
d	=	dia of wire
Pb_2	=	Bond stress between steel & cement
	=	6 kg/cm^2
	=	$\frac{100 \times 10^3}{30 \times \pi \times 0.7 \times 6}$
	=	2.53 m (say 3.0m)

Since $l_1 > l_2$, therefore fixed length of 8m shall be provided.

Selection of Bond strength:

The abstract of pull out test results are produced below for Greywacke rock.

Test No.	Bond strength (kg/cm^2)	Remarks
1.	4.89	Bond failure
2.	5.14	Bar failed
3.	7.88	Bar failed
4.	8.37	Bond failure
5.	19.46	Bar failure
6.	8.79	Bond failure

Test results No. 4 and 6 seems to be similar and taken for calculating the anchor length. A safety factor of 2 has been taken which gives average bond strength between rock and grout as 4.29 kg/cm^2. A value of 4 kg/cm^2 has been taken.

4.3.3 Slope stabilization measures

In case of Dharasu Power House, slope protection measures have been taken towards the hill facing upstream longitudinal wall of Power House. The prestressed anchors have been provided on the hill slopes facing upstream wall of unit nos. 1 to 4 in the area where penstock are approaching to machine hall (Fig.4.24). Mainly two types of rock exist i.e. Greywacke and Phyllite in the area under study. On the basis of failure mechanism the slope stabilization measures have been divided in two broad categories i.e. based on slip circle failure and on the basis of wedge failure.

4.3.3.1 Measures based on slip circle failure

Mainly in the area upstream of unit 4, there is predominance of Phyllite rock. Considering the nature of slope material viz. thinly foliated with puckered, sheared and brecciated and poorer grade Phyllite with R.B.M. over it and also the slopes are not stable, it is considered that a stable slope be provided after checking the stability of slope by slip circle method since wedge failure is not possible in this type of strata. The slopes in this portion have been stabilized by providing prestressed anchors of different capacities varying from 70 to 100 tonnes. The total depth of anchors having bore dia as 100mm varies from 39 m to 56 m at different locations at different inclinations (Fig.4.27). The anchors have been provided at spacing 2.4 m c/c along the profile in inclined plane and at spacing 2.8 m c/c in horizontal direction. The fixed length of anchors varies from 8.75 m to 16 m and measured after the assumed slip circle as shown in Fig. 4.27. At certain locations, where openings in the rock portion (e.g. Penstock adit etc.) have also been encountered during the installation works, the provision has also been made for 40mm dia H.T. steel tor bars. These have been provided in the 75mm dia bore holes and both ends have been kept threaded, so that after grout is set, steel bars can be tightened in position with nut over steel plates.

4.3.3.2 Measures based on wedge failure

There is predominance of Greywacke type of rock on the upstream side slopes of Power House facing unit nos. 1 to 3. Fresh Greywacke rock is hard, moderately to highly jointed. These joints are spaced from few mm to 30 cm apart and are almost filled with clay gauge material of thickness upto 1 cm and joints are smooth. Nine major types of joints are present established from the geological details. The various joints lead to wedge failure in the slopes for unit 1, 2 and 3 having Greywacke rock

Brief about wedge failure:

In case two discontinuities strike obliquely across the slope face and their line of inter section day light in the slope face, the wedge of rock resting on these discontinuities will slide down the line of intersection, provided that the inclination of this line is significantly greater that the angle of friction. Wedge failure means failure on two intersecting discontinuities. A condition of failure of wedge sliding is defined by $\Psi f_i > \Psi_i > \phi$ where Ψ_{fi} is the inclination of the slope face, Ψ_i is the dip of the line of Intersection (Fig. 4.28).

In this case, slopes for unit 1, 2, & 3 having Greywacke rock with nine prominent joints lead to wedge failure.

In this reach the steep rock portion has been stabilized by providing 100 tonnes capacity anchors, varying in length from 14 to 19m including the fixed length of 8m. The length of anchors is based on anticipated failure wedge as shown in Fig. 4.29. The anchors shall be provided at spacing of 2.6m c/c both ways in upward slope of 5° w.r.t. horizontal covering El. 811.0 m to El. 855.0m.

4.3.3.3 Measures adopted in both cases

On slopes adjoining to unit nos. 1 to 4, prestressed anchors ranging from 70 to 100 tonnes capacities totalling about 472 nos. have been installed. The total depth involved in the drilling of holes involved works out to be about 14000m, redrilled depth involved about 12000m and consumption of cement bags for grouting purpose comes to about 14700 nos.

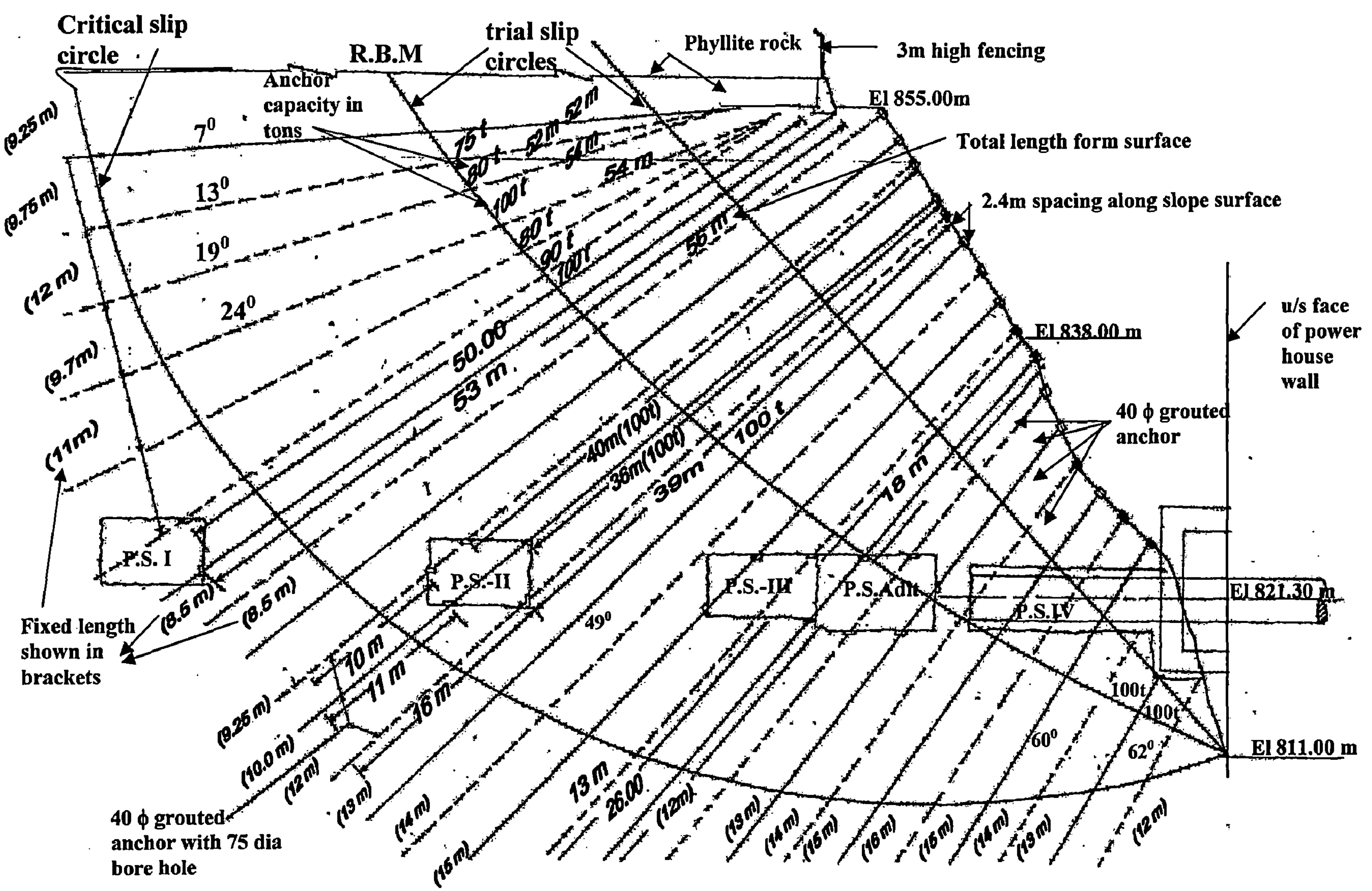


Fig. 4.27: Section showing prestressed anchors (slip circle failure case)

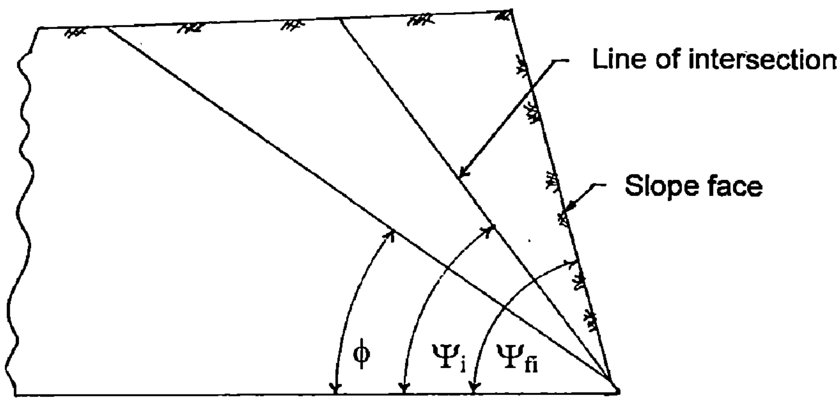


Fig. 4.28: Typical detail showing condition of wedge failure

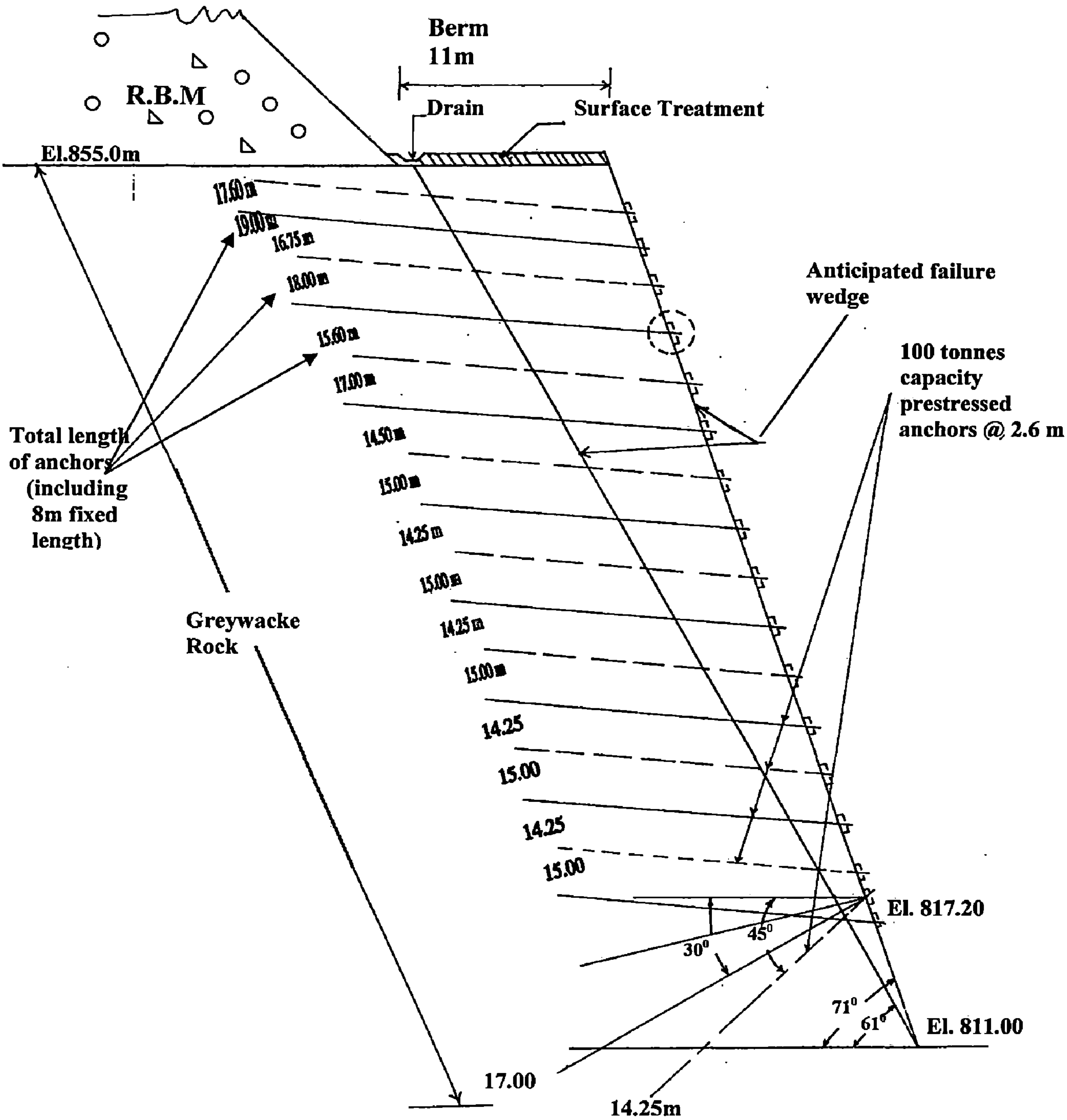


Fig. 4.29: Section showing prestressed anchors (wedge failure case)

4.3.4 Various aspects of Prestressed Anchors

(i) For achieving working load of 100 tonnes, 30 numbers of 7 mm H.T. wires have been used. Anchors of lower capacity up to 70 tonnes have also been used and number of H.T. wires chosen proportionally i.e. 21 H.T. wires used for working load of 70 tonnes.

(ii) The wires shall be kept parallel throughout their length to utilize full strength. The wires shall have minimum tensile strength equal to 160 kg/mm^2. To ensure good bond between wires and grout, wires shall be cleaned and degreased thoroughly.

(iii) The wires shall be wound with respect to each other with 2.2 mm dia winding wire @ 75 – 100 cm distance apart to keep them in position.

(iv) Grouting of hole shall be done from bottom upwards and grout shall be prepared from fresh good quality cement and using small size clean sand grains up to 2 mm. The applied grout pressure shall range from 5 to 10 atmospheres.

(v) The space left between concrete anchor pads provided for anchors shall be shotcreted with chain link fabrics.

(vi) The loading on anchors shall be increased / decreased by degrees of 0.40 – 0.80 – 1.00 – 1.20 and 1.40 of the permissible anchor load.

(vii) The bore hole of the free length of anchor shall be filled by pressure grouting with cement mortar or cement slurry to avoid corrosion of wires and to increase strength of rock.

(viii) The excavation work of penstock tunnel towards the power house side shall be completed prior to start of prestressed anchor work.

The following parameters have been adopted for rock/ steel and concrete as mentioned below:-

The bond strength between steel and concrete has been assumed as 6 kg/ cm^2 and bond strength between Phyllite rock and concrete has been taken as 2.75 kg/cm^2.

(a) Uniaxial compressive strengths of Greywacke and Phyllite rock have been assumed as 975 kg/cm^2 and 500 kg/cm^2 respectively.

(b) Angle of friction between joint planes with clay gauge has been assumed as 12^0.

4.3.5 Behaviour of Slopes

In case of Dharasu Power House, slope protection measures have been taken towards the hill facing upstream of longitudinal wall of Power House. The prestressed anchors have been provided (during 1987-91) on the hill slopes facing upstream wall of unit nos. 1 to 4 in the area where penstock are approaching to machine hall. After taking the adequate measures for slope stabilization, the area between hill face and upstream wall of Power House has been utilized for keeping the transformers after filling the gap with lean concrete. Till now, the slopes are functioning well and structures adjoining to slopes are safe.

CHAPTER-5

CONCLUSIONS AND SUGGESTIONS FOR FURTHER STUDY

5.1 CONCLUSIONS

Stabilization of geologically poor slopes associated with different components of Hydro-Electric and other projects in Himalayas is very vital. In most of the Hydro-Electric Projects, various kinds of stabilization measures have been attempted. These are discussed in preceding chapters of this report with the help of three case studies.

The detailed design principles, construction methodology, difficulties faced and further improvements are described in this study in general and particularly for case studies for stabilization of slopes. The following are the findings of the study.

a) The general slope protection measures such as placing wire crates or constructing retaining walls/ breast walls including drainage arrangement, rock bolts etc. are attempted but these are generally found inadequate to counteract the major de-stabilization forces. To safeguard against the forces of large magnitude, Prestressed Anchors are used in all the three cases described in this report. In Yamuna stage II project (commissioned in 1975) the prestressing was done from both ends because of presence of a gallery constructed for other purpose. In other cases prestressing is done from outside end and other end was grouted in the bore hole. In these cases grouting and corrosion protection are important for the success of these anchors. Therefore prestressed anchors should be used to stabilize big slopes at important structures. Other measures should be adopted depending on site conditions as extra safety.

b) The important specifications for various parameters of prestressed anchoring are as below.

   i) The tendons (used in anchors as prestressing steel) comprising of strands is better option as compared with solid bar or wire type tendons, particularly for high capacity anchors. The selection of strands provide better bond due to presence of V grooves between the helically wound peripheral wires and difficulty faced by long bar type tendons during handling and coupling is avoided.

ii) The selection of size of borehole is very important as it is directly related with bond efficiency. It is established that efficiency of tendon bond may fall for particular size of hole by increasing in number of strands and increasing the steel density. Hence the nos. and spacing among strands should be simulated with suitability tests prior to installation of anchors.

iii) For effective transference of load in the fixed length portion, minimum grout (usually>10mm) cover should be maintained among strands as well as between outer periphery of strands and borehole wall. In case of multi-strand tendon having longer lengths, the use of centralizer and spacer system is necessary to maintain desired grout cover.

iv) In the anchor design, the role of corrosion is very important as serious corrosion vanishes the purpose of whole anchoring system. For permanent anchors particularly of high capacity, the double protection system should be used i.e. comprising of at least two physical barriers against corrosion. Normally 1st barrier consists of sheaths made of poly propylene provided over strands after filling the gap between strand and sheath with grease to exclude the entrapped air pockets. The 2nd barrier may be provided as cement grout or epoxy resin grout.

v) The corrosion attack for anchor is severe particularly near surface because of easy intrusion of water. The anchor head protection system should be very sound. This comprising of protection to bearing plate, the stressing plate and the protruded ends of the strands with acid alkali, resistant black bitumastic paint including end cap grouting. The void, if any, left behind the anchor plate in the initial reach after the completion of secondary grouting should be monitored and if any gap is found it should be again grouted under pressure with suitable arrangements.

5.2 SUGGESTIONS FOR FURTHER STUDY

i) The adoption of safety factor to establish the working load with respect to GUTS (Guaranteed Ultimate Tensile Strength) needs proper study and attention, as it is directly related with the life of anchor. Normally working load varies from 50% to 70% of GUTS.

ii) In case epoxy coating is being used for corrosion protection system, its suitability needs to be properly established particularly with respect to stressing effects and effectiveness on the strands.

iii) In case of anchors installed in Nathpa Jhakri Hydro-Electric Project, the free length was secondary grouted i.e. anchors can neither be restressed nor further monitored. It is therefore suggested that feasibility of restressable and monitorable anchors be examined for high capacity anchors.

iv) The feasibility of using Fiber Reinforced Polymer (FRP) anchors should be examined for slope stabilization purpose, as these types of anchors are best suited where corrosion attack is high and no corrosion protection is required.

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APPENDIX A

DETAILS COVERING STRESSING, SECONDARY GROUTING AND CAPPING OF ANCHORS

I) Stressing of anchors

(i) Stressing is carried out with the help of multiple jacks through which all 12 strands of anchors can be pulled simultaneously.

(ii) The stressing is done when either the grout had achieved minimum cylinder strength of 25 MPa or 21 days had elapsed, whichever is earlier.

(iii) Stressing shall be carried out in steps and the Table showing load chart for stressing is given below for reference (Table A-1).

(iv) All anchors shall be checked by simple stressing test and at least 5% of the total number of anchors installed shall undergo the comprehensive stressing test. Stressing will be carried out as per details given in the succeeding paras.

(v) Clean the protruding anchor strands with wire brush and remove all traces of rust, dust and any other foreign material.

(vi) Mount the stressing plate over the embedded bearing plate through the 12 protruding strands of the anchor.

(vii) Insert 12 nos. wedges (consisting of 2 parts or 3 parts) on all the strands pushing them into the stressing plate with a pipe driven by hand. In one and the same anchor only one type of wedges i.e. either 2 parts or 3 parts shall be used.

(viii) Fix the stressing chair over the stressing plate, insert the hydraulic locking device inside the chair and mount the multiple strand jack on the chair. The anchor is ready for stressing.

(ix) After fixing the multiple strand jack apply pressure through hydraulic pump from zero and stress up to 10% of the load i.e. 20 tonnes. The corresponding pump pressure is 19.5 or say 20 kg/cm^2.

(x) Release the pressure to zero. This is to remove the slackness of the strands

(xi) Restart the pump pressure and stress to 10% of the working load i.e. 20 tonnes as above. Note the elongation of the jack in mm and enter the elongation in column 4 of the Anchor Stressing Record showing sample calculations (Table A-2).

(xii) Increase the pump pressure further to 40% of the working load i.e. 80 tonnes which corresponds to a pressure reading of 78 kg/cm^2. Note the elongation and enter in column no. 5.

(xiii) Stress further up to 100% of the working load i.e. 200 ton which corresponds to a pressure of 195 kg/cm^2. Note the elongation in column no. 6.

(xiv) Lock the wedges and release the pressure to zero. The maximum travel of the jack is 210 mm which is sufficient for an anchor of 35m length. Since the anchors being installed are longer than 35m, the stressing has to be carried out in two strokes. Therefore, do not stress beyond 190 mm in the first stroke.

(xv) Start increasing the pressure and unlock the wedges that were locked as mentioned in para (xiv). Stress up to 80% of the GUTS (Guaranteed Ultimate Tensile Strength). For 15.20 mm dia 7 ply strands of class II, the GUTS is 26.07 tonnes. Therefore, 80% GUTS works out to be 20.86 tonnes which corresponds to a pressure of 244 kg/cm^2. Therefore stress to 250 kg/cm^2 and note the elongation and enter in column 7.

(xvi) After stressing to the test load as per para (xv) wait for 5 minutes and then note the elongation in 2nd stroke row, column 8.

(xvii) After reaching the test load detension the anchor by reducing the pressure to 215 kg/cm^2 which corresponds to the proof load (which is design load + 10%) i.e. 220 tonnes. Note the elongation at this load and enter in 2nd stroke row, column 9.

(xviii) Thereafter stress the anchor by further 3-5mm to compensate wedge seating losses. Lock the wedges and note the elongation and enter it in 2nd stroke row, column 10.

(xix) Release the pressure to 50 kg/cm^2 and note the elongation in column 11 (Table A-2).

(xx) Release the pressure to zero and remove multiple strand jack, locking device and the chair.

(xxi) Calculate the elongation as per specimen at Table A-3.

(xxii) Check all the anchors that have been stressed according to the simple stressing method as above 3-5 days after stressing by lift off test. Enter the observation in specimen at Table A-4.

(xxiii) If the residual anchor load is more than 98% of the proof load the anchor is accepted. If the loss is more, stress the anchor to proof load and then further stress as per para (xv) to (xviii) and recheck after 3-5 days and enter the observations in Table A-4.

(xxiv) During the comprehensive stressing test, the anchor shall be tensioned in steps. The behaviour of the anchor shall be observed during the intermediate steps, the unloading periods and at the test load.

(xxv) Maintain the load at each step for at least 5 minutes and record the load deformation curve for loading and unloading.

(xxvi) After reaching the test load, the anchor shall be fully detensioned. Thereafter the anchor shall be stressed to proof load and stressed further by 3-5mm to compensate for the wedge seating losses.

(xxvii) All anchors which shall be tested according to the compressive stressing test shall be short term monitored during the time of installation of the working anchors by means of measuring the residual load with a hydraulic load cell. If the loss is more than 2% of the proof load, the anchor shall be restressed as per paras (xv) to (xviii).

(xxviii)The measuring interval for the residual load with a hydraulic load cell shall be as under:

First week after stressing	Daily
Upto 3 weeks after stressing	Each 3rd day
Up to 6 weeks after stressing	Weekly
After 6 weeks to six months or till the anchor is accepted.	Each 2nd week

(xxix) After the anchor has been accepted, cut the ends of the strands leaving 15mm length of the strand protruding beyond the anchor head.

II) Secondary grouting of anchors and capping of anchor heads

(i) After stressing or restressing has been completed and the anchor has been accepted, the annular space between the hole and the strands shall be filled with neat cement grout.

(ii) Before carrying out the secondary grouting of the free length of the anchor, air at high pressure shall be blown into the hole to remove any water accumulated in the free length.

(iii) Neat cement grout of w/c ratio 0.40 and with non-shrink grout compound @ 250 gm per 50 kg of cement will be injected at a high pressure say 25 psi through the grout hole of the stressing plate till the grout starts flowing through the air vent pipe fixed between the trumpet pipe and the bearing plate.

(iv) As the water testing of only of lower portion of the hole is carried out, some grout is likely to escape from the hole. Therefore, regrout the anchor next day till the grout starts escaping through the vent pipe.

(v) Thereafter tightly plug the grout hole of the stressing plate.

(vi) To ensure that the secondary grout covers the complete anchor, inject grout through one of the vent holes and continue grouting till the grout starts escaping through the second vent hole.

(vii) Plug the second vent hole first and then remove the grout injection pipe from the first vent hole and plug this hole also.

(viii) After secondary grouting has been completed, the anchor head is to be protected against corrosion and possible damage due to the falling debris from the slopes above in the following manner so that the anchor can carryout its function during its planned service life.

(ix) The bearing plate, the stressing plate and the protruded ends of the strands of all the anchor shall be painted with Acid alkali, resistant black bitumastic paint conforming to IS: 158.

(x) The anchor head will be covered with a steel cap 3mm thick having two vents, on top, one for injecting the grout and the other for escape of the entrapped air.

(xi) Size of the steel cap shall be 310 mm dia and 110mm deep having a minimum clearance of 50mm from the anchor head mechanism.

(xii) The steel cap shall be centered over the stressing plate and then spot welded to the bearing plate. Steel/wood putty shall be applied all around the steel cap at its junction with the bearing plate to make it air/water tight.

(xiii) Cement grout with a w/c ratio of 0.40 and non shrink compound as used for primary/secondary grouting shall be filled in through one of the vents permitting escape of the air through the other vent.

(xiv) After completion of the grouting of the anchor head, the cap will be painted with a coat of paint as mentioned in para (ix).

Table A-1: Load chart for 200 tonnes capacity cable anchors

Type of load	% of Anchor Load	Load (tonnes) on Anchor	Load (tonnes) on each Strand	Corresponding Jack Pressure (Kg/cm^2)
% of working load	10%	20	1.67	19.50 or say 20
% of working load	40%	80	6.67	78 or say 80
Design or Working load	100%	200	16.67	195 or say 195
Proof load	110%	220	18.33	214.50 or say 215
Test load	80% of GUTS	250	20.8	244 or say 250

1. GUTS = Guaranteed Ultimate Tensile Strength of the Strand
2. GUTS of 15.2 mm Class II Str = 26.07 tonnes
3. Ram Area of ISMAL 4000 MG = 1025.70 cm^2
4. Max. Jack Pressure = 455 kg/cm^2

Table A-2: Anchor stressing record (sample calculations) (200 ton capacity anchors)

ANCHOR DATA

Anchor Location	: DAM SITE
Anchor No	: D-3
Type of Test	: Suitability – Acceptance
Total length of Anchor	: 41.51 Meters
Free Stressing Length	: 31.51 Meters
No. of Strands	: 12

Dated of Primary Grouting: 10 Dec, 1996

STRAND DATA

Nominal/Actual Dia & Class	: 15.20/15.54 mm CL2
Actual E-Value	: 19994 Kg/mm^2
Nominal/Actual area of C/S	: 140/146.38 mm^2
Coil No.	: 216A
GUTS of Steel min/actual	: 26.07/28.40.T
80% GUTS of steel min/actual	: 20.80/22.72 T

Note: Maximum Elongation not to exceed 190mm

GUTS- Guaranteed Ultimate Tensile Strength

EQUIPMENT DATA:

Jack Make/capacity	: ISMAL 4000 MG
Jack Sl.No.	: 96002
Pump Sl.No.	: 96001
Pressure Gauge	: 512046, 512047
Jack Ram Area	: 1025.70 cm^2
Stroke length/max. elongation	: 210/190 mm
Date of stressing	: 18 Jan 1997

Maximum Jack Pressure Not to Exceed Under any Circumstance : 455 kg/cm^2														
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Anchor Stressing Sequence	Calculated elongation: at Proof Load	Calculated elongation: at Proof Load									Total elongation	Slip & draw in immediately after locking & release :	Net elongation	Remarks:
All strands stressed simultaneously	Before slip & drawn-in:	After slip & drawn-in:	At 10% of W.L. (20 kg/cm^2)	At 40% of W.L. (80 kg/cm^2)	At 100% of W.L. (195 kg/cm^2)	At Test Load 80% of GUTS (250 kg/cm^2)	After 5 Min. (250 kg/cm^2)	At Proof Load 110% of W.L. (215 kg/cm^2)	At instant of locking:	After release (50 kg/cm^2)	Ratio 11/7 (Col 9-Col 5)	(Col. 10-11)	(Col. 12-13)	
1st stroke			17	75	185									
2nd stroke					25	85	85	73	81	75				
Total			17	75	185	245	245	233	241	235	248.28	6	242.28	

Table A-3: Specimen for calculations of elongation

200 tonnes capacity anchor no. D-

Length of Anchor	L	=	-----------m
Free Stressing length of Anchor	Ls	=	------------m
Design Working Load	P_0	=	200T
Proof Load	PP	=	220 T
No. of Strands/Anchor	N	=	12 strands 15.2 mm
Ram Area of Jack Used (ISMAL 4000 M Jack)	A	=	1025.70 cm^2
Wedge set (Slip) at Stressing Anchorage Jacks	(ΔLw1)	=	5mm for MG
Jack Wedge Set	(ΔLw2)	=	5mm
Length of Strand within the Jack	L_j	=	750 mm
Actual E-value of strand supplied	Es	=	-----------kg/mm^2
Actual cross sectional area of strand supplied	Ac	=	-----------mm^2
Prestressing force required on each stand	Ps	=	220/12 = 18.33 T
		=	18333.33 kg
Elongation required in Free length	ΔLs	=	(PP / Es x Ac) Ls
		=	(220 x 1000)/ (------x 12x----)x Ls
		=	--------mm say------mm
Elongation of strand within jack	Δ Lj	=	(PP/Es X Ac) 750
		=	(---- x 12 x --------) x 750
		=	-------- say -----------mm
Actual Elongation to be noted on Jack Ram			ΔL = ΔLs + Δ Lj + Δ Lw1 + ΔLw2
		=	----- + ------+ 5+5
	ΔL	=	-------------mm
Net Elongation after locking & Release	Δ Ls	=	-------------mm
Corresponding Jack Pressure required for achieving elongation of ΔL MM on Jack Ram		=	PP/Ram Area of Jack
		=	220 x 1000/1025.70
		=	say 214.5 kg/cm^2

Table A-4: Specimen for Restressing & Monitoring Record (200 T capacity Anchors)

ANCHOR DATA
Anchor capacity (working load) : 200T
Anchor location : Dam Site/Power Intake
Ram Area of pressure gauge : 1025.70 cm^2
Stressing date : --------------- 1997

Anchor Data		Stressing gauge reading in kg/cm^2					
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Anchor N*	Date of stressing	At locking (110% W.L. = 220 T)	At reopening of wedges After 3-5 days	At relocking to Proof load (110% W.L = 220T)	At reopening of wedges for retest	Deviation % {(Coo. 3-Col. 4 OR 6)/Col.3} x100	Remarks
D-		215		215			
D-		215		215			
D-		215		215			
D-		215		215			
D-		215		215			
D-		215		215			
D-		215		215			
D-		215		215			
D-		215		215			

APPENDIX B

DETAILS COVERING MONITORING OF ANCHORS

I) Monitoring related with Corrosion Protection System

Epoxy Coating Application

Epoxy Composition: Type Araldite, 2 parts Resin Gy-257 and 1 part Hardener Hy-840

Earlier method adopted

Initial practice involved cutting of 48 strands of required length by laying parallel in one layer and touching each other. The strands are supported by steel channels spaced about 3 meters. In the steel factory the strands stands cleaned from water soluble oil by cloth and water manually. Epoxy components are mixed by hand and applied by paint brush manually to the top and to the bottom of the strand layer. After 24 hours drying period a second coat is applied in the same manner, the strands not having been moved, and quartz sand is sprinkled over and under.

Also it was found that epoxy coating stop short of the stressing anchorage as it prevents proper functioning of the anchoring mechanism.

Improvements

i) Later on it was decided that each strand shall be rotated by 90 degrees before the second coat is applied. The sides of the strands which may not have received a continuous coating in the first application will now be exposed to the brush.

ii) Initially anchor fabrication was done in open on road side but after some time it was shifted to a less dusty area covered with roof.

iii) Further it was decided to carry out the works in a more systematic manner e.g. by separating from each other the operations comprising of cleaning, epoxy coating and anchor assembly. These operations have been performed on separate benches. Stop ends have been provided for correct alignment of the strands, colour marks to indicate where strand must be cut and coating must end. For epoxy application now it has been decided to keep 12 strands to 1 worker instead of 48 to 2 workers done earlier.

iv) Now each anchor has been tagged for identification and traceability, which was not, practiced earlier (e.g. anchor number, top/bottom row, strand coil).

v) The epoxy coating is sensitive to abrasion. In some cases it was noticed that wires become blank when an anchor assembly is pulled across a steel edge.

vi) In order to protect the epoxy coating, avoid moving epoxy coated strand longitudinally across the supporting steel profile. Clad with wood the support channels of the assembly bench.

vii) It has been further suggested that abrasion is tested between an anchor assembly and the rock, e.g. by homing an anchor and pulling it back out. If abrasion is noticed, then a plastic pipe has been placed over the free length and pull the pipe back out after homing. By this method at least the free length is protected against abrasion.

viii) Suitability Test: The three suitability tests conducted earlier did not serve the purpose. Then attempt was made to modify the method to confirm the bond length and corrosion protection. In the test anchors a bigger number of strands have been chosen, e.g. 16, so that ultimate load of a 12-strand anchor can be developed without overstressing strands and equipment. Borehole diameter adopted was same, i.e. 150 mm diameter. The bond length of 10m for 200 tonnes capacity anchors has been confirmed. Also pulled out strands were checked for corrosion protection and have satisfactory results.

II) Monitoring related with stressing work

Earlier method adopted

At the initial stage of work, in few anchors 2 types of wedges i.e. consisting of 3-part and 2-part wedges have been used and due to this uneven wedge seating (up to 10 mm difference) has been observed, which is not desired.

In the beginning, it has been observed that the stressing equipment in use is unable to release the cable from test load to proof load. When the piston is returned the wedges in the stressing block engage and lock the cable. Locking takes place after the cable has returned 5-10mm (say 8mm). Subsequent overstressing by 8 mm resulted in a stressing force near of equal to test load, immediately before lock-off.

Improvements

i) Now one type of wedges i.e. either consisting of 3–part or 2-part wedges have been used in single anchor.

ii) The following options have been explored for simple stressing tests.

Option 1: Using a jack able to reach test load in one stroke, removing wedges in the stressing block, then releasing pressure to zero, installing the wedges and stressing further to proof load as usual.

Option 2: Using 2 jacks on top of each other providing combined enough stroke to reach test load directly, removing wedges in the stressing block, then releasing pressure to zero, installing the wedges and stressing further to proof load as usual.

Option 3: Placing the jack on a 'chair', stressing up to test load, removing the wedges (accessibility provided by chair), releasing to a load below proof load, then installing the wedges and stressing up to proof load as usual.

Finally, option 3 has been considered by designing special 'chair' to fulfil the requirements. But, till this method has been adopted, the interim method has been used for few anchors to cope up with the time schedule as mentioned below.

Interim method: Stress to proof load plus 8 mm, hold for 5 minutes and lock-off and cancel the test load. In this case 5 measurements shall be taken and recorded between zero and proof load.

iii) In the case of 200 tonnes capacity anchors, paint marks shall be applied to all 12 strands at an equal distance from the jack for control of eventual elongation differences during stressing. The paint marks shall be applied when 10% of working load has reached. The cables shall be stressed by multi-strand jack. The single strand jack shall be used only when a problem arises with one or several strands in a cable if required.

iv) Re-stressing: The restressing of anchors has been done to check the load and generally performed within 3-5 days after initial stressing. If the load is found to be 98% or more than proof load, the anchor is accepted and secondary grouting can be done. For this purpose the jack is applied and stresses up to proof load. If the wedges don't open, it is assumed that the load in the cable is at least 98% of proof load.

v) The load checking has been performed by the lift-off test. In this method jack is positioned on a 'chair', as mentioned under *'Simple Stressing Test, Option 3'*, keeping the stressing block clear. The load applied to the strands which is required to lift off the stressing block from the bearing plate, is equal to the load in the cable. Lift –off shall be minimal. It can be checked by a feeler gauge.

III) Monitoring related with grouting work

Earlier method adopted

i) Apart from assuring load transference in the fixed length from strands to rock, grout is also vital for corrosion protection of strands and other anchor steel. The quantity of primary grout defines the extent of the fixed length. The specified fixed length is 10m throughout for 200 tonnes anchors. On the basis of the elongation values recorded in the initial stage of work the actual fixed lengths of anchors stressed so far have been determined. These vary between 7.3 m and 20.5m, in average 13.9m.To assure the anchor force is transferred to the rock behind the second layer of biotite, the value of 10m as computed from a design point of view and further confirmed from suitability test shall not be exceeded.

ii) Strands shall have a grout cover of at least 10mm. At initial monitoring of works after installation of few anchors, some strands in the fixed length portion have been found lying directly on the rock, which is not desirable. Also in few cases the strands in free length touch the bore hole wall.

iii) Being anchoring activity at critical path, earlier it was practiced to use various grout mixes in an effort to cut down waiting time between primary grouting and stressing from 21 days to 10 days. By use of super plasticizer cylinder strength of 250 kg/cm^2 was attained in less than 10 days.

iv) At initial stage of work secondary grouting has been performed by gravity and not under pressure. This has created a void of considerable size behind the anchor plate and consequently the strands did not receive grout cover in the upper end near the anchorage. Also secondary grouting has been performed from top and not from bottom.

Improvements

i) The quantity of grout has been reduced i.e. use of cement bags have been restricted to 5 bags from 6.5 bags as practiced earlier. A trial to this aspect has been made outside the rock by means of a pipe simulating the last part of the borehole confirming the requirement of cement bags.

ii) The spacers have been modified to assure adequate clearance between strands and borehole in the fixed length portion. In free length a clamping ring has been used within trumpet to reduce the strand bundle to a diameter of 115mm.

iii) The use of super plasticizer to attain the cylinder strength of 250 kg/cm^2 in less than 10 days has been reviewed. This is because apart from satisfying strength part, other properties like shrinkage or expansion, water absorption and flow are also important and shall be satisfied. Accordingly to achieve required strength in 10 days with ordinary Portland cement, an appropriate admixture and potable water of ambient temperature have been used. Suitability tests have also been conducted with the selected grout and sample tested right after the curing time, which is intended for the production of anchors. In the majority of cases stressing has been done after 21 days, hence no admixture added accordingly.

iv) Now the secondary grouting has been performed in 2 stages from bottom up. In the 1st stage grouting was performed by gravity and using the center hole of stressing block as vent. In the 2nd stage grouting was performed in the void area behind the anchor plate and using center hole as inlet, and providing vent at top of trumpet. Apply the pressure until grout penetrates the wedges and seal off.

v) Shortly after secondary grouting 2nd stage the end cap shall be installed. Borehole, trumpet and end cap shall be filled in such a way that no void is left and no air is trapped.