MANUAL ON THE PLANNING AND DESIGN OF HYDRAULIC TUNNELS

CENTRAL BOARD OF IRRIGATION AND POWER

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FOREWORD

Electrical energy is of prime importance for the socio-economic development of our country. The present emphasis of our planners is to develop and expand electrical energy through pollution-free and renewable resources. The holy waters flowing down our mountain streams and rivers can contribute to a solution of our energy problems, if properly harnessed. Only a part of the hydro-electric potential of India has been exploited so far. Some of the easiest projects have already been exploited. Those under planning now, include some of the most difficult and complex sites in unpredictable Himalayan rocks. Many of these schemes envisage the construction of a large number of tunnels of various shapes, sizes and lengths passing through varying geological formations. The art of tunnel engineering has undergone many changes during the past half a century. With the present state of the art of rock mechanics, it has now become possible to design safer and more economical tunnel sections. The information on this subject is scattered in various articles on the subject. In this manual, a lot of such information has been compiled and arranged chapter-wise so as to enable the readers to understand the subject in a comprehensive manner. Wherever necessary, worked-out examples have been given. The matter presented is based on design practice followed in the Central Water Commission supplemented by reference to relevant literature on recent trends followed in other countries.

With the rapid strides being made in the field of rock engineering, this manual would require to be updated in the next few years.

However, I am sure that the Manual in its present form will meet the immediate long-felt need of engineers concerned with the planning, design and construction of tunnels and underground openings.

I would like to place on record the tremendous effort put in by Shri P.K. Sood, Deputy Director (HCD-I) in the preparation of this manual. The contributions of Shri K. Madhavan, Member (D & R), Shri M.P. Parasuraman, Director (HCD-I) and Dr. H.R. Sharma, former Director (HCD-I) in the preparation of this manual are also worthy of acknowledgement.

Pritam Singh

(PRITAM SINGH)
Chairman
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CONTENTS

CHAPTER 1

INTRODUCTION, CLASSIFICATION AND SCOPE

1.1 Introduction

Tunnels can be defined as underground passages made without removing the overlying rock or soil.

Even in ancient times underground structures presented a challenge to man. Apart from natural caves, tunnels driven to undermine fortifications constituted an important and effective means in ancient warfare. Second to military purposes was the extraction of valuable mineral resources from the hidden depths. Besides military and mining purposes, the earliest uses of tunnels included store houses, tombs, temples etc. More recent applications are routes for roads, railways, canals, hydro-electric power and water supply schemes.

Through recent progress made in the field of mechanical engineering equipments like pneumatic drills, improved drill bits, improvement in the quality and strength of modern explosives, advent of sophisticated clearance machinery, highly successful application of electrical power for lighting and ventilation, introduction of new techniques for supporting the tunnel, like the use of steel ribs, precast and cast-in-situ concrete lining, cast iron lining in conjunction with pressure grouting, shotcreting and rock bolting, and the application of various forms of shield for driving tunnels through water bearing strata etc., speed of construction has been greatly increased and danger to workmen has been drastically reduced. Tunnelling has become much more simpler, safer and faster than what it was, say fifty years ago. Man's ever increasing need for energy has opened up new possibilities of developing power through run-of-the-river schemes in difficult and mountainous terrains where a major part of the water conductor system is in the tunnels. Most of the geologically competent sites have already been explored and presently man is facing the challenge of making tunnels in difficult and unpredictable mountains. As such, it becomes more important to learn about the latest techniques in planning, design and execution of tunnelling operations.

In this manual, the above three aspects in relation to hydraulic tunnels have been discussed in details.

1.2 Classification

Tunnels may be classified according to their purpose, shape and supporting arrangements.

1.2.1 Depending on their purpose the following two main groups of tunnels may be distinguished:

(A) Traffic Tunnels:

(1) Railway Tunnels
(2) Highway Tunnels
(3) Navigation Tunnels

(B) Conveyance Tunnels:

(1) Hydroelectric power station tunnels—these shall be referred to as "Hydraulic Tunnels" in all further discussions.
(2) Water Supply Tunnels
(3) Sewer Tunnels
(4) Transportation tunnels in industrial plants.

Hydraulic tunnels can be further sub-divided into the following categories :

(1) Pressure Tunnels
(2) Free flowing Tunnels
(3) Free flowing-cum-pressure tunnels

1.2.2 Depending on their shape, tunnels may be classified as :

(1) D-shaped
(2) Horse-shoe shaped
(3) Circular shaped
(4) Elliptical shaped
(5) Square or rectangular shaped

The selection of the tunnel cross-section is influenced by :

(1) The clearances specified in view of the vehicles and materials transported in the tunnel,
(2) Geological conditions,
(3) The method of driving the tunnel, and
(4) The material and strength of tunnel lining.

The first step in the design of a tunnel is the determination of the cross-section required for the trouble free operation. For the mucking and hauling equipment that will be used during construction, the clearance of tunnels in which rail tracks are to be provided for mucking should be so designed that it should be at least 30 to 40 cm larger than the clearance required for the open line. This is to provide additional safety against constructional in-accuracies and deformation of the section due to rock pressure and presence of water. While determining the size of the section, space required for ventilation ducts, compressed air pipes, supply cables for lighting power safety equipment etc. should be kept in mind.

The type of geological environment in which the tunnel is to be constructed has got a considerable influence on the shape of the cross-section. In hard and infact rock, tunnel sections excavated with an arched roof may serve. In loose and weak rocks, a relatively considerable lateral thrust may be expected. The greater the magnitude of the lateral pressure, the more advantageous a circular cross-section would be. In such cases, selection has to be limited either to a circular section or a horse-shoe shaped section. Economy of both the sections should be carefully studied before recommending either of it.

The method of construction must be chosen in accordance with the prevailing site conditions, but may be influenced by the availability of equipment, machinery and materials.

Conventional tunnelling methods are suitable for driving horse-shoe and flat arched sections, and are less economical for circular sections.

The shield method is restricted to circular sections only.

The free face method can be used for cross-sections of any desired shape.

The material used for tunnel lining also influences the shape of the cross-sections, since materials capable of resisting compressive stresses only are limited to structures composed of arches such as horse-shoe, circular and elliptical sections. Materials capable of resisting tensile and bending stresses alike (RCC, Steel etc.) can be used for lining sections of any desired shape.

1.2.3 Tunnels may also be classified as "lined" or "unlined" tunnels. If the rock conditions are favourable and the tunnel is required to be used for a short period of time, e.g., a diversion tunnel constructed for the construction of a dam, the tunnel may be left unlined. However, in most cases, hydraulic tunnels are invariably lined with cement concrete (Plain or reinforced) or shotcrete. Hydraulic tunnels discharging silt-ladden water under high velocities. (e.g., silt flushing tunnels) are required to be steel-lined.

1.2.4 Lastly, tunnels may also be classified based on the supporting arrangements. Under excellent rock conditions, the tunnels may be left unsupported. Depending upon the type of supports, tunnels may be classified as:

(i) Tunnels supported by R.S.J. sections.
(ii) Tunnels supported by Rock bolts.
(iii) Tunnels supported by Shotcrete.
(iv) Tunnels supported by a combination of (i), (ii) and (iii).

1.3 Scope

This manual deals with the planning, design and execution of Hydraulic tunnels used for Power Houses and Diversion works only.

CHAPTER 2

PLANNING AND INVESTIGATION

2.1 General

The planning and investigation of Hydraulic Tunnels shall include the selection of its alignment and the geological competence along the alignment selected.

2.2 Selection of the Alignment

While selecting the alignment the following points should be considered :

(i) *It should be the Shortest Possible* : This would ensure minimum losses and shall be economically cheapest.
(ii) *It should be Straight as far as Possible* : Introduction of bends in the alignment shall involve losses at all such bends and the cost of tunnelling would also increase.
(iii) *It should be Easily Accessible* : An easy access near the entrance and exit to the tunnel becomes essential for the construction facility.
(iv) Careful selection of entry and exit locations with minimum length and depth of approach cuttings and no weathered, loose fractured layers slope towards portals.

However, it is not always possible to follow a straight alignment because of the following parameters affecting the design of hydraulic tunnels ·

(i) *Topography* : There may not be sufficient vertical and/or lateral cover.
(ii) *Geological Section along the Alignment* : It may show certain difficult strata through which the process of tunnelling may be cumbersome and uneconomical.
(iii) Ground and/or Rock Water Loads along the tunnel alignment may be excessive thus increasing the overall cost of the tunnel.
(iv) *Rock Mechanics Properties* : The in-situ stresses, joint pattern, shear strength, unconfirmed compressive strength, shear modulus of deformation etc. may not be favourable along a particular alignment.
(v) *Creep or Tectonic Movement along the Tunnel* : The tunnel may be passing through active faults or lineaments which could cause collapse of the tunnel.

In addition to the above, certain other parameters may also affect the tunnel alignment because of the construction difficulties. These are :

(i) Rock temperatures,
(ii) Presence of methane gas,
(iii) Presence of jointed rock acquifers confined by impervious strata which might lead to heavy inflow of water.
(iv) Squeezing rock conditions.

2.3 Investigations

The investigations for selecting the route or alignment of tunnels are done by means of selective drilling and by making drifts and test tunnels to obtain as much information as possible of the geological conditions prevailing at the site.

The most important phase of preliminary work in tunnelling is the careful exploration of geological conditions.

The purpose of geological explorations are as below :

(i) The determination of the origin and actual condition of rock,
(ii) The collection of hydrological data and information on underground gases and soil temperatures,
(iii) The determination of physical, mechanical and strength properties of rock along the proposed line of the tunnel, and
(iv) Determination of geological features which may affect the magnitude of rock pressures to be anticipated along the proposed locations.

Explorations should be extended to determine :

(i) Top cover,
(ii) Quality of subsurface rock,
(iii) Surface drainage conditions,
(iv) Position, type and volume of water and gases contained in subsurface rock,
(v) The physical properties and resistance to tunnelling offered by the rock encountered.

Drilling provides us with valuable information for delineating geological profile all along the proposed alignment and also gives information about rock conditions, pattern of joints, presence of weak zones, extent of rock cover etc. which are very important parameters for ascertaining the stability of the tunnels and the types of construction problems likely to be met with during tunnelling operations.

From the drilling data, the ground water levels and Rock Quality Designation (RQD) can be obtained. RQD is defined as the percentage of core samples longer than 10 cm recovered during drilling and is indicative of the rock characteristics and tunnelling conditions.

The investigations should also be carried out to gether additional information for determining the support conditions, stand up time etc. These are : Uniaxial and Point load compressive strength of rocks; spacing of joints; condition of joints; ground water conditions and orientation of joints. Beiniawskie defined a classification system wherein different ratings are given for the above factors and the final rating for the particular rock mass could be arrived at. This classification system has been discussed in details in Chapter 10.

The sequence of geological explorations referring to tunnel constructions may be carried out in the following order :

(i) Investigation of general character prior to planning. This should be done by thorough field reconnaissance on foot and shall include the bibliographical and statistical survey of morphology, petrography, stratigraphy and hydrology of the environment.

(ii) Detailed geotechnical (Subsurface) investigation parallel to planning but prior to construction. This would furnish information regarding physical strength, chemical properties of rock to be penetrated as well as their condition.

(iii) Geological investigations should be continued during construction, not only in the interest of checking design data but also for ascertaining whether the drilling method adopted is correct or needs to be modified.

One of the most important items for investigations is the determination of in-situ stresses. The in-situ stresses have a marked effect on tunnelling conditions. The various methods presently available for determining the in-situ stresses can be obtained from any standard text book on Rock Mechanics.

Pre-construction stage test tunnels or drifts can assist designs and result in substantial economy. In India, at Khodri Project, a test drift was driven 400 metres long with cross drifts to intersect the intra-thrust zones to obtain advance information about tunnelling conditions. Studies were carried out to obtain rock properties like shear modulus, deformation modulus, behaviour of rock on formation of cavity, activity of thrust planes, measurement of creep etc. Tests were also carried out in experimental logging sections. The results obtained from these investigations proved to be of tremendous help in the proper planning and execution of the tunnelling operations.

However, test tunnel section or drift can be justified only in case it is supported by properly planned instrumentation programme. The minimum programme shall include :

(i) Closure measurements from boreholes,

(ii) Stress measurements—both in-situ stresses as well as stresses in lining,

(iii) Detailed geological logging,

(iv) Seismological observations from an observatory located at the site,

(v) Installation of deformation gauges, and

(vi) Rock pressure measurements from Pressure Cells.

The above listed measurements should also be supplemented by angular and levelling measurements of pillars constructed on the surface at typical location.

The site of the test drifts may also be selected with a view to use the same for future construction requirements of adits.

Recently the underground construction Research Council set up by the American Society of Civil Engineers and National Sciences Academy of U.S.A. has strongly recommended such investigation and instrumentation programmes wherever feasible particularly where new construction and supporting techniques are contemplated. It may be worthwhile to mention here that proper and systematic investigations and instrumentation could give lot of information before hand which could go in a long way to solve many of the varied tunnelling problems.

CHAPTER 3

GEOMETRIC AND HYDRAULIC DESIGN

3.0 Geometric Design

After the final alignment of the tunnel has been chosen by carrying out detailed planning and investigations, the next step for the designer is to choose an appropriate geometric section of the tunnel. For doing this the judgement of the designer is required for making a final choice of a section considering the prevailing site conditions. No general recommendations can be made to fit in each and every individual case but a few important and widely used geometrical sections for hydraulic tunnels are discussed below :

The following shapes are generally used for hydraulic tunnels :

(a) Circular section
(b) D-shaped section
(c) Horse-shoe section
(d) Modified horse-shoe section

Sometimes egg-shaped and eggilipse sections are also used.

3.1 Tunnel Cross-Section

Cross-section of a tunnel depends on the following factors :

(i) Geological conditions prevailing along the alignment,
(ii) Hydraulic requirements,
(iii) Structural considerations, and
(iv) Functional requirements.

A final choice of the section is made by carefully scrutinizing the above four factors. However, once a section has been adopted, it is not binding on the part of the designer to stick to the same section throughout the length of the tunnel. There are many instances where the sections have been modified during the course of construction of the tunnel. Such contingencies should, however, be avoided as far as possible so as to fulfil the contractual obligations.

The general requirements and design parameters for the few widely used shaped are discussed below:

3.1.1 *Circular Section*

The circular section is most suitable from structural considerations. However, it is difficult for excavation, particularly where the cross-sectional area is small. In a case where the tunnel is subjected to high internal pressure but does not have good quality of rock and/or adequate rock cover around it, circular section is considered to be most suitable.

3.1.2 *D-shaped Section*

D-shaped section is found to be suitable in tunnels located in good quality, intact sedimentary rocks and massive external igneous, hard, compacted, metamorphic rocks where the external pressures due to rock and water are not very large and where the lining is not designed to carry any external or internal pressures. The main advantages of this section over horse-shoe section are the added width of the invert which gives more working floor space in the tunnel during driving and flatter invert which helps to eliminate the tendency of wet concrete to slump and draw away from the tunnel sides. The added invert width also permits the use of concurrent lining of the tunnel which may not be possible for circular and horse-shoe tunnels of the same dimensions.

3.1.3 *Horse-shoe and Modified Horse-shoe Sections*

These sections are a compromise between circular and D-shaped sections. These sections are structurally strong to withstand external rock and water pressures. Where a moderately good rock is available and the tunnel has to resist internal pressures also, these sections are found to be most suitable. Where advantages of a flatter invert are required for constructional ease, modified horse-shoe sections are advantageous. These modified horse-shoe sections also afford easy change over to circular sections with minimum additional cost in reaches where rock quality is poor or rock cover is inadequate.

Figure 3.1 shows the geometrical properties of circular, D-shaped, horse-shoe and modified horse-shoe sections respectively.

3.2 Economic Diameter Studies

Having finalised the alignment and the geometric shape of the tunnel, the next step is to work out the economic diameter of the tunnel. The following factors should be considered while working out the economic diameter :

(a) Velocity requirements,
(b) Head loss in tunnel,
(c) Interest on capital cost of tunnel,
(d) Annual Maintenance charges,
(e) Whether lined or unlined, and
(f) Cost of gates and their hoists.

Permissible velocity in a concrete lined tunnel is of the order of 6 m/sec. Higher velocities are allowed for steel-lined tunnels. For diversion tunnels and tunnel spillways there is no limitation on the maximum permissible velocity provided that the lining (or the surface, if the tunnel is unlined) is adequate to withstand the velocities which would occur. The maximum permissible velocity shall be limited to approximately 3 m/sec in the case of tunnels conveying suspended abrasive materials.

Thus, for making economic diameter studies, a permissible velocity is assumed in the tunnel and losses are calculated as detailed in para 3.3. An example for working on the economic diameter of a tunnel is given in Annexure I.

3.3 Losses in Hydraulic Tunnels

Following are the types of losses generally occurring in a hydraulic tunnel :

(a) Friction Loss
(b) Trash Rack Loss
(c) Entrance Loss
(d) Transition Loss
(e) Bend and Junction Loss
(f) Gate Loss
(g) Exit Loss

Fach of the above losses are discussed in detail below :

3.3.1 *Friction Loss*

For tunnels flowing full friction may be calculated either by using Manning's formula or Darcy Waisbach formula :

Manning's Formula :

$$h_f = \frac{V^2 N^2 L}{R^{4/3}}$$

where h_f = head loss due to friction in m,
V = velocity of water in the tunnel in m/s
L = length of the tunnel in m,
R = hydraulic radius $\left(\frac{\text{Area}}{\text{Wetted perimeter}}\right)$ in m,
N = Rugosity coefficient.

For concrete lined tunnels the value of rugosity-coefficient N varies from 0.012 to 0.018.

For unlined tunnels, the value of N depends upon the nature of rock and the quality of trimming. Recommended values of N for various rock surface conditions are given below :

Surface Characteristics	*Value of N*	
	Minimum	*Maximum*
Very rough	0.04	0.06
Surface trimmed	0.025	0.035
Surface trimmed and invert concreted	0.02	0.03

Darcy-Waisbach formula :

$$h_f = \frac{fL}{D} \times \frac{V^2}{2g}$$

where h_f = head losss due to friction in m,
f = friction coefficient,
L = length of tunnel in m,
D = Diameter of tunnel in m,
V = Velocity of water in the tunnel in m/s and
g = acceleration due to gravity in m/sec^2.

The friction coefficient f depends upon the Reynolds number and the relative roughness K_s/D where K_s is the equivalent sand grain roughness and its value depends upon the surface characteristics. For new concrete lined tunnels using steel forms the value of K_s varies from 0.015 mm to 0.18 mm. For welded steel-lined tunnels, the value of K_s ranges from 0.05 mm to 0.1 mm.

Because of fluctuations in the load demand, the turbines keep an accepting or rejecting water. This causes the flow in the water conductor system to be turbulent. For turbulent flow and in the range of Reynolds number between 3000 to 10000 (normally expected in concrete lined and steel-lined tunnels), the friction factor f is calculated by using the formula :

$$\frac{1}{f} = 2.0 \log_{10} \frac{1}{2E} + 1.74$$

where f = friction factor (for use in Darcy formula)

$$E = \text{relative roughness} = \frac{K_s}{D}$$

For unlined tunnels, the value of f depends upon the variation in cross-sectional area obtained in the field. The frictional loss factor may be estimated by measuring cross-sectional areas at intervals and determining the value of f by the following formula :

$$f = 0.00257\,\delta$$

where $$\delta = \frac{A_{99} - A_1}{A_1} \times 100$$

A_{99} = area corresponding to 99 percent frequency, and
A_1 = area corresponding to 1 percent frequency

For tunnels of non-circular section, the diameter D in Darcy's formula shall be replaced by $4R$ where R is the hydraulic mean radius (A/P). The Darcy's formula shall thus read as follows :

$$h_f = \frac{fLV^2}{8gR}$$

For tunnels flowing partly full, the head loss due to friction will be calculated by using Manning's formula given earlier.

3.3.2 *Trash Rack Loss*

Tunnel openings are provided with trash racks at the intake to prevent the entry of floating debris into the tunnel. Where maximum loss values are desired, it is usual to assume 50 percent of the rack area as clogged. This would result in twice the velocity through the trash rack. Since the loss varies directly as the square of the velocity, it is desirable to limit the velocity at the intake to about 1 m per second for the worst conditions. For maximum trash rack losses, the racks may not be considered clogged when computing the head loss or the loss may be neglected altogether. The trash rack loss shall be computed by using the following formula :

$$h_t = K_t \frac{V^2}{2g}$$

where h_t = trash rack head loss,

K_t = loss coefficient for trash rack,

$$= 1.45 - 0.45 \frac{a_n}{a_t} - \left(\frac{a_n}{a_t}\right)^2$$

a_n = net area through the trash rack bars,

a_t = gross area of the opening

V = Velocity of water in net area, and

g = acceleration due to gravity.

3.3.3 *Entrance Loss*

To minimise the head losses and to avoid zones where cavitation pressures may develop, the entrance to a pressure tunnel should be steam-lined to provide a gradual and smooth changes in flow. For best efficiency the shape of the entrance should stimulate that of a jet discharging into air and should guide and support the jet with minimum interference until it is contracted to the tunnel dimensions.

For a circular tunnel the bell-mouth entrance shape may be approximated by an elliptical entrance curve given by the equation :

$$\frac{x^2}{(0.5D)^2} + \frac{y^2}{(0.15D)^2} = 1$$

where x and y are the coordinate axes and D is the tunnel diameter at the end of entrance transition. For head race tunnels since a gate is essential at the entrance, the opening shall be either rectangular or square. For such an opening the elliptical curve for the entrance shall be approximated by the equation :

$$\frac{x^2}{D^2} + \frac{y^2}{(0.33D)^2} = 1$$

Where D is the vertical height of the tunnel for defining the top and bottom curves and is also the horizontal width of the tunnel for defining the side curves.

For a rectangular entrance with the bottom placed even with the upstream floor and with curved side piers at each side of the entrance openings, both the bottom and side contractions will take place at the top of the opening. For such a case, the top curve may be obtained from the equation :

$$\frac{x^2}{D^2} + \frac{y^2}{(0.67D)^2} = 1$$

Where D is the vertical height of the tunnel downstream from the entrance.

The above three types of entrance transitions are shown in Figure 3.2.

Entrance loss shall be computed by the following formula :

$$h_e = K_e \frac{V^2}{2g}$$

where h_e = head loss at entrance,

K_e = loss coefficient for entrance,

V = velocity of flow

g = acceleration due to gravity.

The value of the K_e for circular bell mouth entrance varies from 0.04 to 0.10 with an average value of 0.05 and that for square bell mouth entrances varies from 0.07 to 0.20 with an average value of 0.16.

3.3.4 *Transition Loss*

In a hydraulic tunnel transitions are often required at the intake, junctions with de-silting chambers, gate galleries, surge shafts etc. and at outlets. All these transitions cause head loss in tunnels. To minimise the head loss and to avoid cavitation tendencies along the tunnel surfaces, the transitions should be gradual. Transitions can either be for contraction or for expansion.

For contractions, the maximum convergent angle should not exceed that given by the relationship

$$\tan \alpha = \frac{1}{U}$$

where α = angle of the tunnel wall surface with respect to its centre line,

U = an arbitrary parameter $= \frac{V}{\sqrt{gD}}$

V and D = average of the velocities and diameters at the beginning and end of the transitions, and

g = acceleration due to gravity.

Expansions should be more gradual than contractions because of the danger of cavitation where sharp changes in the side walls occur. Expansion angle should be based upon the following relationship :

$$\tan \alpha = \frac{1}{2U}$$

The notations are the same as given for contractions.

It has been noticed that head loss increases rapidly in the case of expansions where the angle α exceeds 10°. Hence, for all hydraulic tunnels and for pressure tunnels in particular, angle α must be limited to 10°.

Where a circular tunnel flowing partly full discharges into a chute or channel, the transition from the circular section to the one with flat bottom may be made either within the tunnel itself or in the open channel downstream from the tunnel portal. The length of the transition for exit velocities of upto 6 m/sec may be obtained by using the relationship :

$$L = \frac{2VD}{3}$$

where L = length of transition in m,
V = exit velocity in m/sec,
D = tunnel diameter in m.

For expanding transitions, the head loss is given by the following formula :

$$h_t = K_e \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g} \right)$$

where h_t = head loss in expanding transition in m,
V_1 = average velocity in m/sec at the beginning of transition
V_2 = average velocity in m/sec at the end of transition
g = acceleration due to gravity in m/sec^2
K_e = loss coefficient for expansion

$$= 3.50 \left(\tan \frac{\alpha}{2} \right)^{1.22}$$

α = angle of the tunnel wall surface with respect to its centre line.

For contractions, the head loss shall be computed using the following formula :

$$h_c = K_c \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g} \right)$$

where h_c = head loss in contracting transition in m,
V_2 = velocity in contracted section in m/sec,
V_1 = velocity in normal section in m/sec,
g = acceleration due to gravity in m/sec^2,
K_c = loss coefficient for contraction.

The value of K_c varies from 0.1 to 0.5. For gradual contractions where the flare angle does not exceed 10° the value of K_c shall be taken as 0.1.

3.3.5 *Bend and Junction Loss*

Bends and junctions in hydraulic tunnels are unavoidable owing to their functional and constructional requirements. These bends and junctions also cause loss of head which must also be computed.

Bend Loss : Bend loss depends upon the relative roughness K_s/D and r/d ratio, where K_s is the absolute roughness, D is the diameter of the tunnel and r is the radius of the bend. The head loss due to bend is given by :

$$h_b = K_b \frac{V^2}{2g}$$

knowing the values of K_s/D and r/d, the value of K_b for 90° bends may be obtained from Figure 3.3 and that for bends with deflection angles other than 90° from Figure 3.4.

3.3.6 *Junction and Branching Loss*

Head loss at the tunnel junctions and branching can be obtained from Figures 3.5 and 3.6 respectively.

Gate Loss : If the entrance to a tunnel is designed properly the velocity of flow would be approx. 1 m/sec. In such a case no gate loss need be considered. However, there will be head loss due to the gate groove and the same shall be given by :

$$h_g = K_g \frac{V^2}{2g}$$

where h_g = gate head loss in m,
K_g = loss coefficient for gate,
V = velocity of flow in m/sec, and
g = acceleration due to gravity in m/sec^2.

The value of K_g can be assumed to be 0.10. For partly open gates, the value of K_g will depend upon the top contractions and it varies from 0.20 to 0.10.

3.3.7 *Exit Loss*

Where no recovery of velocity head will occur, such as where the release from a pressure tunnel discharges freely or is submerged or supported on downstream floor, the velocity head loss coefficient K_{ex} shall be taken as unity. Head loss at the exit would be calculated by using the formula :

$$h_{ex} = K_{ex} \frac{V^2}{2g}$$

where h_{ex} = exit head loss in m,
K_{ex} = loss coefficient for exit,
V = exit velocity in m/sec,
g = acceleration due to gravity in m/sec^2.

3.4 Surges in Tunnels

Water hammer is created in long closed tunnels by the sudden closure of the turbine gates. The water hammer pressure provides the necessary force to retard the flow in tunnel when load is rejected by the turbine. For very long tunnels, the water hammer corresponding to normal operation of the turbine may be very great and may require extra ordinary strength of the tunnel to withstand it and the violent fluctuations of pressure in the tunnel may seriously interfere with proper turbine regulation. Similarly, for sudden opening of the gates, the resulting negative water hammer, or reduction of pressure, provides the necessary force to accelerate the water and is correspondingly objectionable for very long tunnels.

The simplest means of eliminating the positive and negative water hammer pressures is to provide a surge tank at the lower end of the tunnel. The steady state water level in the surge tank fluctuates up and down as the turbine rejects or accepts the load. With the help of digital computers it is very convenient to calculate the maximum and minimum pressures occurring all along the length of the tunnel. The tunnel should be designed to withstand the maximum excess pressure that is likely to occur. Similarly, it is very essential to determine the sub-normal pressures in the surge tank for sudden acceptance of the load. Care should be taken that the pressure in the tunnel never becomes negative as, under such conditions, the tunnel is likely to collapse. For head race tunnels flowing full, maximum pressures occur at the time of load rejection while minimum pressure can be expected at the time of load acceptance. However, for tail race tunnels flowing full, minimum pressures exist at the time of sudden load rejection whereas maximum pressures may occur at the time of load acceptance. Hence, to meet the safety requirements of a tail race tunnel, a surge tank may be provided downstream of the power house also.

3.5 Air Locking in Hydraulic Tunnels

Air may enter and accumulate in a tunnel by the following measures :

(a) During filling, air may be trapped along the crown at high points or at changes in cross-sectional size or shape.
(b) Air may be entrained at intake either by vortex action or by means of hydraulic jump associated with gate opening, and
(c) Air dissolved in the flowing water may come out of solution as a result of decrease in pressure along the tunnel.

The presence of air in a pressure tunnel can be a source of grave damage as detailed below :

(a) The localization of an air pocket at the high point in a tunnel or at a change in slope which occasions a marked loss of head and dimunition of discharge.
(b) The slipping of a pocket of air in a tunnel and its rapid elimination by an air vent can cause a water hammer.
(c) The supply of a mixture of air and water to a turbine affects its operation by a drop in output and efficiency thus adversely affecting the operation of generator.

The following steps are recommended to prevent the entry of air in a tunnel :

(a) Intakes should be designed properly. A shallow intake is likely to cause air being sucked in.
(b) Throughout the length of tunnel the velocity should remain constant or increase towards the outlet end.
(c) Partial gate openings resulting in hydraulic jumps should be avoided.
(d) Traps of pockets along high points and crown should be avoided.
(e) Thorough and careful surge analysis should be carried out to see that at no point on the tunnel section, negative pressures are developed.

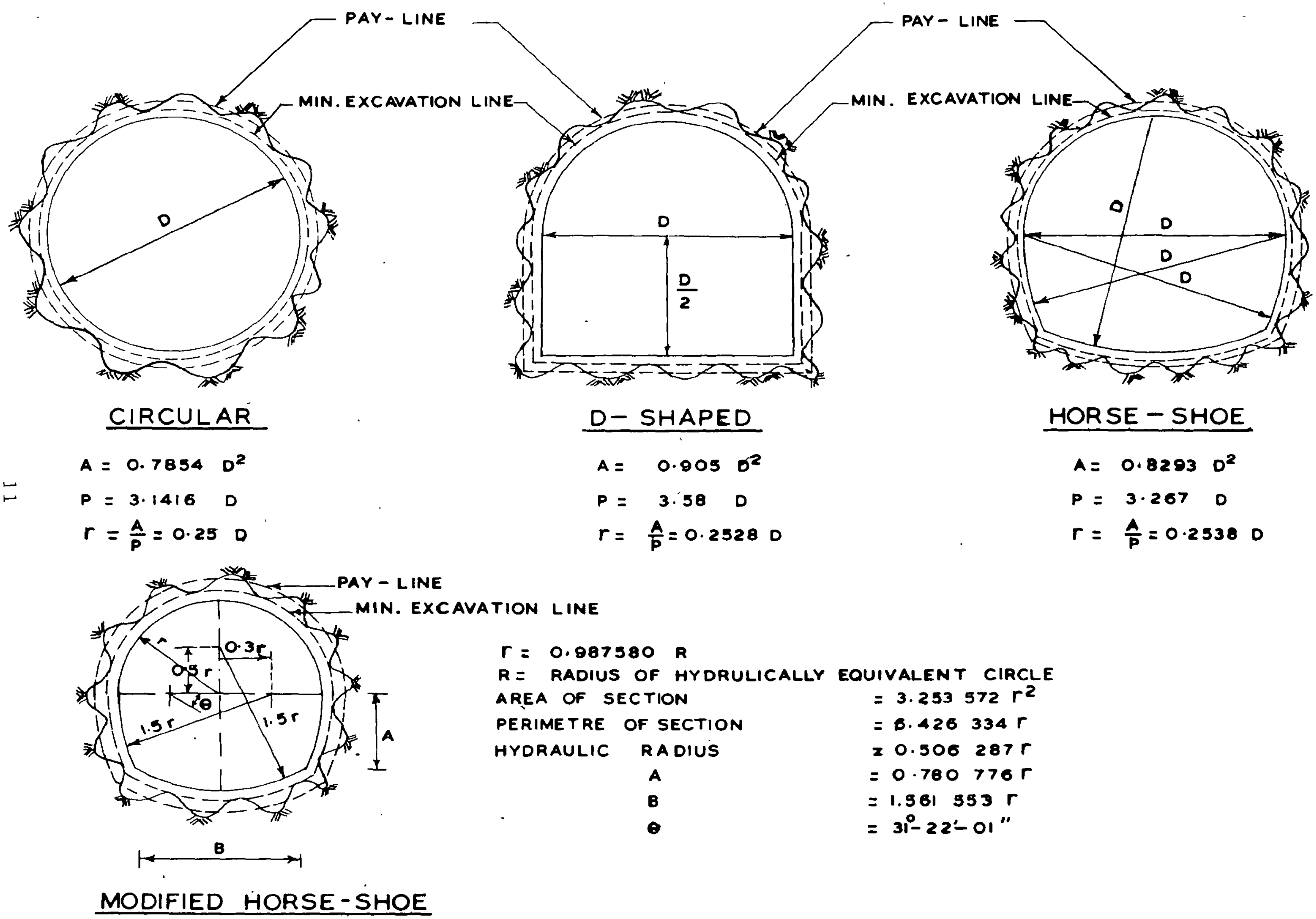
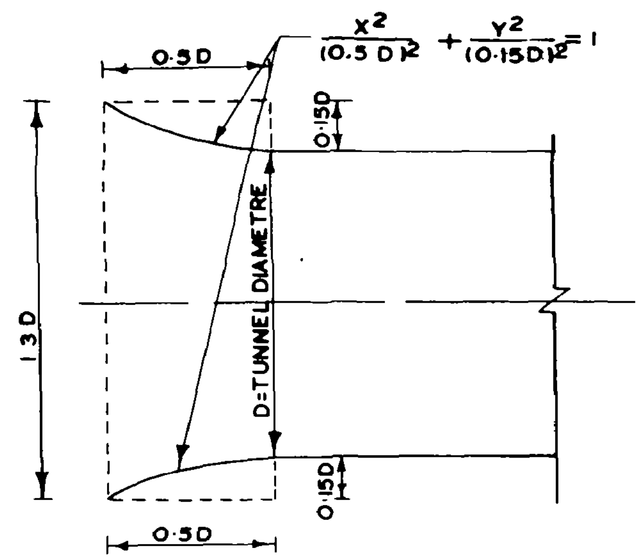
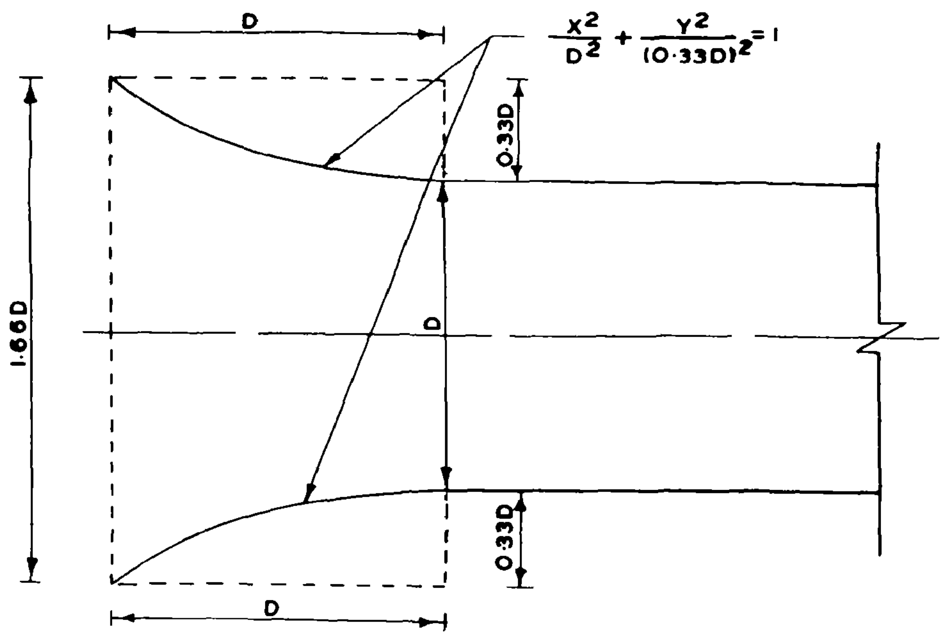


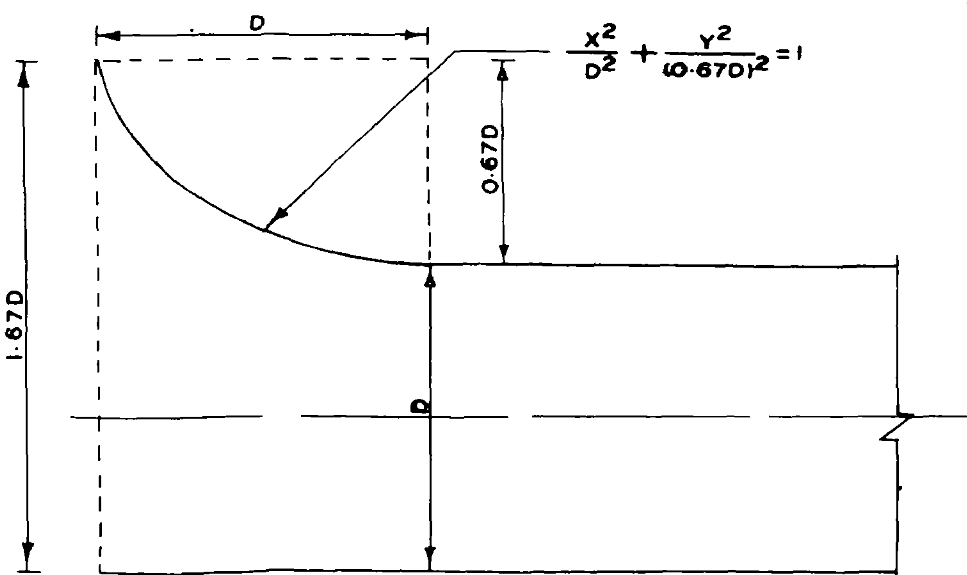
FIG. 3·1 GEOMETRIC PROPERTIES



CIRCULAR TUNNEL WITH BELLMOUTH INTAKE



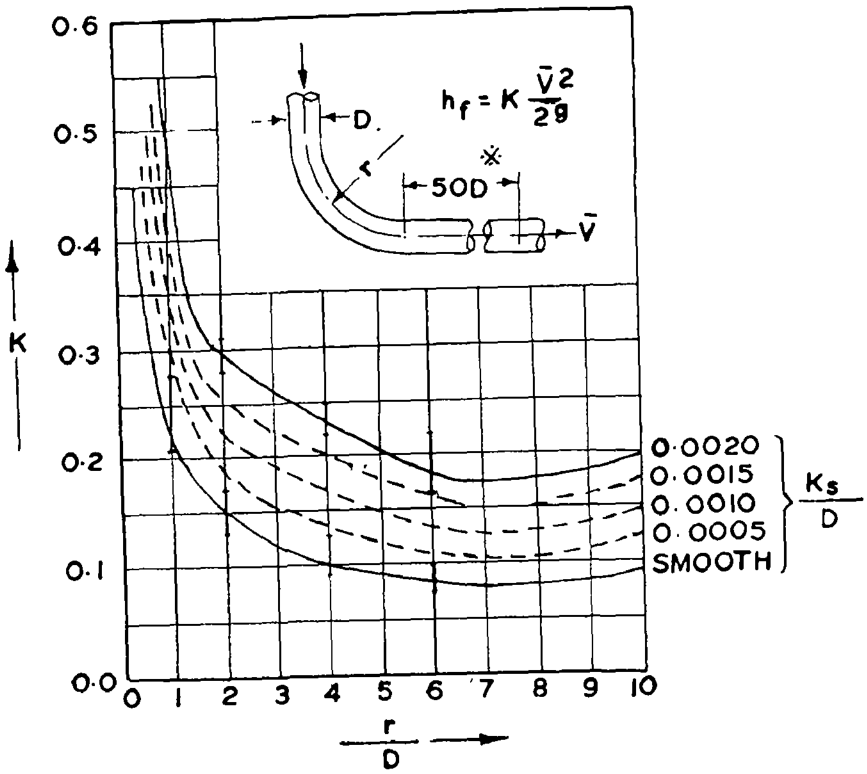
RECTANGULAR OR SQUARE OPENING



RECTANGULAR OR SQUARE OPENING WITH BOTTOM AND SIDE CONTRACTIONS SUPPRESSED

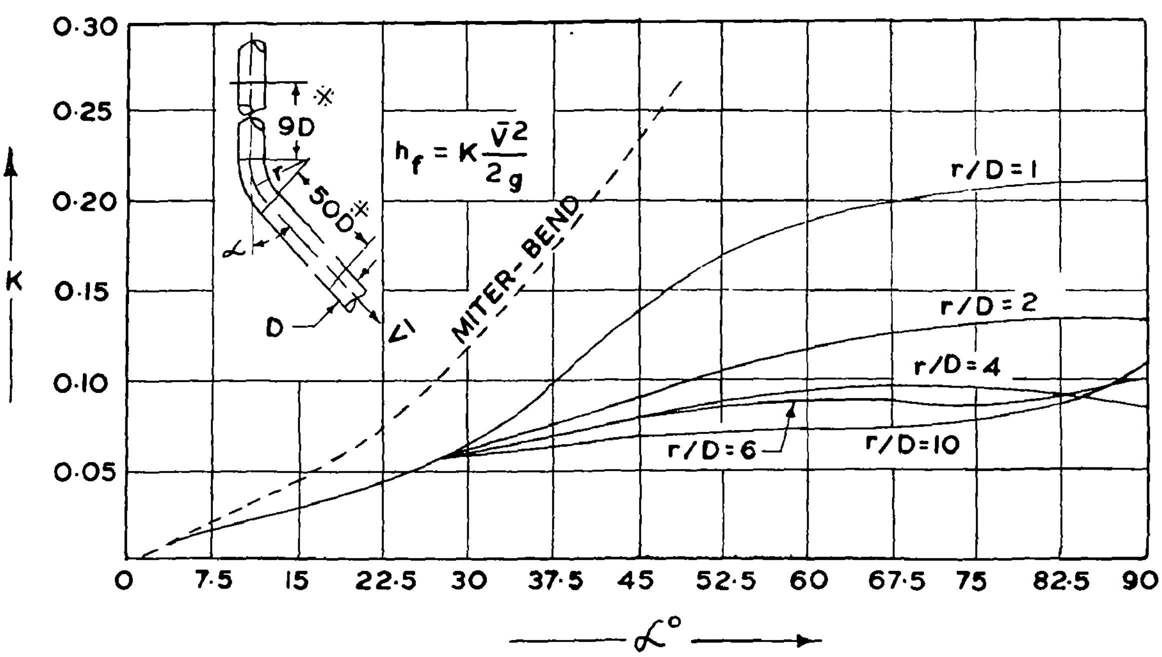
FIG.3.2 TYPES OF TRANSITIONS

FIG. 3·3 HEAD LOSS COEFFICIENT FOR 90° BENDS



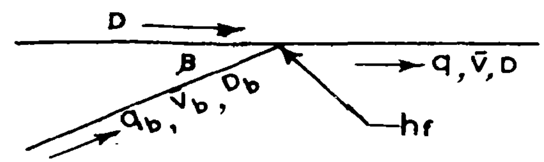
※ MINIMUM STRAIGHT LENGTH

FIG. 3·4 HEAD LOSS COEFFICIENT FOR BENDS OTHER THAN 90°



※ MINIMUM STRAIGHT LENGTH

FIG. 3.5 HEAD LOSS COEFFICIENTS AT JUNCTIONS



HEAD LOSS AT JUNCTION $(h_f) = K \frac{\bar{V}^2}{2g}$

WHERE

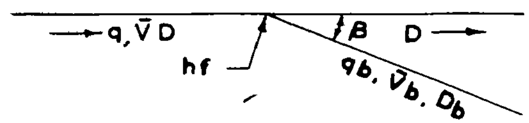
K IS RESISTANCE COEFFICIENT, AND

$q_b, \bar{V}_b, D_b$ AND $q, \bar{V}, D$ ARE DISCHARGE, AVERAGE VELOCITY AND DIAMETER OF AUXILIARY AND COMBINED PIPES RESPECTIVELY.

ANGLE OF CONVERGENCE β IN DEGREES	$q_b/q = 0.3$		$q_b/q = 0.5$		$q_b/q = 0.7$		$q_b/q = 1.0$	
	SHARP EDGED	ROUNDED	SHARP EDGED	ROUNDED	SHARP EDGED	ROUNDED	SHARP EDGED	ROUNDED
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
60	$D_b = 0.58D$ $\bar{V}_b = 0.9\bar{V}$ $K = 0.475$	$D_b = D.$ $\bar{V}_b = 0.3\bar{V}$ $K = 0.33$	$D_b = 0.58D$ $\bar{V}_b = 1.5\bar{V}$ $K = 0.637$	$D_b = 0.58D$ $\bar{V}_b = 1.5\bar{V}$ $K = 0.563$	$D_b = 0.58D$ $\bar{V}_b = 2.0\bar{V}$ $K = 0.715$	$D_b = 0.58D$ $\bar{V}_b = 2.0\bar{V}$ $K = 0.655$	$D_b = D$ $\bar{V}_b = \bar{V}$ $K = 0.645$	$D_b = D$ $\bar{V}_b = \bar{V}$ $K = 0.53$
45	$D_b = 0.58D$ $\bar{V}_b = 0.9\bar{V}$ $K = 0.2$	$D_b = 0.58D$ $\bar{V}_b = 0.9\bar{V}$ $K = 0.2$	$D_b = 0.58D$ $\bar{V}_b = 1.5\bar{V}$ $K = 0.425$	$D_b = 0.58D$ $\bar{V}_b = 1.5\bar{V}$ $K = 0.425$	$D_b = D$ $\bar{V}_b = 0.7\bar{V}$ $K = 0.540$	$D_b = D$ $\bar{V}_b = 0.7\bar{V}$ $K = 0.525$	$D_b = D$ $\bar{V}_b = \bar{V}$ $K = 0.38$	$D_b = D$ $\bar{V}_b = \bar{V}$ $K = 0.38$

NOTE — THESE VALUES ARE BASED ON THE EXPERIMENTS CONDUCTED AT THE HYDRAULIC LABORATORY OF THE TECHNICAL UNIVERSITY OF MUNICH, GERMANY, FOR MOST EFFICIENT CASE.

FIG. 3·6 HEAD LOSS COEFFICIENT AT BRANCHING



HEAD LOSS AT BRANCHING $(hf) = K \frac{\bar{u}^2}{2g}$

WHERE

K IS RESISTANCE COEFFICIENT, AND
$q, \bar{u}, D$ AND $q_b, \bar{u}_b, D_b$, ARE DISCHARGE, AVERAGE VELOCITY AND DIAMETER OF ORIGINAL AND BRANCH PIPES RESPECTIVELY

ANGLE OF DIVERGENCE β IN DEGREES	$q_b/q = 0.3$		$q_b/q = 0.5$		$q_b/q = 0.7$	
	SHARP EDGED	ROUNDED $r = 0.1 D_b$	SHARP EDGED	ROUNDED $r = 0.1 D_b$	SHARP EDGED	ROUNDED $r = 0.1 D_b$
(1)	(2)	(3)	(4)	(5)	(6)	(7)
90	$D_b = D$ $\bar{u}_b = 0.3\bar{u}$ $K = 0.85$	$D_b = D$ $\bar{u}_b = 0.3\bar{u}$ $K = 0.76$	$D_b = D$ $\bar{u}_b = 0.5\bar{u}$ $K = 0.87$	$D_b = D$ $\bar{u}_b = 0.5\bar{u}$ $K = 0.74$	$D_b = D$ $\bar{u}_b = 0.7\bar{u}$ $K = 1.00$	$D_b = D$ $\bar{u}_b = 0.7\bar{u}$ $K = 0.80$
60	$D_b = D$ $\bar{u}_b = 0.3\bar{u}$ $K = 0.7$	$D_b = 0.61D$ $\bar{u}_b = 0.8\bar{u}$ $K = 0.59$	$D_b = D$ $\bar{u}_b = 0.5\bar{u}$ $K = 0.59$	$D_b = 0.79D$ $\bar{u}_b = 0.8\bar{u}$ $K = 0.54$	$D_b = D$ $\bar{u}_b = 0.7\bar{u}$ $K = 0.57$	$D_b = D$ $\bar{u}_b = 0.7\bar{u}$ $K = 0.52$
45	$D_b = 0.58D$ $\bar{u}_b = 0.9\bar{u}$ $K = 0.43$	$D_b = 0.58D$ $\bar{u}_b = 0.9\bar{u}$ $K = 0.35$	$D_b = D$ $\bar{u}_b = 0.5\bar{u}$ $K = 0.42$	$D_b = 0.75D$ $\bar{u}_b = 0.9\bar{u}$ $K = 0.32$	$D_b = D$ $\bar{u}_b = 7\bar{u}$ $K = 0.34$	$D_b = D$ $\bar{u}_b = 0.7$ $K = 0.3$

NOTE :– THESE VALUES ARE BASED ON THE EXPERIMENTS CONDUCTED AT THE HYDRAULIC LABORATORY OF THE TECHNICAL UNIVERSITY OF MUNICH, GERMANY, FOR MOST EFFICIENT CASE.

CHAPTER 4

ESTIMATION OF ROCK LOADS AND PRESSURES

4.0 General

Rocks in nature are affected by the weight of the overlying strata and by their own weight. Stresses develop in the rock mass because of these factors. In general, every stress produces a strain and displaces individual rock particles. But, to be displaced, a rock particle needs to have space available for movement. While the rock is confined, thus preventing its motion, the stresses will be accumulated and may reach very high values—far in excess of their yield point. As soon as a rock particle acted upon by such stored stress is permitted to move, a displacement occurs, which may take the form either of "plastic flow" or "rock bursts" depending upon the formation characteristics of the rock material.

Thus, whenever underground cavities are excavated in rocks, the weight of the overlying rock layers will act as a uniformly distributed load on the deeper strata and consequently on the roof of the cavity excavated. The load acting thus is referred to as "rock load" or "rock pressure".

The determination of the magnitude of this rock load or rock pressure is one of the most intricate problem in rock mechanics. This complexity is due not only to the inherent difficulty of predicting the primary stress conditions prevailing in the interior of the non-uniform rock mass, but also to the fact that, in addition to the strength properties of the rock, the magnitude of secondary pressures developing after excavation around the cavity is governed by a variety of factors such as size of cavity, method of excavation, rigidity of support and length of period during which the cavity is left unsupported.

4.1 Rock Pressures

Terzaghi has defined this secondary rock pressure as the weight of the rock mass of a certain height above the tunnel soffit, which, when, left unsupported would gradually drop out of the roof. It, therefore, implies that if this amount of rock load that is likely to drop out of the roof of the tunnel, could be estimated, then the job of the engineer would be to design a system capable of supporting the above rock load.

Rabcewicz has classified the rock pressures into three main categories depending upon the reasons for their development. They are shown in Table 4.1.

It may be noted, however, that the conditions under which these loads develop, the probability of their occurrence and their magnitude differ greatly from one another and require the adoption of different construction methods. The possibility of their simultaneous action also cannot be excluded. Therefore, efforts should be made by way of thorough geological and geo-physical investigations of identifying each of the above types of rock pressures and designing the supports accordingly.

TABLE 4.1

Rock Pressure Development—Types and Reasons

Sl. No.	Type of Rock Pressure	Reasons for its Development
1.	Loosening Pressure	Loosening of the rock mass on account of blasting, scaling, mucking etc.
2.	Genunien Mountain Pressure	Weight of the overlying rock mass, ground water etc. and Tectonic forces.
3.	Swelling Pressure	Volume expansion of the rock mass, swelling due to physical and/or chemical action.

4.2 The Concept of Ground Arch and Terzaghi's Table

It has been observed that when a tunnel is excavated at shallow depths, the entire overlying strata (overburden) caves-in and the muck falls into the excavated cavity. Similarly, if two tunnels are excavated side by side and enough clearance is not kept between the two, the wedge has the tendency to fall thus causing unstable conditions. If, however, a tunnel is excavated reasonably deep below the ground surface and it is left unsupported, there is a particular limit upto which the ultimate overbreak extends as shown in Figure 4.1. The limit of this overbreak depends upon various parameters like stratification of deposits, type of rock, size of opening, etc. The above phenomenon was first studied by Terzaghi and later by Ikeda, Tanaka and Higuchi.

Solid rocks transmit the load acting on them by beam action to the sound supports while in loose and fractured rocks the load transfer to the undisturbed lateral parts is because of the friction developing during mass displacements. The load transferring formations developing in vertically and horizontally stratified rocks over supported cavities are shown in Figure 4.1, while the same is shown for loose, fractured rock in Figure 4.2. The process of load transfer continues till a natural ground arch is formed above the roof and on sides of the cavity which is capable of resisting the entire rock loads coming above this ground arch. Thus, it is the load of the rock mass bound with in the ground arch which is acting on the cavity and the job of the designer is to design a suitable support system to take care of this load. Based upon this concept and also from the experience gained on numerous rock tunnels in the Alps, Terzaghi gave his well known table of rock loads under various conditions (Table 4.2).

TABLE 4.2

Rock Load on Tunnels (After Terzaghi)

Sl. No.	Rock Condition	Rock Load	Remarks
1.	Hard and Intact	0 to 0.25 B	Requires no supports.
2.	Massive, Moderately Jointed	0 to 0.5 B	Requires Rock Bolts or light supports.
3.	Hard stratified	0 to 0.5 B	Requires Rock bolts or light supports.
4.	Moderately Blocky and Seamy	0.15 B to 0.35 $(B+H_t)$	Requires light to heavy ribs.
5.	Very blocky and Seamy	0.35 to 1.10 $(B+H_t)$	Requires Heavy Supports
6.	Completely crushed but chemically intact.	1.10 $(B+H_t)$	Requires continuous heavy ribs.
7.	Squeezing rock at moderate depth	1.10 to 2.10 $(B+H_t)$	Heavy ribs—conversion to circular section recommended.
8.	Squeezing rock at great depth	2.10 to 4.50 $(B+H_t)$	Heavy ribs—conversion to circular section recommended.
9.	Swelling Rock	75 m of rock load irrespective of the value of $(B+H_t)$	In extreme cases use yielding support.

Notes

1. Width of the opening = B and Height of opening = H_t.

2. The roof of the tunnel is assumed to be located below the water table. If it is located permanently above the water table, the values given for types 4 to 6 can be reduced by 50 percent.

3. If a rock formation consists of a sequence of horizontal layers of sandstone or lime stone and of immature shale, the excavation of the tunnel is commonly associated with a gradual compression of the rock on both sides of the tunnel, involving a downward movement of the roof. Furthermore, the relatively low resistance against slippage at the boundaries between the shale and the rock is likely to reduce very considerably the capacity of the rock located above the roof to bridge. Hence, in such rock formations, the roof pressure may be as heavy as in a very blocky and seamy rock.

However, it may be noted here that even though Terzaghi's classical work is, till today, the only classification which gives the exact quantum of load for which a tunnel is to be designed, it suffers from a basic draw back that the loads have been assumed for an unsupported cavity and no allowance has been made for the tendency of supporting elements in arresting the further rate of growth of rock loads after they have been installed. With the instrumented data now available from various tunnels in India and abroad, it has been observed that the loads actually developing and being carried by steel ribs are only about 40 percent to 50 percent of the loads calculated from Terzaghi's table. However, going by the existing tunnelling practices in India and in order to achieve an additional factor of safety, it would be advisable to go for the design of supports based upon the loads given by Terzaghi's table.

4.3 Other Methods of Rock Load Estimation

There are numerous other methods of rock load estimation based on the assumption of the formation of a natural ground arch above the cavity. These methods are detailed below :

4.3.1 *Bierbaumer's Method*

The theory of Bierbaumer was developed during the construction of great Alpine tunnels. According to this theory the tunnel is acted upon by a rock mass bounded by a parabola of height $h=\alpha H$ as shown in Figure. 4.3 where H is the height of the overburden above the crown of the excavated cavity.

For determining the value of reduction coefficient α, it is assumed that upon excavation of the tunnel, the rock material tends to slide along rupture planes inclined at $45°+\phi/2$, where ϕ is the angle of internal friction.

The base width B of the parabola of rock load is then computed using the formula :

$$B=b+2m \tan\left(45°-\frac{\phi}{2}\right)$$

where b = excavated width of cavity
and m = excavated depth of tunnel

The value of reduction coefficient α is given by the formula :

$$\alpha=1-\frac{\tan\phi\cdot\tan^2(45°-\phi/2)\cdot H}{B}$$

The reduction coefficient α has two limit values: For very small overburden depths $\alpha=1$.

For very large overburden or rock cover, whenever $H>5\,B$, the magnitude of α is no longer affected by depth and becomes :

$$\alpha=\tan^4(45°-\phi/2)$$

The maximum vertical pressure coming on the roof of the tunnel could then be assumed as :

$$p=\gamma h$$

where γ = bulk density of rock.

4.3.2 *Kommerell's Method*

Kommerell's Method is a further extension of the method developed by Bierbaumer. In his analysis, Kommerell assumed that the sliding surfaces are started from the lower corners of the walls at an angle of $45°+\phi/2$ extending to the crown and then continuing in a parabola of height h thus defining the zone to be supported by steel ribs etc. as shown in Figure 4.4. For computing the height h of the parabola, Kommerell used the equation developed earlier by Bierbaumer. For analysing the problem further, Kommerell adopted a graphical approach. As suggested by him, a diagram can then be constructed for the lateral pressures in the usual manner and the arch as well as the walls are separated to form individual members. The loads for each element of the arch can be obtained by projecting the limits to the diagrams horizontally and vertically as shown in Figure 4.5. The vertical load V is given by:

$$V = \frac{y_1 + y_2}{2} \cdot l_h \cdot \gamma$$

The horizontal load H is given by

$$H = \frac{x_1 + x_2}{2} \cdot l_v \cdot \gamma$$

where γ is the bulk density of the rock. In addition to V and H, each support element also has to carry its own weight G. These loads are plotted to some suitable scale as shown in Figure 4.5. The resultant R_1 of these forces can then be constructed as shown in Figure 4.5.

For the construction of horizontal pressure diagram, the end values e_1 and e_2 are required to be computed.

These values can be obtained using the following equations:

$$e_1 = h \cdot \gamma \cdot \tan^2 (45° - \phi/2) - 2c \tan (45° - \phi/2)$$

and $e_2 = (h + m) \cdot \gamma \cdot \tan^2 (45° - \phi/2) - 2c \tan (45° - \phi/2)$
where c is the coefficient of friction.

The term $2c \tan (45° - \phi/2)$ is so small in comparison to the first term in the above two equations that it is usually neglected while computing the values of e_1 and e_2.

4.3.3 *Triangular Loading Method*

In this method it is assumed that the rock loads would correspond to the weight of the rock confined within a triangle whose sides are sloping at $(45° - \phi/2)$ with the vertical, as shown in Figure 4.6. This method is analogous with the lintel design and is used for a very rough estimates of the rock load.

4.3.4 *Fenner's Ellipse Method*

In this method it is assumed that failure occurs along an elliptical surface enveloping the opening and passing through the springing of the arch. The rock loads on the roof are, thus, due to the weight of the rock between the ellipse and the roof intrados as shown in Figure 4.7. For massive, moderately jointed rock, the ellipse may be drawn for no tension condition. For blocky and seamy rock, the ellipse may be drawn for the boundary tangential stress equal to that existing prior to excavation of the cavity. In a biaxial stress field, the boundary tangential stress σ_t on the vertical axis is given by:

$$\sigma_t = S_h \left(1 + \frac{2q}{p}\right) - S_v$$

where S_h and S_v are the horizontal and vertical stresses and p and q, the horizontal and vertical axes of the ellipse respectively.

If $S_h = NS_v$,

$$\sigma_t = \left\{ N\left(1 + \frac{2q}{p}\right) - 1 \right\}$$

For Fenner's ellipse of no tension,

$$\sigma_t = 0$$

$$\therefore \quad \frac{q}{p} = \frac{1 - N}{2N}$$

For Fenner's ellipse with $\sigma_t = S_h = NS_v$

$$\frac{q}{p} = \frac{1}{2N}$$

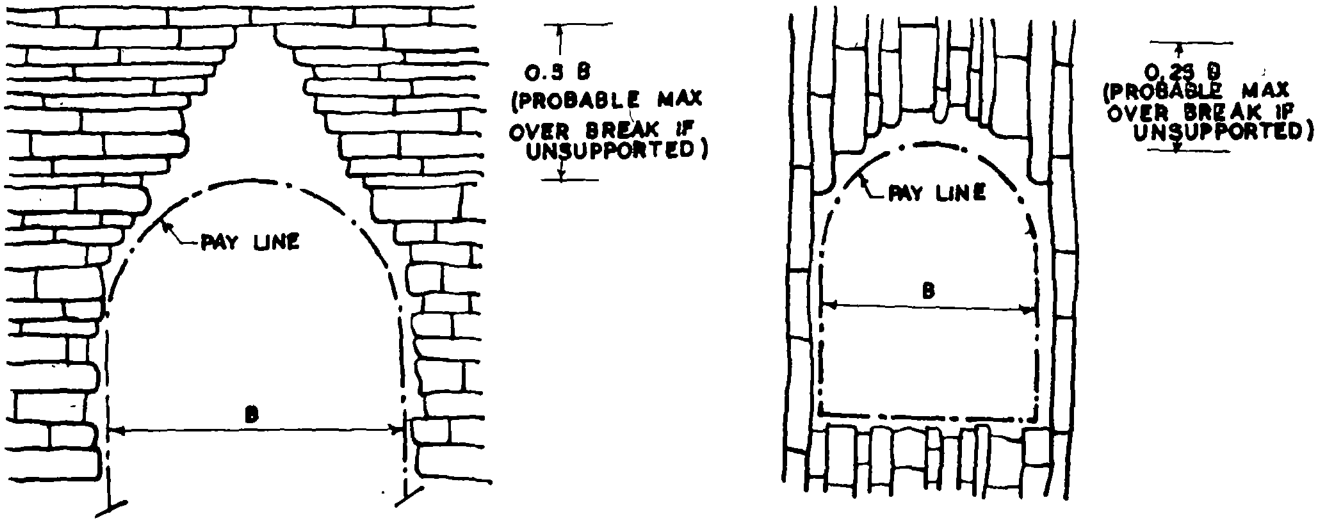
4.3.5 *Norwegian Method*

The Norwegian method consists of drawing a parabola from the springing of the roof and considering the rock load as due to the weight of the rock between the parabola and the roof intrados as shown in Figure 4.8. In most of the cases, the angle of tangent at the abutment is about 40°.

4.4 General Remarks

Irrespective of the method used for computing the rock loads, the most general method of structural design of steel rib supports is the graphical method developed by Proctor and White. This method of design has been explained under para 6.5.5 in Chapter 6 and using the above method, a design illustration has been given in Annexure II.

The modern tunnel designers, however, do not believe in supporting the rock load which is likely to be developed on account of the opening made in the rock. The present concept is that the rock be reinforced immediately after it is disturbed so that no scope is given for rock loads to develop. Based upon this concept, the CSIR, NGI and NATM methods of tunnel design have been evolved and these have been discussed in detail in Chapter 10.



OVERBREAK IN HORIZONTALLY STRATIFIED ROCK

OVERBREAK IN VERTICALLY STRATIFIED ROCK

FIG.-4.1 LOAD TRANSFERANCE IN STRATIFIED ROCK

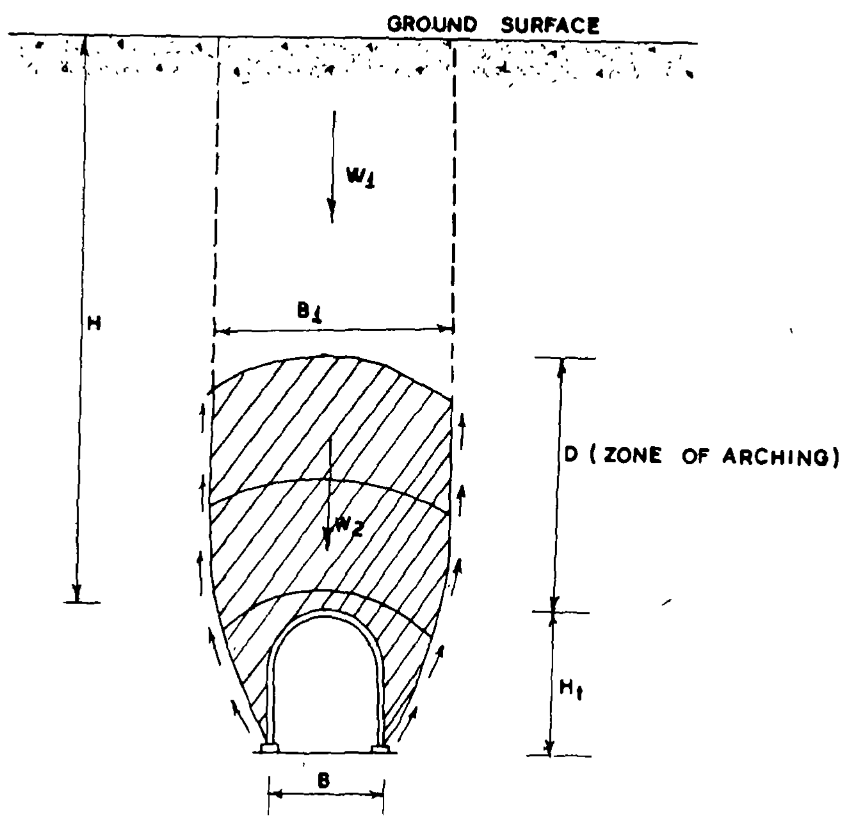
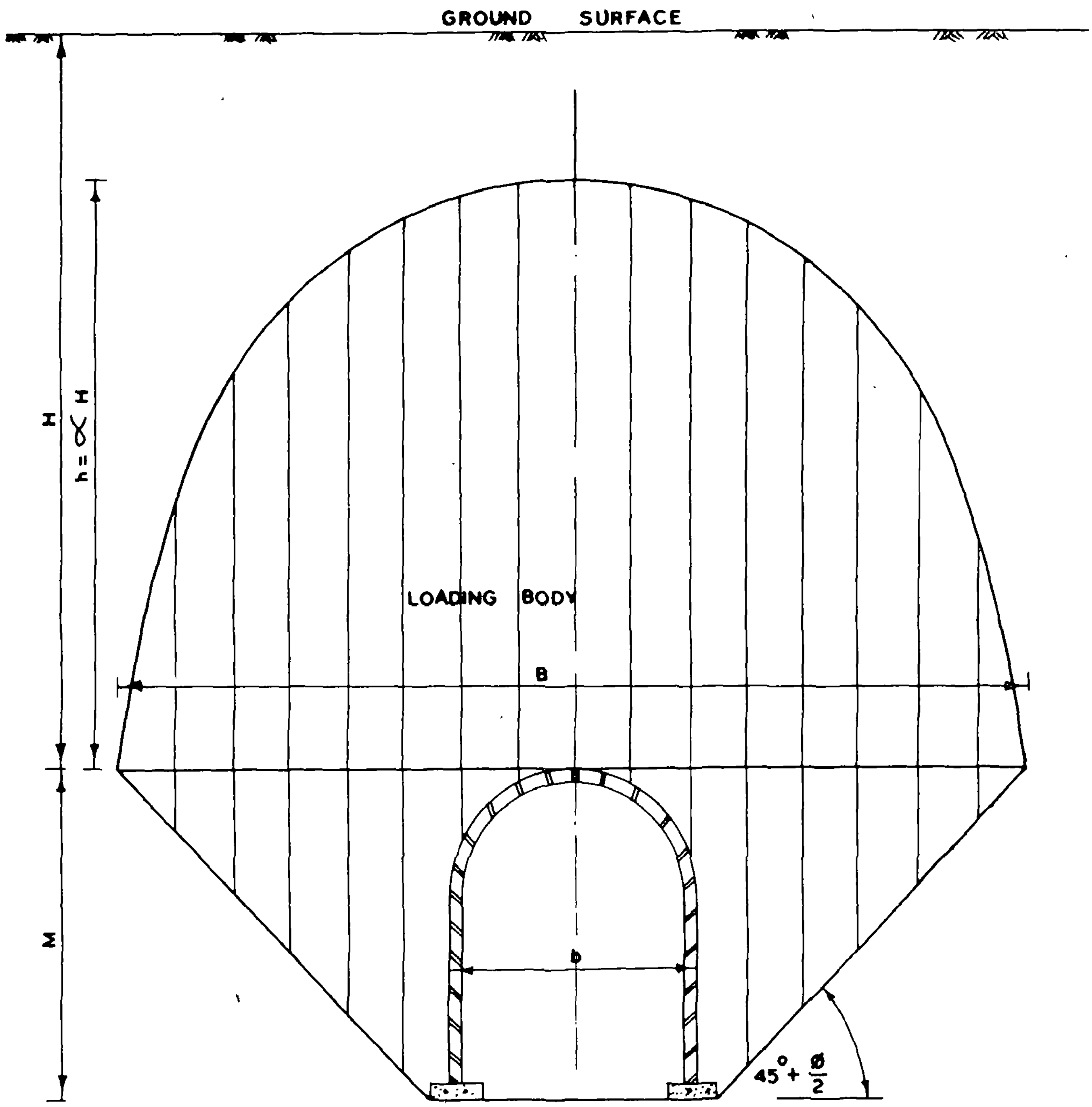


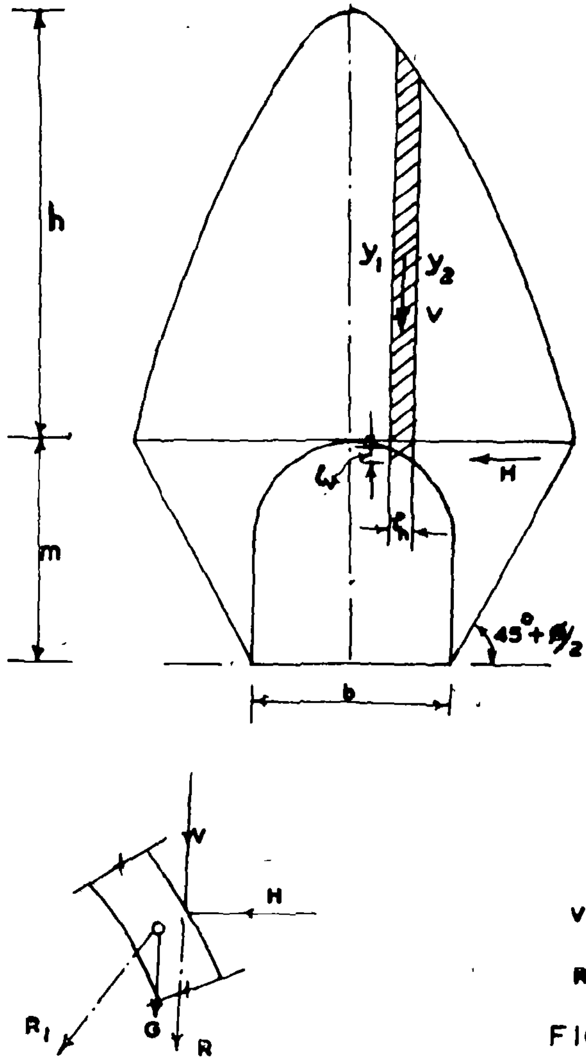
FIG-4.2 LOAD TRANSFERENCE IN LOOSE FRACTURED ROCK

FIG. NO. 4.3



COMPUTATION OF ROCK LOADS

BIERBAUMER'S METHOD



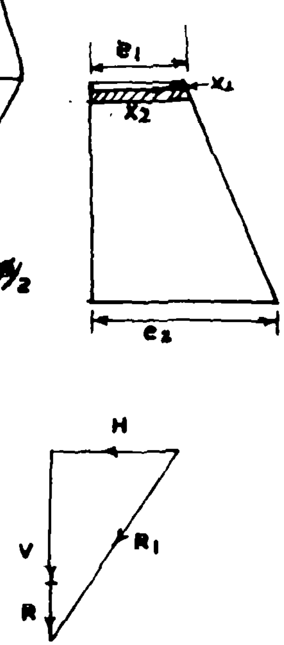


FIG.- 4.5

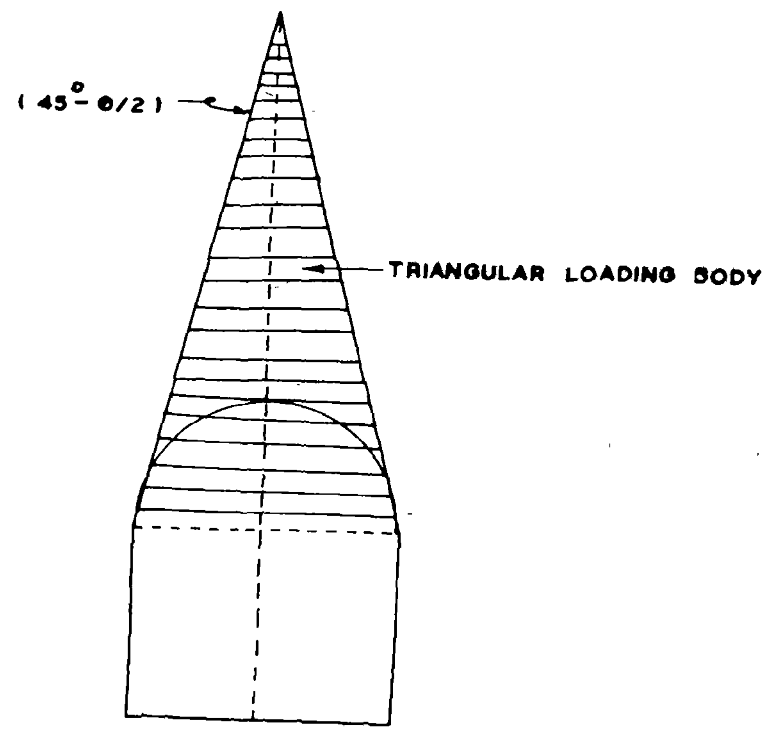
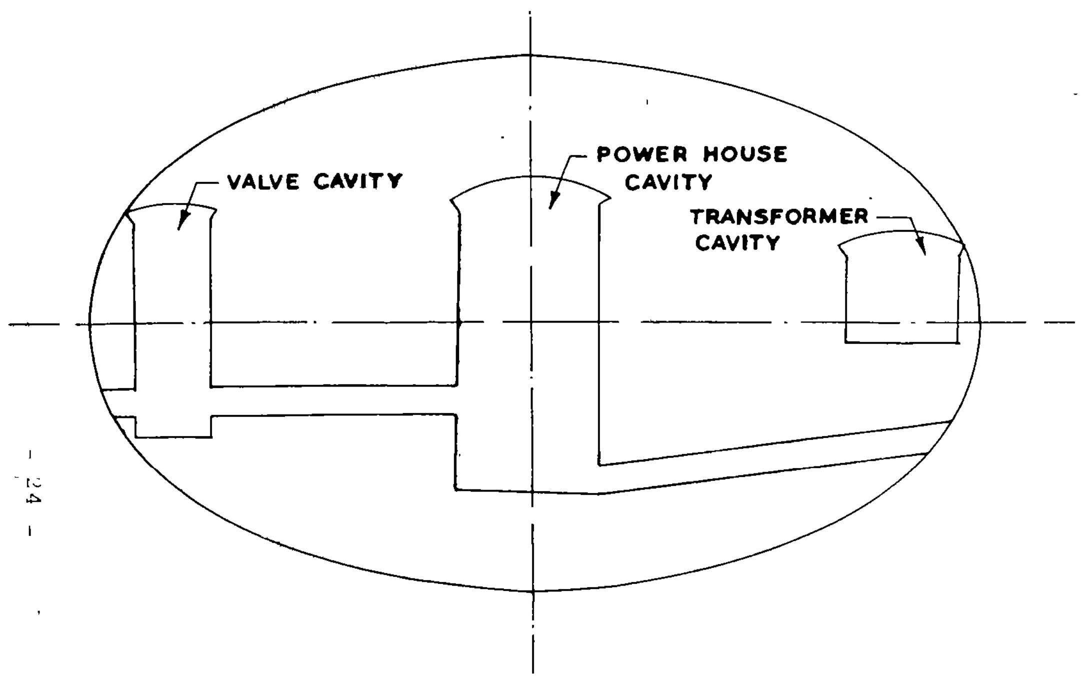


FIG.-4.4 ROCK PRESSURES (BY KOMMERELL) FIG. 4.6 TRIANGULAR LOADING METHOD



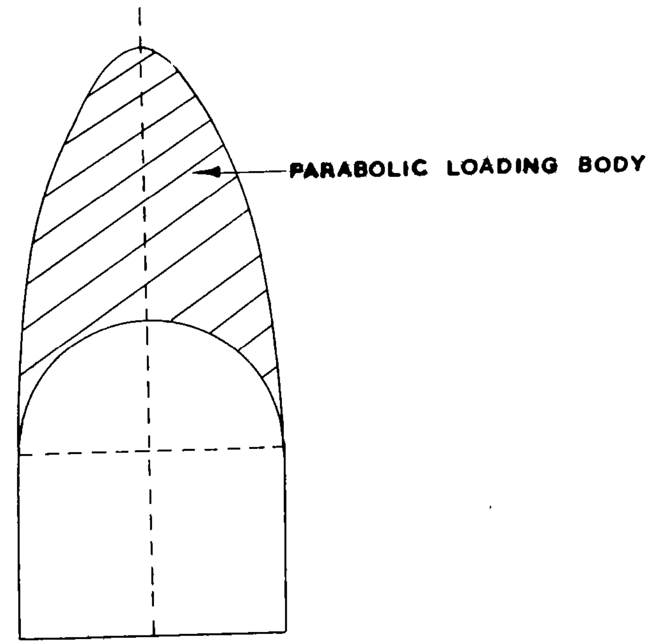


FIG. 4·8 NORWEGIAN METHOD

FIG. 4·7 TYPICAL SKETCH FOR CONSTRUCTION OF FENNER'S ELLIPSE AROUND CLOSELY SPACED CAVITIES

CHAPTER 5

EXCAVATION AND SUPPORT DETAILS

5.0 General

Tunnels can be driven through almost any material in nature, but the methods used and costs differ radically. Thus, the method used in tunnelling in earth, soft sediments or crushed whethered rock depends chiefly on the bridge action period of the material above the roof of the tunnel and the position of water-table, whereas the method used for tunnelling through hard, intact rock requiring little or no supports depends upon the strength and condition of rock. Because of the great longitudinal extent of the work many different kinds of conditions are encountered which for maximum economy should be excavated and supported differently. Hence, it is not possible to give detailed guidelines for each and every possible case and only the most commonly occurring conditions have been discussed in this chapter.

5.1 Preliminary Works

The preliminary works required to commence the excavation of a tunnel consist of the following:

(*i*) Precision survey and setting out of the tunnel alignment;
(*ii*) Open excavation in overburden and rock or excavation of a shaft from the bottom of which the tunnel excavation can start;
(*iii*) Arrangement for collection of surface water and its drainage by gravity flow or pumping;
(*iv*) Access roads or rail tracks to mucking areas;.
(*v*) Erection or winching and hauling equipments; and
(*vi*) Establishment of field workshop, compressors, pumps, water lines, ventilation fans, ducts, lighting, etc.

5.1.1 *Precision Survey and Setting Out*

The precision survey and setting out of the tunnel alignment shall consist of transferring the obligatory points like portal points, shaft location, etc., from topographical maps to the actual site of construction. This is done either by "Direct Setting Out" or by "Triangulation". In the mountainous regions it is an extremely rare possibility that both the ends of the tunne lwill be visible from each other and hence "triangulation" has to be invariably adopted for setting out the alignment. The levels of the various portals along the tunnel are then fixed accurately by means of "Reciprocal Levelling".

5.1.2 *Location of Portals*

A tunnel portal is the face from where a tunnel starts. Its location is decided with reference to the vertical and lateral rock cover. The minimum cover with which tunnel can be started depends upon the type and structure of rock, size and shape of tunnel and the internal water pressure. The length upto which it is economical to provide an open cut in preference to a tunnel depends on the cost of underground and open excavations and the cost of protection works involved. The general requirements and structural design of the portal have been discussed in detail in Chapter 8.

5.2 Methods of Tunnelling

Before proceeding further, it would be worthwhile to discuss a few common methods of tunnelling. The methods of attacking the faces of a tunnel depends upon the size and shape of the tunnel, the rock characteristics, the equipment available, the support system envisaged for the tunnel, and the overall economics. Following are the methods commonly adopted.

5.2.1 *Full Face Attack*

In this method, the entire cross-sectional area of the tunnel to be excavated is attacked simultaneously. This method is generally recommended for small size tunnels and tunnels in good rock conditions where major rockfalls are not anticipated.

5.2.2 *Top Heading and Benching*

Where the tunnel has a very large cross-sectional area or where the rock is not of good quality, the top heading and benching method is generally recommended. In this method, a top heading is excavated first—either to full length or part lengh of the tunnel, and is supported simultaneously. The benching is then removed slowly. The method is explained with the help of Figure 5.1.

5.2.3 *Bottom Heading and Stoping*

Where the rock is consistent and sound and the tunnel section is very large, this method can be easily adopted. In this method a bottom heading is made first and the overhead stope is removed later on as shown in Figure 5.2.

5.2.4 *Drift Method*

In driving a large tunnel it may be economical to drive a small tunnel called a drift or a pilot tunnel prior to excavating the full face. Depending upon the nature of rock and other parameters, a drift may be excavated in the centre, side bottom or top as shown in Figure 5.3.

Apart from these methods, some other methods have been discussed in Chapter 9 on "Special Problems in Tunnelling".

5.3 Steps Involved in Tunnelling Operation

The actual steps involved in boring the tunnel depend upon the mode of tunnel boring. The most commonly adopted method of tunnel boring in India is the "conventional drilling and blasting method". Other methods used are the "shield tunnelling" and the "road heading machine" methods utilizing the Alpine miner.

5.4 Sequence of Operations for Construction of Tunnels

After the tunnel alignment has been marked on the site and portals and/or shafts constructed, the actual tunnelling operations start. The sequence of operations for construction of tunnels using the conventional drilling and blasting method are as under :

(*i*) Setting up of drilling jumbos and drilling of holes,
(*ii*) Loading the drilled holes with explosives and blasting of holes,
(*iii*) Defuming and ventilating the tunnel,
(*iv*) Checking misfires, if any,
(*v*) Scaling the loose material,
(*vi*) Removal of the muck, and
(*vii*) Erection of support system and lining.

5.4.1 *Setting up and Drilling*

Holes are drilled by using pneumatically operated drills in conjunction with pneumatic pushers etc. mounted on drilling jumbos. The pattern of drilling and the number of drill holes is governed by the strength of rock, size and shape of tunnel, strength of explosives and the fragmentation of rock required so as to make it suitable for mucking. As a thumb rule, one drill hole 4 to 5 m^2 of face area may be provided. Diameter of the drill hole may be kept at least 6 mm more than the diameter of the explosive cartridge.

Generally, three types of drill holes are provided in any drilling pattern, viz., cut holes, easers and trimmers. The cut holes are usually provided in the centre of the round about 15 to 30 cm deeper than the other holes and are drilled converging towards the centre of the face with the idea of producing an initial cone or wedge as a free face for breaking-off succeeding holes. The other holes, namely the easers and trimmers are placed around the cut holes and are fired in that order so as to induce a perfect fracture with minimum overbreak.

The most commonly used pattern of drilling is the "horizontal wedge cut". In this pattern, holes are placed symmetrically with respect to the vertical centre line of the section. The drill holes are horizontal and the angle towards the working face is large, and, therefore, is easy to drill. This pattern of drilling shown in Figure 5.4 is almost universly applicable. The other types of pattern adopted for different conditions of excavation are shown in Figures 5.5, 5.6 and 5.7.

5.4.2 *Loading and Blasting*

Explosives used in blasting are generally made of gelatine in varying percentage depending upon the strength of rock to be blasted. Before loading is started, each hole is blown out with a high pressure air jet to remove loose cuttings and water. The explosive, which is available in the form of cartridges ranging from 25 mm to 63 mm in diameter and 200 mm to 245 mm in length is then inserted in the hole and tamped well into the bottom with the help of a wooden pole. A primer cartridge containing the detonator pointing towards the bottom of the hole and tamped lightly to prevent jarring of the detonator. The remaining cartridges are then lowered into the hole and then tamped firmly in place taking care to prevent breaking of detonator lead wires. Finally, the remainder of the hole not occupied by explosives is filled with an inert material (like a damp mixture of clay and sand) and tightly tamped. The holes are then blasted from a safe distance.

5.4.3 *Defuming and Ventilating*

The tunnel is allowed to defume and all foul gases and dust particles are driven out by blowing fresh air into the tunnel through blowers.

5.4.4 *Checking Misfires*

Immediately after the tunnel has been defumed and ventilated, the blasted face is checked carefully for any misfires. For this an experienced and competent Foreman enters the tunnel, removes the loose rock carefully and makes sure that all cartridges have been fired.

5.4.5 *Scaling the Loose Material*

After the checking of misfires is over, workers enter the tunnel and remove all loose material by the use of crow bars.

5.4.6 *Mucking*

The material removed as a result of blasting is loaded into tippers, dumpers or mine cars, as the case may, and is taken out of the tunnel.

Depending upon the "bridge action period" or the "stand up time" of the excavated rock, efforts are made to install the supports and to make the face ready for the next sequence of operations.

5.5 Tunnel Supports

When an underground opening is made, it generally becomes necessary to install supports to hold the rock which has a tendency to drop out of the roof of the opening. In the earlier days timber sections were used as temporary supported till permanent lining could be placed. With the gradual availability of steel sections, timber supports have now become almost obsolete. More recently on the basis of work done by Beiniawaski, Barton, Rabcewicz and others, even steel supports are being dispensed with and the present trend is to reinforce the rock by means of rock bolting and shotcreting.

5.5.1 *The Need for Tunnel Support*

The necessity of tunnel support arises from the fact that the excavated rock has a tendency to drop out of the roof of the tunnel. The time which the loosened rock takes to drop out and also the amount of rock expected to fall depend upon the "bridge action period" of the rock. The bridge action period "t_b" is defined as the time which elapses between blasting and the beginning of collapse of the unsupported roof. It may range from a few hours to a few weeks. The bridge action period for cohesionless sand or completely crushed rock is almost zero. Hence, if a tunnel passes abrupty from fairly sound rock into such materials, excessive overbreaks at the point of transition are inevitable. Such an accident took place in the Blue Mountain Tunnel Near Carlisle where a wide seam filled with sand and water was encountered. As soon as the rock partition between the rock and the seam was blasted, water and sand flooded the tunnel finally resulting in the formation of a 30 feet high chimney. More recently an accident of this nature occurred during the excavation of Chukha head race tunnel where immediately after the accident took place, sand and water flow of the order of 30 to 50 cusecs was observed.

The importance of bridge action period could be understood from Figure 5.8. which shows the various operations involved in the conventional drilling and blasting method. It will be seen from this figure that excessive overbreaks would be inevitable if t_b is less than the ventilating time t_v. Considerable overbreaks would be inevitable is t_b is greater than t_v but less than t_m, the time required for mucking. The construction engineers must fell happy of t_b lies within t_m and t_s, the time required for support erection. And there should be no cause for major worries if t_b happens to be greater than t_s.

However, if t_b happens to be quite large, one should not misunderstand that the tunnel needs no supports because of the fact that the rock loads go on developing for weeks and months together after the tunnelling operations are over. The restraint on the development of rock loads by the provision of supports can be easily understood from Figure 5.9. In this figure, the unit load at the crown of the supports is plotted on the ordinate while the time is plotted along the abscissa. As soon as the excavation is made, a small amount of unit load (H_o) develops at the crown of the tunnel. The support which is designed for the maximum probable load H_{max} is installed within the bridge action period. At this instant the load acting over the support is because of wedging action alone. The job of tunnel supporting does not end with the provision of a support alone. It should also be ensured that the support is immediately and carefully back packed with concrete. Care should be taken to fill up all the gaps between the excavated surface and the rib. This should then be followed by pressure grouting and contact grouting to seal all the interstices left in the concrete. The importance of a careful back packing could be judged by having a look at the load versus time curve shown in Figure 5.9. It will be seen that the ultimate load developed at the crown of the support is much lesser for a carefully back packed tunnel than a poorly back packed one. It would also be seen from the same figure that if the cavity is left unsupported, H goes on increasing to such an extent that it exceeds the crushing strength of the rock and at that stage the crown caves in resulting in dome formation. Such an accident took place in the Chukha Power House cavity where the cavity (25.5 m span) was left unsupported for 35 days and the accident occurred on the 36th day bringing down about 800 cu m of rock and resulting in the formation of a dome with a maximum height of about 6 m near the crown (Figure 5.10).

5.5.2 *Types of Steel Support Systems*

Tunnel support system by way of steel ribs may be classified into the following :

(*a*) Continuous Rib;
(*b*) Rib and Post;
(*c*) Rib and Wall Plate;
(*d*) Rib, Wall Plate and Post
(*e*) Full Circle Rib; and
(*f*) Invert strut in addition to those shown in types (a) to (d).

The above systems of support are shown in Figures 5.11, 5.12 and 5.13.

5.5.3 *Selection of the Type of System*

The selection of a particular type of system proposed to be installed depends upon the method of attack, rock characteristics and tunnel cross-section. Figures 5.11, 5.12, 5.13 shows the various types of support system and their suitability with regard to the method of attack, rock characteristics and tunnel cross-section.

5.5.4 *Constituents of Tunnel Supports*

Every type of tunnel support system consists of two or more different elements, each of which serves a different function. The basic elements are:

(a) *Ribs* : Ribs are made of structural steel. In India H-beams or I-beams are being used whereas in Europe and Scandenavian countries, U-beams are general used.

(b) *Posts* : The posts should preferably be H-beams and their spacing is kept same as the spacing between the ribs. The spacing of posts could be increased if wall plates are used.

(c) *Invert Struts* : Where side pressures are present, it is necessary to prevent the inward movement of the rib or post feet by the provision of invert struts. The struts may be curved to form an inverted arch where upthrust from the floor of the tunnel is expected.

(d) *Wall Plates* : Wall plates serve the purpose of transmitting loads from the ribs on to the blocks or posts and to expedite the erection of supports. The types of wall plates commonly used are the double beam wall plates; single beam wall plate, and flat wall plate shown in Figure 5.14.

(e) *Crown Bars* : Crown bars are structural elements meant to support the roof immediately after ventilation and thereby gain time for the installation of ribs. Thus, they find use in cases where the stand up time of the rock or bridge action period is small. The crown bars may be made of double channels or H-beams or Square timber beams. They may be either supported over the steel rib or suspended from the rib as shown in Figures 5.15 and 5.16.

(f) *Truss Panels* : These are accessories used for supporting heavy roof loads temporarily for the heading and benching or top heading methods. Their purpose is to form, in combination with the ribs, a truss to span the gap produced by the bench shot. The truss panels are attached to the inside face of the ribs for a distance of one or more ribs ahead of the bench shot as shown in Figure 5.17 and are left there until posts are installed, at which time, they are removed and sent ahead: The connection should be merely by means of two bolts at each rib.

(g) *Bracings and Spreaders* : In case no laggings have been provided bracings and/or spread should be provided to increase the rib resistance to buckling and to prevent a displacement of ribs and posts during blasting. The bracings and spreaders commonly used are shown in Figure 5.18.

(h) *Blocking* : If an underground opening is provided with a tight, back-packed lagging, the rock loads act uniformly on the entire rib. However, in most instances, the rock loads are transferred to the ribs by a relatively small number of blocks which are inserted between the rock and the outside of the ribs. These blocks could be of timber or precast concrete. The places where these blocks bear on the ribs are known as blocking points. At any blocking point, the rib is acted upon by an external force and the block is capable of transmitting this force to the rib only in a direction normal to the tangent at its point of contact with the rib. Hence, the vertical rock loads are resolved into two components one of which is radial. Only this radial component is considered for design purposes.

If the loads are applied to the ribs only at widely spaced points, the rib is acted upon not only by axial thrust but also by bending moments thus decreasing the load carrying capacity of the ribs. The bending moment increases as the square of the distance between blocking points. Therefore, the spacing between the blocking points deserves careful attention. Closer the spacing between the blocking points, lesser is the induced stress in the rib as shown in Figure 5.19.

The rib and the spacing between the blocking points must be dimensioned on the basis of the most unfavourable rock conditions which may prevail in the field. Therefore, the design is based on the maximum spacing between the blocks which is tolerable in the opening. Thus, the spacing between the blocks is dependant upon the type of rock and width and rise of the opening.

(i) *Lagging* : Where rock excavation is made in such conditions that rock falls or spalling occurs, it will be hazardous and dangerous to the workers inside the tunnel. In such cases, the space between two successive ribs is gapped by providing steel, timber or precast concrete lagging slabs. Another function of lagging is to provide a convenient surface against which to place back packing and to serve as an outside form for concreting between the ribs and the rock. Figure 5.20 shows the different types of lagging commonly used.

(j) *Back-packing* : The type of back packing and its function depends on the rock condition. In dry tunnels through jointed rock, packing is only used to fill large cavities produced by excessive overbreaks. In broken, crushed or decomposed rock, it plays a very important function—that of restraining the development of rock loads The restraint on the development of rock loads has already been explained earlier.

The back-packing in the tunnel could be either of dry packed tunnel spoils shoveled or hand packed into the space between the lagging and the rock or pea-gravel packing or concrete. Where considerable rock loads are expected, concrete back-packing is always recommended.

5.5.5 *Design Procedure*

Having assessed the rock loads and the spacing of the blocking points assumed suitably, the actual design of steel supports required could be taken up. The design is carried out graphically by following the stepwise procedure developed by Proctor and White.
Following are the steps :

(*i*) Construction of load diagram;
(*ii*) Construction of force polygon;
(*iii*) Determination of thrusts, and
(*iv*) Computation of stresses in the arch rib.

Using the above procedure the design of an underground power house opening has been worked out and is given in Annexure-II.

5.6 Rock Bolting and Shotcreting

In the earlier days timber sets were being used as permanent supports for the tunnels. With the increased availability of steel sections, the emphasis was shifted to these sections as they were found to be more strong, longer lasting and ideal for tunnel supporting in almost all types of situations. However, as more and more tunnels were excavated and more data became available to the engineers world over, new methods of tunnel supporting were evolved. The feeling now is that any opening can be self supporting permanently if the rock around it is suitably reinforced so that the reinforced rock becomes a competent structural entity. The new means of rock reinforcement are the rock bolts and shotcrete.

The early view was that the work of rock bolts was to "pin" or "nail" blocks/slabs of rock to the sounder rock behind them. Rock bolts have been so used for a very long time, but their use for this purpose without understanding can be dangerous. The hazard is greatly increased where "spot bolting" is used with large spacing between the bolts.

Shotcrete, which is defined as pneumatically applied mortar or concrete, is used to protect and support zones of fractured, crushed, disintegrated or spalling rocks, and to preserve and prevent further deterioration caused by the action of water or atmosphere or the effects of time.

Wire mesh and shotcrete applied to the surface of the rock in conjunction with rock bolts form part of the rock reinforcement system and have a very real structural significance. Methods are now available to design the rock bolts and shotcrete for supporting the cavity and these have been described in Chapter 10 "Recent Advances in Tunnelling".

HEADING & BENCHING METHOD

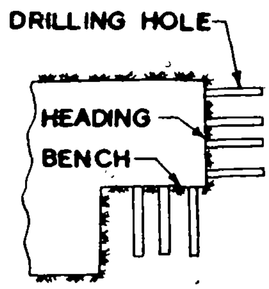


FIG. 5·1

BOTTOM HEADING & STOPING METHOD

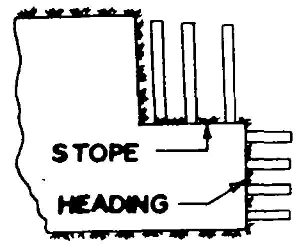
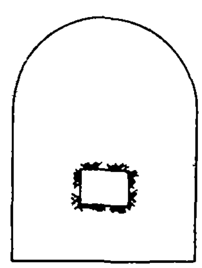


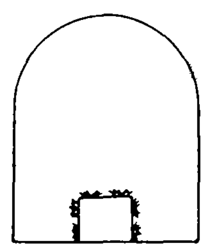
FIG. 5·2

DRIFT METHOD

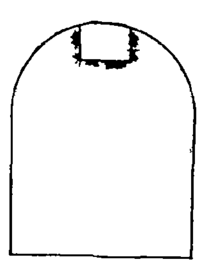
CENTRAL DRIFT



BOTTOM DRIFT



TOP DRIFT



SIDE DRIFT

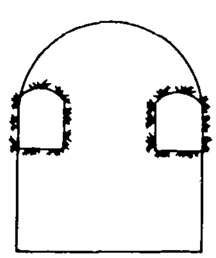


FIG. 5.3

HORIZONTAL WEDGE CUT

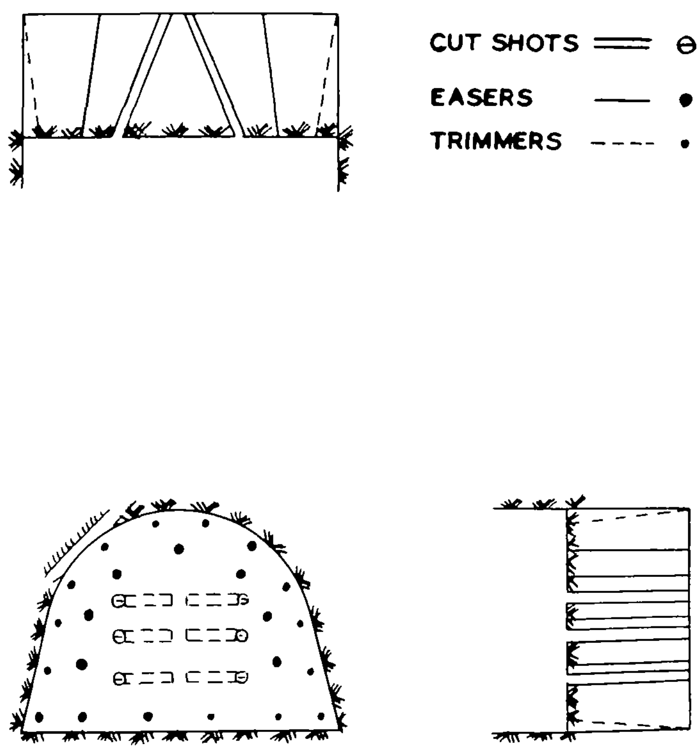


FIG. 5.4

PYRAMID CUT

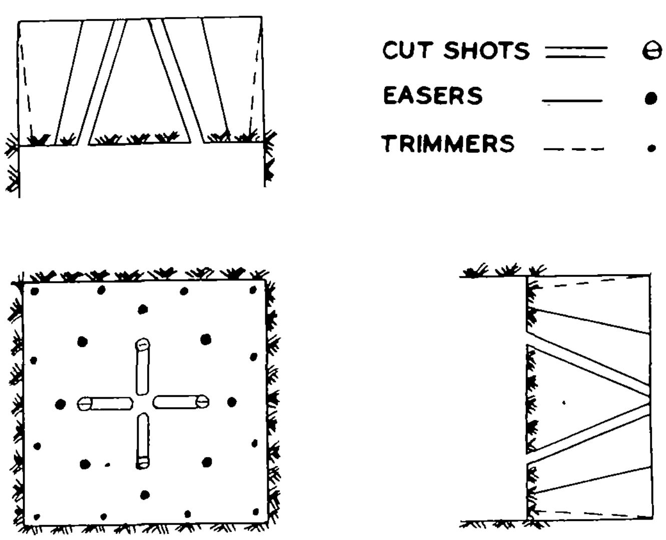


FIG. 5.5

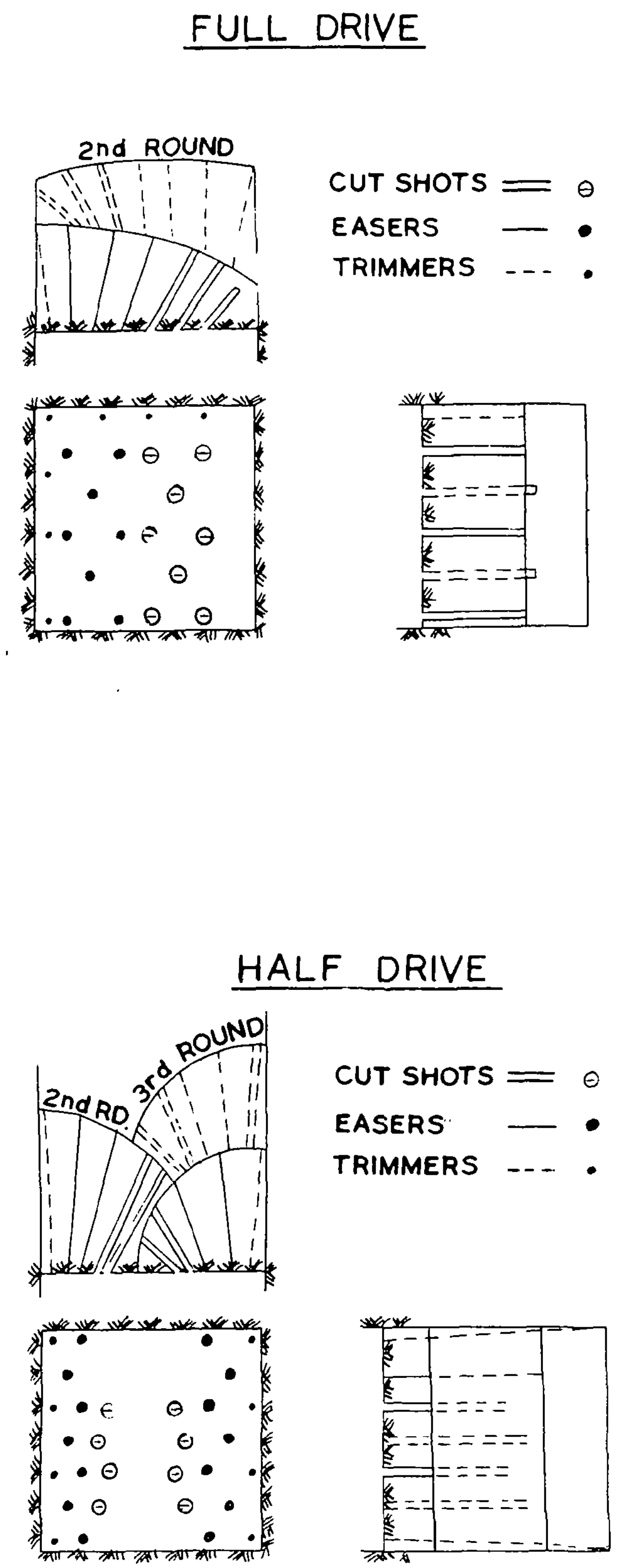

FULL DRIVE
2nd ROUND
CUT SHOTS
EASERS
TRIMMERS
HALF DRIVE
2nd RD.
3rd ROUND
CUT SHOTS
EASERS
TRIMMERS

FIG. 5.6

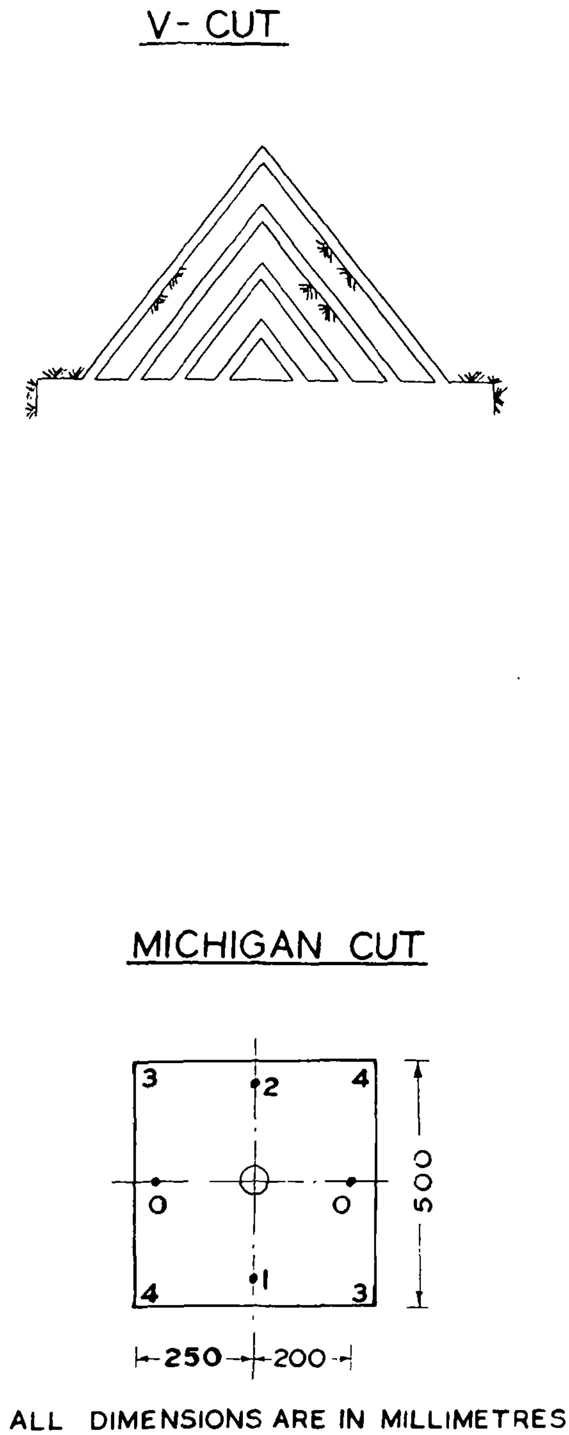

V- CUT
MICHIGAN CUT
3
2
4
O
O
4
1
3
500
250
200
ALL DIMENSIONS ARE IN MILLIMETRES

FIG. 5.7

OPERATING CYCLE FOR ONE ROUND

IMPORTANCE OF BRIDGE ACTION TIME

$tb < tv$ — EXCESSIVE OVER BREAK

$tv < tb < tm$ — OVER BREAK INEVITABLE

$tm < tb \leq ts$ — NO OVER BREAKS

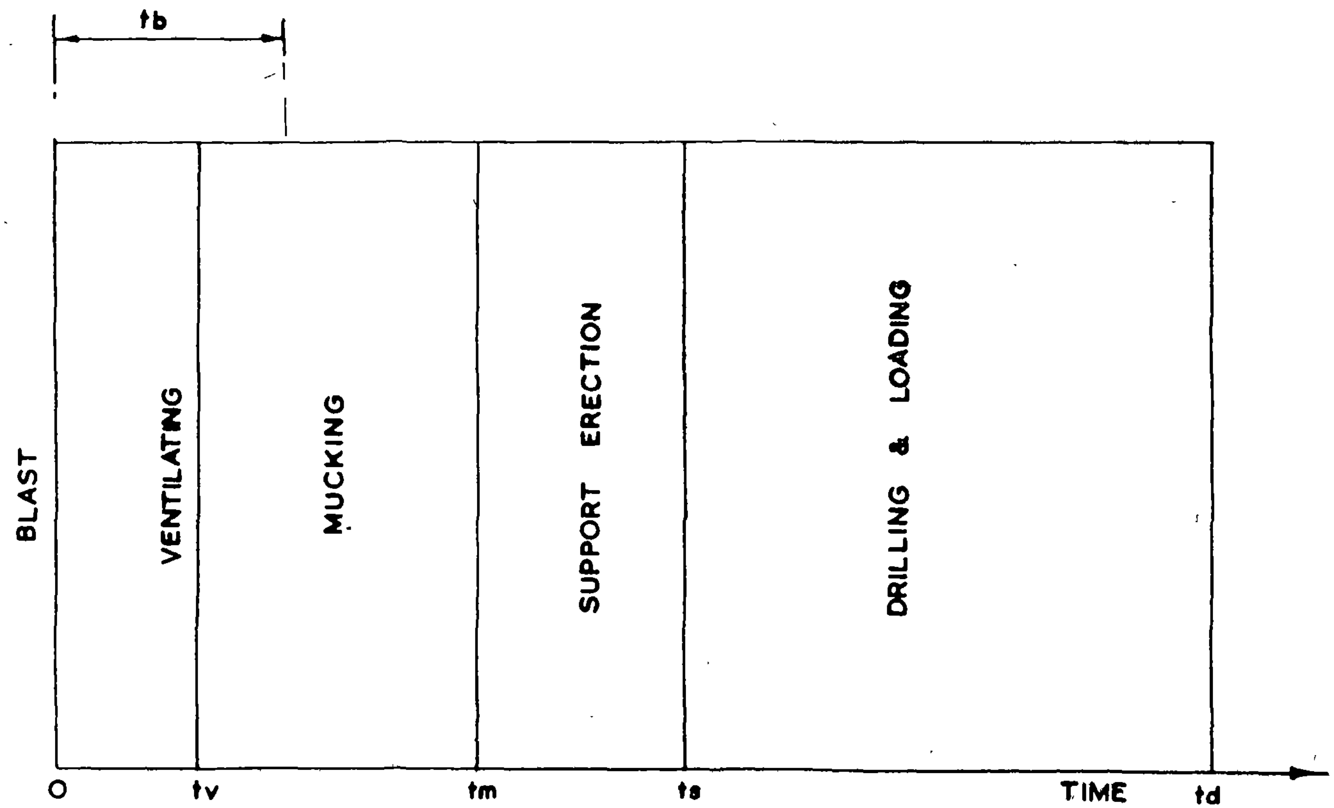


FIG. 5.8

RESTRAINT ON DEVELOPMENT OF ROCK LOAD DUE TO PROVISION OF SUPPORTS

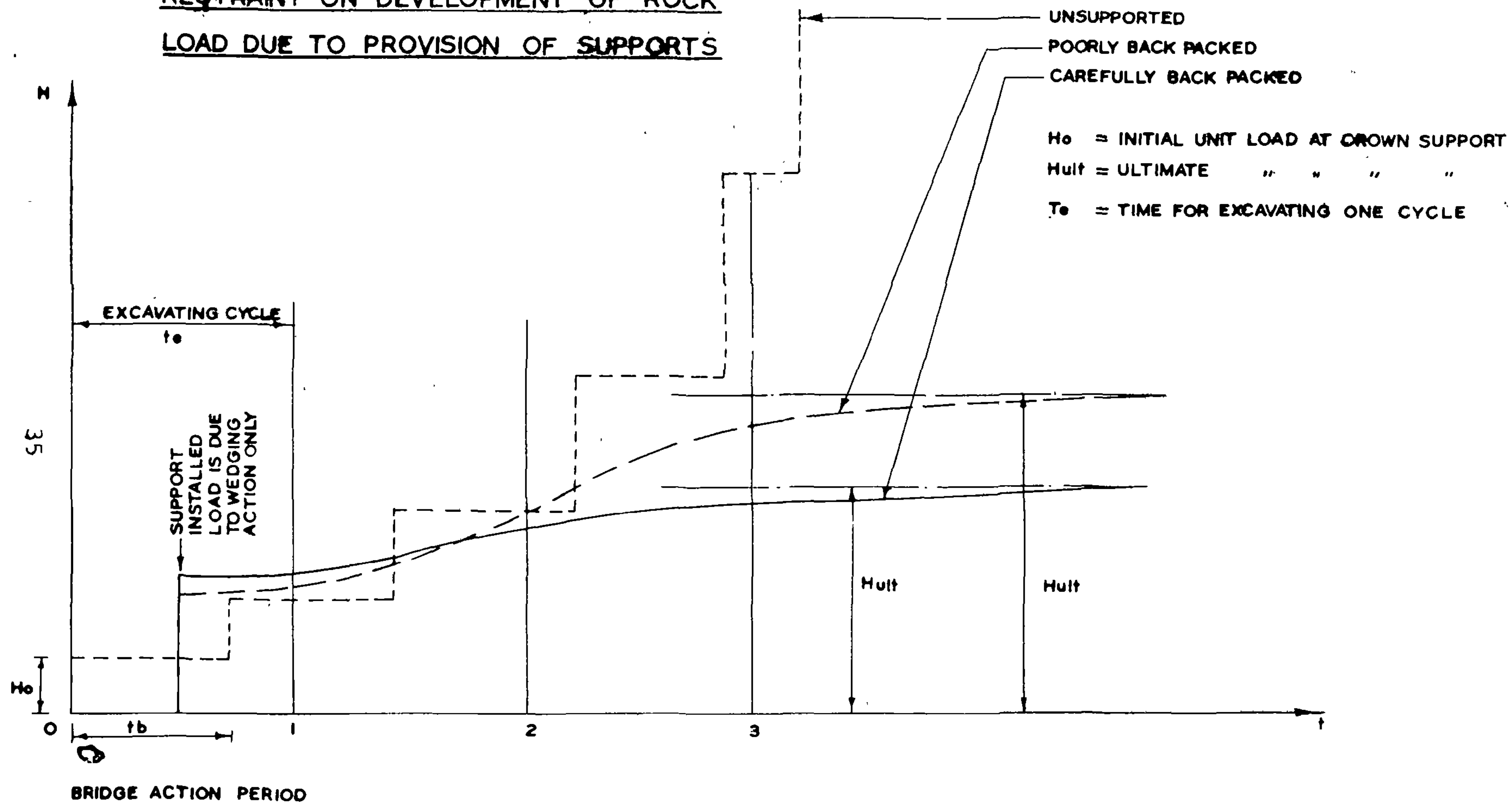


FIG. 5.9

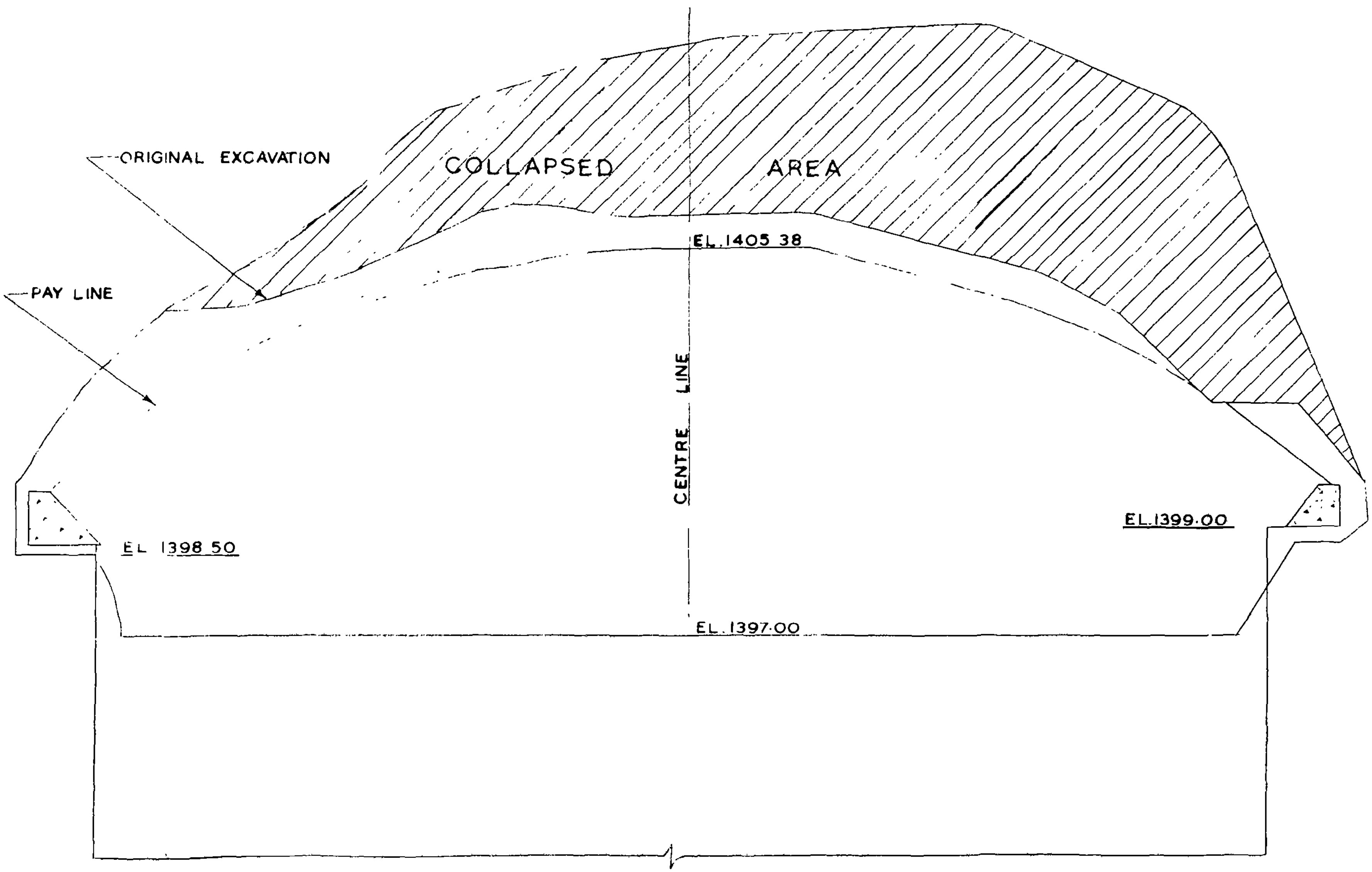
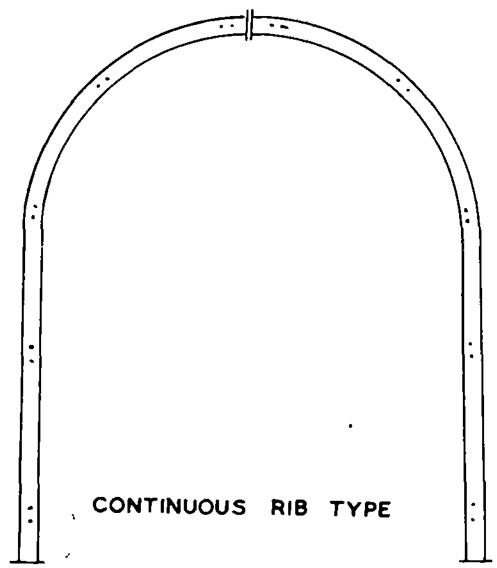


FIG. 5·10 CHUKHA POWER HOUSE CAVITY COLLAPSE AND DOMEFORMATON

FIG. 5·11

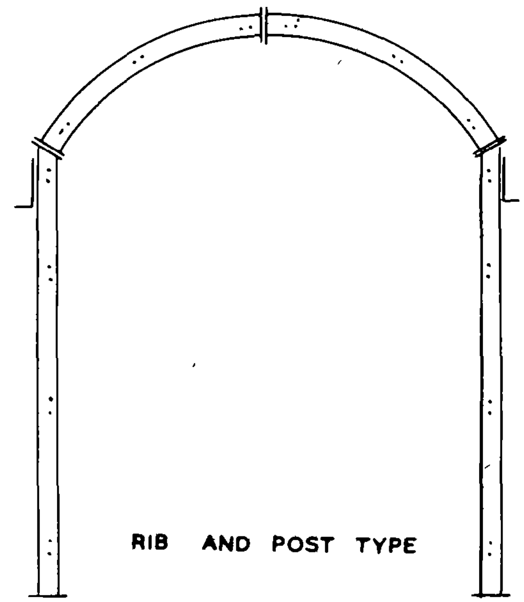


CONTINUOUS RIB TYPE

USUALLY MADE IN TWO PIECES FOR MAXIMUM SPEED OF ERECTION, LOWEST FIRST COST, AND LOWEST ERECTION COST, SOMETIMES USED IN THREE OR FOUR PIECES TO MEET SPECIAL CONDITIONS,

USED WITH THE FOLLOWING METHODS OF ATTACK:-

FULL FACE
SIDE DRIFT
MULTIPLE DRIFT

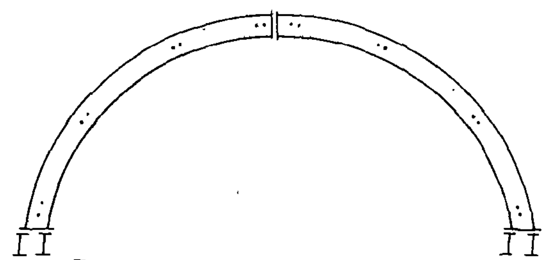


RIB AND POST TYPE

USED WITH THE FOLLOWING METHODS OF ATTACK:-

FULL FACE	IN TUNNELS WHOSE ROOF ARCH MAKES AN ANGLE WITH THE SIDE WALL.
MULTIPLE DRIFT SIDE DRIFT	IN TUNNELS AT SUCH LARGE SIZE THAT 2 PIECE CONTINUOS RIBS CANNOT BE SHIPPED AND/OR HANDLED.
HEADING AND BENCH TOP HEADING	FOR SUPPORT IN THE DRIFTS (WITH TRUSS PANELS) FOR EARLY SUPPORT TO ROOF

FIG. 5·12



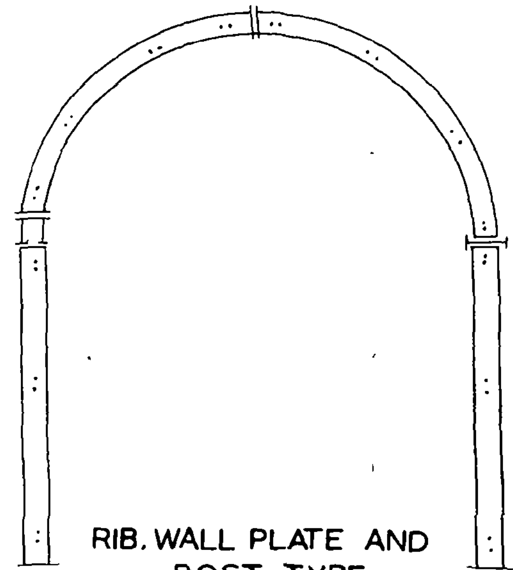
RIB AND WALL PLATE TYPE

RIB USUALLY MADE IN TWO PIECES FOR MAXIMUM SPEED OF ERECTION, LOWEST FIRST COST AND LOWEST ERECTION COST. SOMETIMES USED IN THREE OR MORE PIECES TO MEET SPECIAL CONDITIONS.

USED WITH THE FOLLOWING METHOD OF ATTACK :–

HEADING AND BENCH
TOP HEADING
FULL FACE

THIS TYPE IS ESPECIALLY APPLICABLE TO CIRCULAR AND HIGH SIDED TUNNEL SECTIONS WHERE ONLY A LIGHT ROOF SUPPORT IS NEEDED.



RIB, WALL PLATE AND POST TYPE

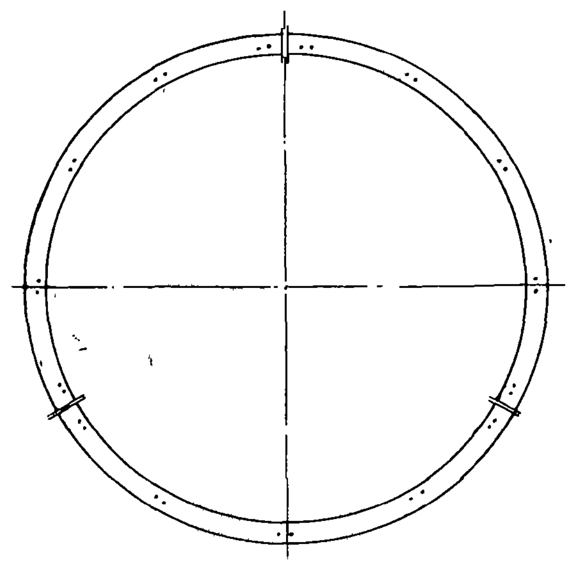
USED WITH THE FOLLOWING METHODS OF ATTACK :–

HEADING AND BENCH TOP HEADING	FOR QUICK SUPPORT TO ROOF
SIDE DRIFT	IN LARGE TUNNEL WHITH BAD ROCK CONDITIONS REQIRING QUICK SUPPORT.
FULL FACE	FOR FAVORABLE ROCK WHERE SUPPORT IS NOT NEEDED RIGHT TO THE FACE.
	FOR TUNNELS WHOSE ROOF MAKES AN ANGLE W ITH THE SIDE WALL.

A
SHOWN WITH DOUBLE BEAM WALL PLATE

B
SHOWN WITH FLAT WALL PLATE.

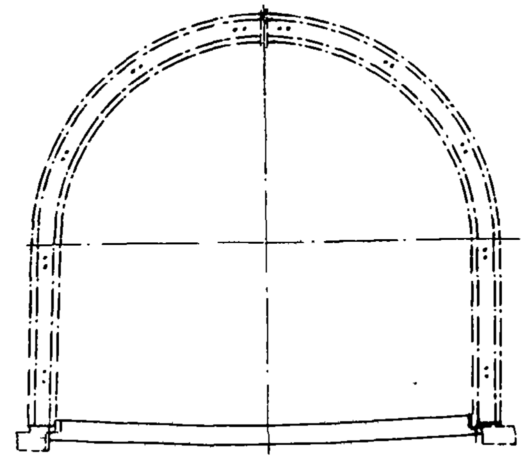
FIG. 5·13.



FULL CIRCLE RIB TYPE

USED WITH THE FOLLOWING METHODS OF ATTACK :—

FULL FACE	IN TUNNEL IN SQUEEZING, SWELLING AND CRUSHED ROCK, OR ANY ROCK THAT IMPOSED CONSID-ERABLE SIDE PRESSURE. ALSO WHERE BOTTOM CONDITION MAKE IT IMPOSSIBLE TO CARRY ROOF LOADS ON FOOT BLOCKS, IN EARTH TUNNEL. CONDITIONS SOMETIMES ENCOUNTERED IN ROCK TUNNELS.
HEADING AND BENCH	UNDER EARTH TUNNEL CONDITIONS WITH JOINT AT SPRING LINE.



INVERT STRUT

USED WHERE MILD SIDE PRESSURE ARE ENCOUNTERED.

ALSO USED TO PREVENT BOTTOM FROM HEAVING.

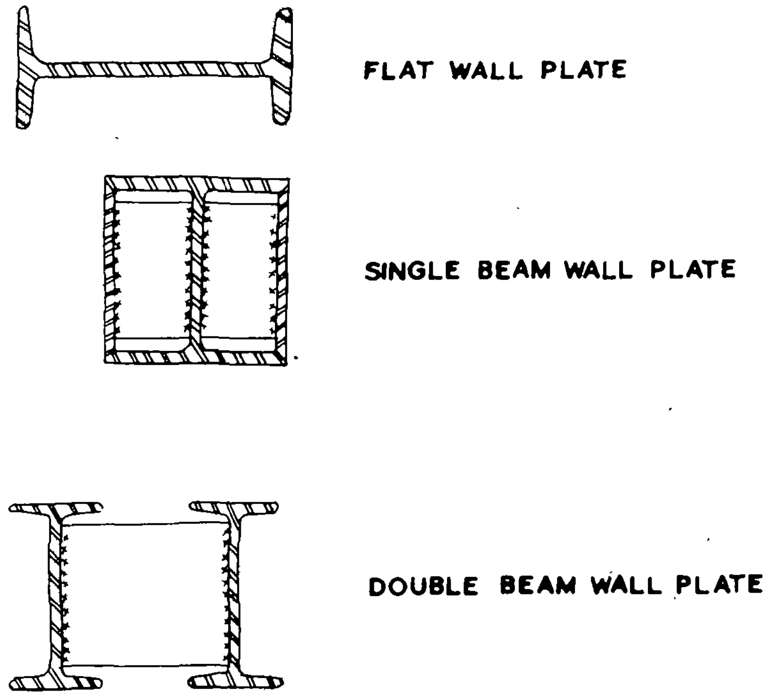
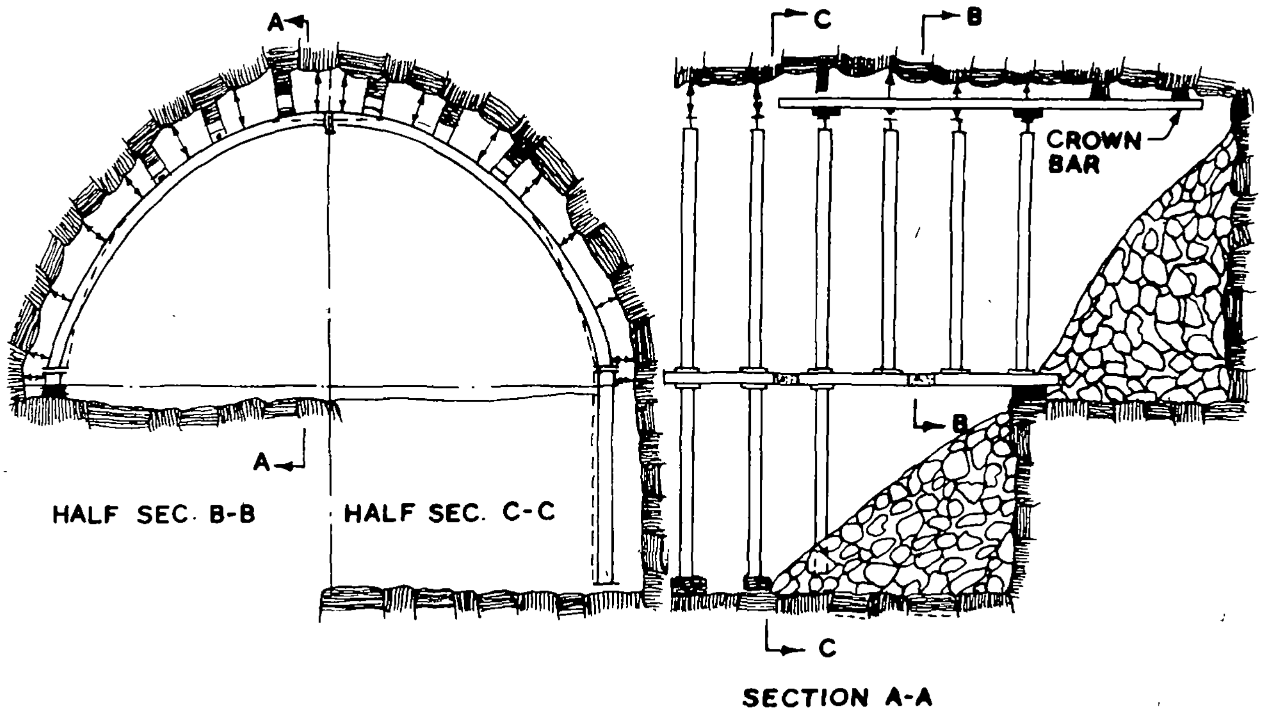


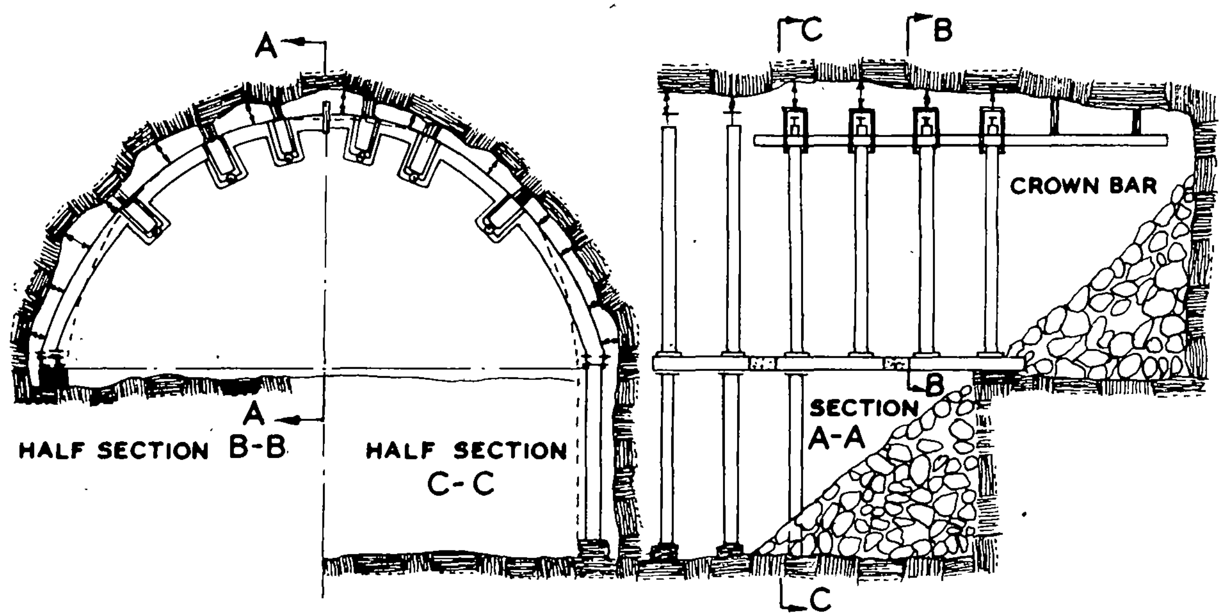
FIG. 5.14 TYPES OF WALL PLATES



INDICATES BLOCKING BETWEEN ROCK AND RIB.

INDICATES BLOCKING BETWEEN CROWN BAR & ROCK OR RIB.

FIG. 5.15 CROWN BARS MOUNTED ON RIBS SUPPORT THE NEW ROOF BY CANTILEVER ACTION.



NOTE :- INDICATE BLOCKING BETWEEN ROCK AND RIB
INDICATE BLOCKING BETWEEN CROWN BAR AND ROCK OR RIB

FIG. 5·16 CROWN BARS, HUNG FROM THE RIBS, SUPPORT THE NEW ROOF BY CANTILEVER ACTION

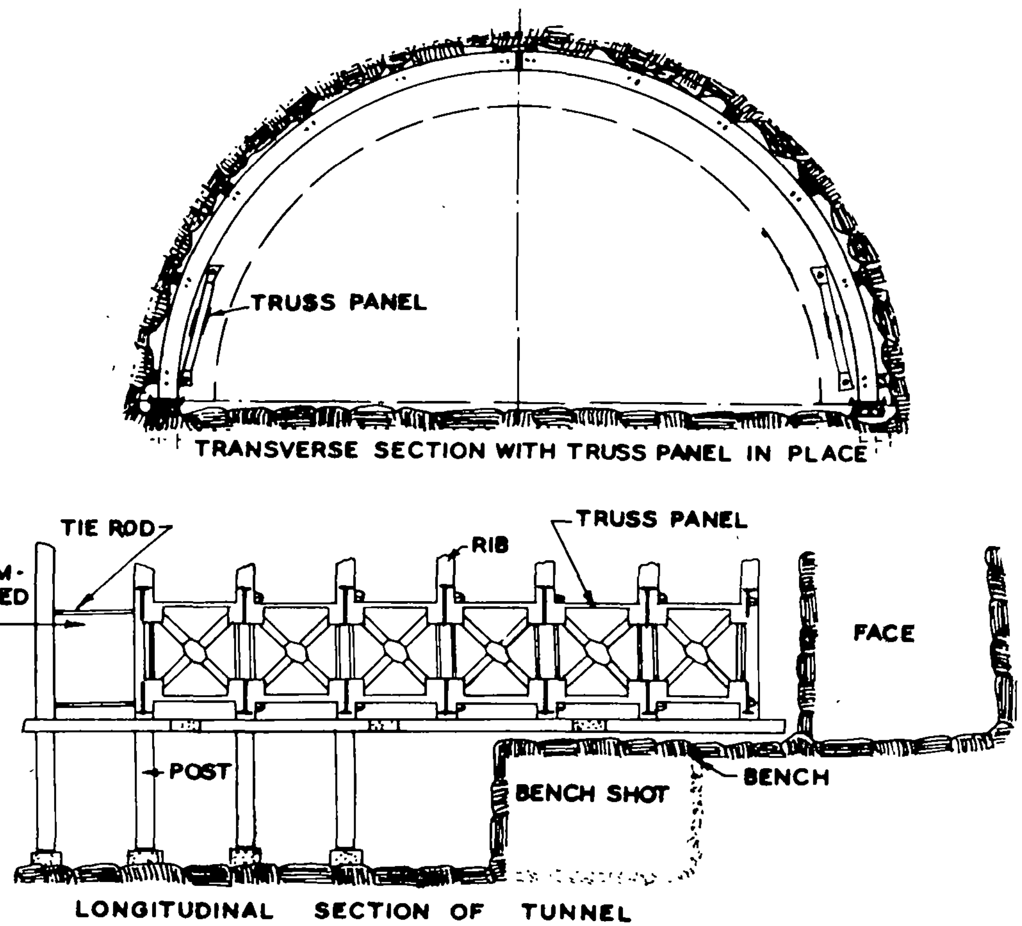
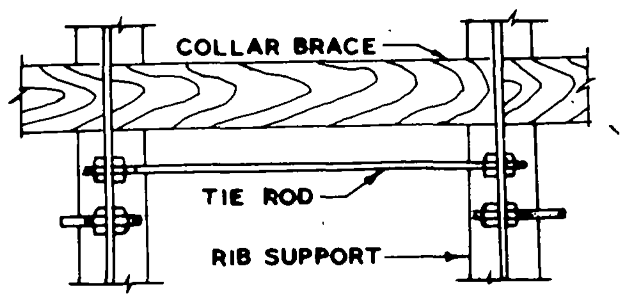
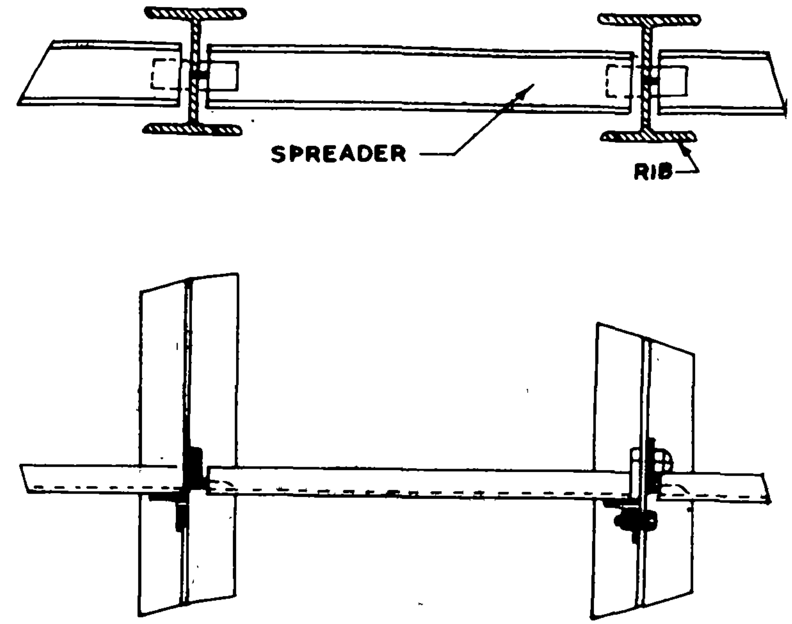


FIG. 5.17 TRUSS PANEL USED TO SUPPORT ROOF RIBS WHILE BENCH IS EXCAVATED

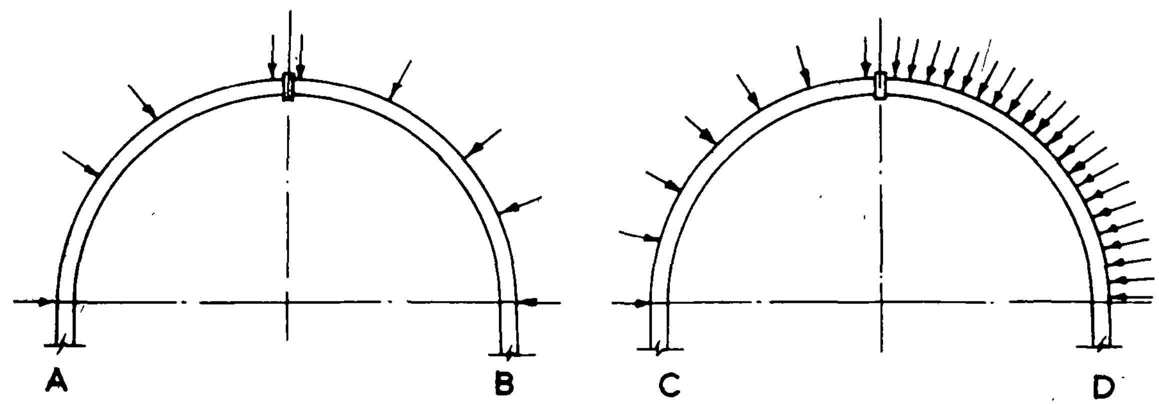


COLLAR BRACE AND TIE RODS



SPREADER DETAILS

FIG. 5.18 BRACING AND SPREADER DETAILS



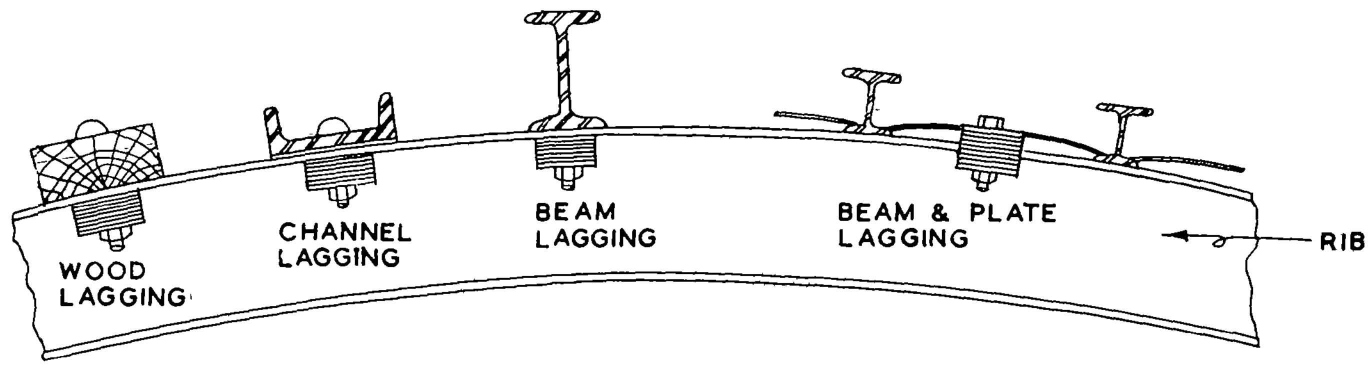
70" MAX. BLOCKING POINT SPACING RIB STRESS=31275 P.S.I

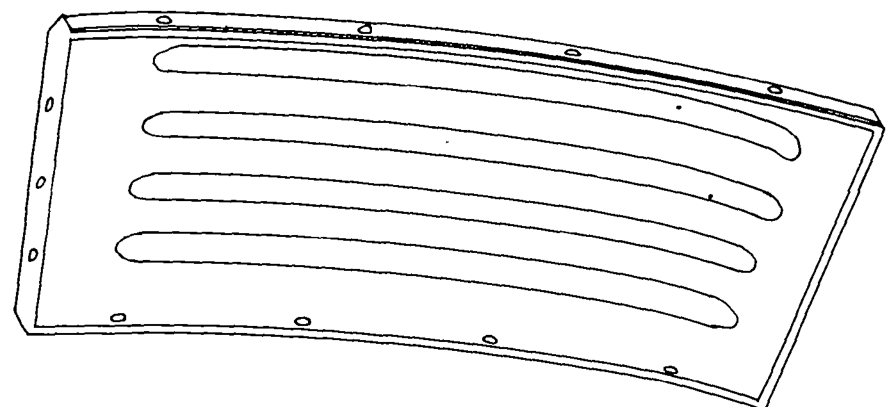
50" MAX, BLOCKING POINT SPACING RIB STRESS = 23000 P.S.I

30" MAX, BLOCKING POINT SPACING RIB STRESS = 17720 P.S.I

ZERO BLOCKING POINT SPACING (TUNNEL BACK PACKED) RIB STRESS = 14175 P.S.I

FIG. 5.19 EFFECT ON RIB STRESS OF CHANGES IN BLOCKING POINT SPACING





DETAILS OF STEEL LINER PLATE
USED AS LAGGING IN EARTH TUNNELS

FIG. 5.20 TYPES OF LAGGING

CHAPTER 6

TUNNEL LINING

6.0 General

Lining in tunnels is a very important component and makes up for 30 to 40 percent of the total cost of tunnel. Tunnels forming part of water conductor system have to be invariably lined with cement concrete—plain or reinforced; or steel lined. However, in cases where a tunnel is meant for operation for short periods and where it has to be abandoned after it has served the purpose e.g., diversion tunnels the lining could be avoided.

Concrete lining is normally provided for power tunnels which connect directly to the pressure pipe lines and thence to the turbines in power houses. The concrete lining is required to ensure that no sand or rock particles are carried from the tunnel system into the machine.

Where rock cover is inadequate to prevent leakage and where high velocity erosion or cavitation can occur as in the case of silt flushing tunnels—a steel lining is required to be provided. The main function of the steel lining is to protect the concrete and to stop leakage of water from the tunnel. In general, the steel lining should be strong enough to withstand the internal pressure which is not taken by the rock surrounding the lining and must be capable of taking the full external water pressure.

6.1 Concrete Lining

The function of concrete lining is one or more of the following:

(*i*) to reduce head losses in the system;
(*ii*) to protect steel ribs from deteriorating;
(*iii*) to prevent leakage of water;
(*iv*) to protect the turbines by preventing loose rock particles falling into the water and being carried to the turbines;
(*v*) to take that part of the internal pressure which is not taken by the rock.

6.1.1 *Effective Cover*

The effective cover is often somewhat smaller than the actual cover over the tunnel soffit. In steeply sloping country, the horizontal cover generally governs while in flat country the effective cover is equal to the vertical distance to the ground surface. In intermediate type of country, the diagonal distance to the surface may determine the effective cover. Figure 6.1 explains the relationship between the actual and the estimated effective covers. This figure is based on the assumption that horizontal cover is half as effective as vertical cover. To obtain the effective cover, a profile of the ground surface (perpendicular to the contour lines) should be drawn, and the curve shown in the figure fitted in such a way that it just touches the ground surface. The vertical distance marked C_e then gives the effective cover.

6.1.2 *Structural Design of Concrete Lining*

The structural design of tunnel lining requires a thorough study of the geology of rock mass, the effective cover (as detailed above), in-situ modulus of elasticity, Poisson's ratio, state of stress, crushing strength and other mechanical properties of the rock mass. The presence or absence of water in the rock being tunnelled through has a lot of influence on the design of lining. Wherever water bearing strata is met with, efforts should be made to release the water by provision of drainage holes provided at suitable locations.

Ordinarily, tunnels should be provided with plain cement concrete lining unless the conditions warrant for reinforcement to be provided. Free flow tunnels should be invariably provided with P.C.C. lining since no load—external or internal is supposed to be acting on the lining. For pressure tunnels reinforcements may be needed should the depth of effective cover be less than the internal pressure head. However, in competent rock where there is no danger of blowout or landslides in adjacent areas due to saturation surcharge, the reinforcements need not be provided even when cover is between 1.0 H and 0.7 H.

6.1.2.1 CIRCULAR LINING

A circular lining is generally adopted where the rock cover is low and as such sections require reinforcements, to minimise leakage and to ensure structural stability. Circular lining has to be invariably adopted where the effective cover in good rock is lesss than the internal pressure head and in poor rock where the effective cover is less than 1.25 times the internal head.

6.1.2.2 DESIGN LOADINGS

(A) *Water Load*

The magnitude of design loads caused by water pressure on the tunnel lining either externally or internally follows closely the operating conditions for which the system has been designed. The operating conditions are classified by Parmakian as "Normal", "Emergency" and "Emergency not to be considered as basis for design". He accepts the principle of using stresses higher than normal working stresses for loadings, which while possible, are either very infrequent or most unlikely to occur during the life span of the project.

The design loadings applicable to tunnel lining are classified similarly except that the load corresponding to "Emergency Operating Conditions" are designated as "Extreme Design Loads". As definition of third and last operation suggests, it should not be considered for design purposes but should be checked so that complete failure does not occur under this condition. The design loadings are as follows:

(a) EXTERNAL DESIGN LOADS

(i) *Normal Design Loading Condition:* The maximum loading obtained from either maximum steady or steady state condition with loading equal to normal maximum ground water pressure and no internal pressure (applicable only if no ground water drains are provided); or, maximum difference in levels between the hydraulic gradient in the tunnel under steady state or static conditions and the maximum downsurge under normal transient operation.

(ii) *Extreme Design Loading Condition:* Loading equal to maximum difference in levels between the hydraulic gradient in the tunnel under steady state and the maximum down surge under extreme transient operation.

(b) INTERNAL DESIGN LOADS

(i) *Normal Design Loading Condition:* This condition is to be taken as the loading requiring maximum reinforcement in accordance with design criteria shown in Figure 6.2 for either of the following to cases.

Maximum static condition with maximum water level in the reservoir and no external pressure, for condition of leakage being important, or loading equal to the difference in levels between the maximum upsurge occurring under normal transient operation and the tunnel invert, for condition of leakage being not important, as the loading is of a very short duration.

(ii) *Extreme Design Loading Condition:* Loading equal to the difference between the highest level of hydraulic gradient in the tunnel under emergency transient operation and invert of the tunnel.

(B) *External Rock Pressure*

Except in the immediate vicinity of portals, no load shall be taken due to external rock pressure. Squeezing ground is to be considered as a special condition when encountered during excavation and is not covered by the above criteria.

(C) *Grout Pressure*

Concrete lining should also be checked for an external pressure corresponding to 50% maximum grout pressure specified.

6.1.2.3 DESIGN CHART

The graphs in the design chart shown in in Figure 6.2, are based upon the experience obtained in tunnels associated with the Snowy Mountains Hydro-Electric Schemes. The design chart aiso takes into account the effect of available rock cover on the amount of reinforcement required in the tunnel linings.

The graphs differentiate between competent and incompetent rocks. Competent rock is defined as rock which deflects under a load equivalent to maximum internal pressure at static condition only to the extent that the stress in lining reinforcement consisting of one row of 45 mm dia bars @ 150 mm centres does not exceed 1120 kg/cm^2. Rock, which would deflect to the extent that the stress in the reinforcing hoops exceeds 1120 kg/cm^2 is defined as incompetent. Figure 6.3 shows the relationship between the internal pressure and moduli of deformation of rock for various tunnel diameters required to keep the stress in the reinforcing hoops (equivalent to 1 row of 45 mm dia bars @ 150 mm centre, i.e., 102 cm^2 per metre of the tunnel) equal to 1120 kg/cm^2. This graph is based on the assumption that the only gap between the lining and the rock is due to temperature change caused by cold water flowing through the tunnel. The gap due to shrinkage, shattering of rock etc. has been considered as eliminated by the process of grouting.

If the estimated modulus of deformation is equal to or greater than required by the graph shown in Figure 6.3, the rock is considered to be competent and the full lines on the design chart (Figure. 6.2) should be used. If, on the other hand, the estimated modulus of deformation is smaller than that required by the graph (Figure. 6.3), the dotted lines on design chart (Figure. 6.2) should be used. To minimise construction difficulties, it is considered that even in poorer quality of rock the amount of lining reinforcement should not exceed a maximum of 204 cm^2 per metre of tunnel, i.e., 2 rows of 45 mm dia bars @ 150 mm centres. If this quantity of steel reinforcement is still insufficient to keep the stress at or below 1120 kg/cm^2, a steel lining should be provided.

The design chart (6.2) is divided into three graphs. One is for static conditions, second is for normal transient conditions and third is for extreme loading conditions. The graphs have been set in such a way that for a small cover of upto 3D the maximum stresses in the reinforcement do not exceed 850 kg/cm^2 for no cover and 1120 kg/cm^2 for cover equal to 3 times the tunnel dia regardless of the loading conditions. For static conditions, leakage is considered for major importance upto that point along the tunnel line where effective cover is equal to the maximum static head and reinforcement in the lining is provided to this point. For incompetent rock the reinforcement is extended to a point where the cover is 1.25 times the maximum static head. The stresses have been choosen so that at the point where no blow out can occur, i.e., where the depth of cover equals one half of the static head, the design stress is approximately at yield point of the full load is taken by the lining. The actual stresses in the reinforcement, however, do not exceed 1120 kg/cm^2. From this point on, with the increase in cover, increased design stresses are allowed so that for a cover 0.7 times the static head, a design stress of 4200 kg/cm² would be reached if the entire load was taken by the lining. From there on, the amount of reinforcement is gradually reduced to the nominal reinforcement with an area $A_s = 0.005\ A_c$

where A_s = area of steel reinforcement
A_c = area of concrete upto the pay-line.

For normal transient conditions, leakage is not considered important as the transient condition operates only over a relatively short time and for this case an increase of 50 percent is allowed in the design stress.

For extreme loading condition, the design stresses are increased by further 50 percent as compared with design stresses for normal transient condition.

6.1.3 *Design of Concrete Lining Using I.S. Criteria*

Indian Standards Institution has recommended the use of certain definite formulae and equations for design of concrete lining—both for external as well as internal pressures. The design for external loads may be done by considering the lining as independent structural member whereas the design for internal water pressure may be done by considering it as a part of composite thick cylinder consisting of peripheral concrete and the surrounding rock mass subjected to specific boundary condions.

6.1.3.1 DESIGN FOR EXTERNAL LOADS

A tunnel lining may be subjected to external loads due to rock, external water pressure, grout pressure, self weight, and weight of water contained in the tunnel as shown in Figure 6.4. Following formulae have been developed to obtain the values of bending moments, normal thrust, radial shear, horizontal and vertical deflections based on the assumption that it deflects under the active external loads and its deflection is restricted by the passive resistance developed in the surrounding rock mass. Following notations shall apply for these formulae:

E—Young's modulus of elasticity of lining material;
I—Moment of inertia of the section;
K—Intensity of lateral triangular load at horizontal diameter;
P—Total rock load on mean diameter;
r—Internal radius of tunnel;
R—Mean radius of tunnel lining;
t—Thickness of lining;
W—Unit weight of water;
W_c—Unit weight of concrete; and
ϕ—angle that the section makes with the vertical diameter at the centre measured from the invert,

Following sign conventions shall apply for this formulae

(*i*) Positive moment indicates tension in inside face and compression on outside face;
(*ii*) Positive thrust means compression on the section;
(*iii*) Positive shear means that considering left half of the ring the sum of all the forces on the left of the section acts outwards when viewed from inside;
(*iv*) Positive horizontal deflection means outward deflection with reference to centre of conduit; and
(*v*) Positive vertical deflection means downward deflection.

Values of Bending Moments

ϕ	Uniform Vertical Load	Conduit Weight	Contained Water	Lateral Pressure
0°	+0.125 PR	$+0.4406\ W_ctR^2$	$+0.2203\ Wr^2R$	$-0.1434\ KR^3$
45°	Zero	$-0.0334\ W_ctR^2$	$-0.0167\ Wr^2R$	$-0.0084KR^2$
90°	−0.125 PR	$-0.3927\ W_ctR^2$	$-0.1963\ Wr^2R$	$+0.1653\ KR^2$
135°	Zero	$+0.0334\ W_ctR^2$	$+0.0167\ Wr^2R$	$-0.0187\ KR^2$
180°	+0.125 PR	$+0.3448\ W_ctR^2$	$+0.1724\ Wr^2R$	$-0.1295\ KR^3$

Values of Normal Thrust

ϕ	Uniform Vertical Load	Conduit Weight	Contained Water	Lateral Pressure
0°	Zero	$+0.1667\ W_ctR$	$-1.4166\ Wr^2$	+0.4754 KR
45°	+0.250 P	$+1.1332\ W_ctR$	$-0.7869\ Wr^2$	+0.3058 KR
90°	+0.500 P	$+1.5708\ W_ctR$	$-0.2146\ Wr^2$	Zero
135°	+0.250 P	$+0.4376\ W_ctR$	$-0.4277\ Wr^2$	+0.2674 KR
180°	Zero	$-0.1667\ W_ctR$	$-0.5834\ Wr^2$	+0.3782 KR

Values of Radial Shear

ϕ	Uniform Vertical Load	Conduit Weight	Contained Water	Lateral Pressure
0°	Zero	Zero	Zero	Zero
45°	−0.250 P	$-0.8976\ W_ctR$	$-0.4488\ Wr^2$	+0.3058 KR
90°	Zero	$+0.1667\ W_ctR$	$+0.0833\ Wr^2$	−0.0246 KR
135°	+0.250 P	$+0.6732\ W_ctR$	$+0.3366\ Wr^2$	−0.2674 KR
180°	Zero	Zero	Zero	Zero

Values of Horizontal Deflection

ϕ	Uniform Vertical Load	Conduit Weight	Contained Water	Lateral Pressure
0°	Zero	Zero	Zero	Zero
45°	$+0.01473\,\frac{PR^3}{EI}$	$+0.05040\,\frac{W_ctR^4}{EI}$	$+0.02520\,\frac{Wr^2R^3}{EI}$	$-0.01750\frac{KR^4}{EI}$
90°	$+0.04167\,\frac{PR^3}{EI}$	$+1.13090\,\frac{W_ctR^4}{EI}$	$+0.06545\,\frac{Wr^2R^3}{EI}$	$-0.05055\frac{KR^4}{EI}$
135°	$+0.01473\,\frac{PR^3}{EI}$	$+0.04216\,\frac{W_ctR^4}{EI}$	$+0.02108\,\frac{Wr^2R^3}{EI}$	$-0.01624\frac{KR^4}{EI}$
180°	Zero	Zero	Zero	Zero

Values of Vertical Deflection

ϕ	Uniform Vertical Load	Conduit Weight	Contained Water	Lateral Pressure
0°	Zero	Zero	Zero	Zero
45°	$+0.02694\,\frac{PR^3}{EI}$	$+0.09279\,\frac{W_ctR^4}{EI}$	$+0.04640\,\frac{Wr^2R^3}{EI}$	$-0.03176\frac{KR^4}{EI}$
90°	$+0.04167\,\frac{PR^3}{EI}$	$+0.13917\,\frac{W_ctR^4}{EI}$	$+0.06958\,\frac{Wr^2R^3}{EI}$	$-0.04995\frac{KR^4}{EI}$
135°	$+0.05640\,\frac{PR^3}{EI}$	$+0.18535\,\frac{W_ctR^3}{EI}$	$+0.07268\,\frac{Wr^2R^3}{EI}$	$-0.06810\frac{KR^4}{EI}$
180°	$+0.0833\,\frac{PR^3}{EI}$	$+0.26180\,\frac{W_ctR^4}{EI}$	$+0.13090\,\frac{Wr^2R^3}{EI}$	$-0.09739\frac{KR^1}{EI}$

Using the above formulae, the tunnel lining for the Chukha Head Race Tunnel (Bhutan) has been checked. The calculations are appended in Annexure III. Strictly speaking, the above formulae are applicable for circular tunnels only but the same can be applied for hores-shoe and D-shaped tunnels also accepting some marginal error.

6.1.3.2 Design for Internal Pressure

The basic assumption in the design is that the lining shall be considered as a part of composite thick cylinder consisting of peripheral concrete and surrounding rock mass subjected to specified boundary conditions. To make the above assumption realistic, effective pressure grouting has to be done to fill up all the gaps and cracks in the surrounding rock mass.

Sometimes, if the rock is good and cracking of the lining does not involve much loss of water, the cracking of lining may be permitted to some extent. But if the surrounding rock is poor, reinforcement may be provided to reduce the tensile stress in concrete thereby distributing the cracks in the whole periphery in the form of hair cracks which are not harmful.

The basic equations given below are for the design of circular section alone. For non-circular sections, it is recommended that the stress pattern may be obtained by carrying out photo-elastic studies. The equations given below may also be used for non-circular sections but the results obtained can not be regarded as true representatives of the inherent stress conditions.

The following notations shall apply for the equations:

P = internal hydrostatic pressure;

t_1, t_2, t_3 = tangential stress in rock, concrete and steel respectively;

E_1, E_2, E_3 = modulus of elasticity of rock, concrete and steel respectively;

m_1, m_2 = Poisson's ratio of rock and concrete respectively;

U_1, U_2, U_3 = radial deformation in rock, concrete and steel respectively;

x = radius of the element;

B, C etc. = constants of integration;

A_s = areas of reinforcement for unit length of tunnel;

a = internal diameter of the tunnel; and

b = external diameter of the lining upto minimum excavation line.

Case 1: Plain cement concrete lining considering that it is not cracked.

(a) Basic Equations:

$$\sigma r = \frac{mE}{m^2-1}\left(B(m+1)-\frac{C}{x^2}(m-1)\right)$$

$$\sigma t = \frac{mE}{m^2-1}\left(B(m+1)+\frac{C}{x^2}(m-1)\right)$$

$$U = Bx+\frac{C}{x}$$

(b) Limit Conditions and Constants:

(1) When $x=\infty$, $\sigma ri = 0$
(2) When $x=b$, $\sigma ri = \sigma_{r2}$
(3) When $x=b$, $\sigma r_2 = -p$
(4) When $x=b$, $U_1 = U_2$

Case 2: Plain Cement Concrete lining considering that it is cracked.

(a) Basic equations for Rock:

$$\sigma r_1 = \frac{m_1E_1}{m_1^2-1}\left(B_1(m_1+1)-\frac{C_1}{x^2}(m_1-1)\right)$$

$$\sigma t_1 = \frac{m_1E_1}{m_1^2-1}\left(B_1(m_1+1)+\frac{C_1}{x_2}(m_1-1)\right)$$

(b) Basic equations for concrete:

$$\sigma r_2 = \frac{a(\sigma r_2)_{x=a}}{x}$$

$$\sigma t_2 = 0$$

(c) Limit Conditions:

(1) When $x=\infty$, $\sigma r_1 = 0$
(2) When $x=b$, $\sigma r_1 = \sigma r_2$
(3) When $x=a$, $\sigma r_2 = -p$

(d) Constants of integration are calculated as:

$$B_1 = 0$$

$$C_1 = \frac{a\cdot b\cdot p\cdot(m_1+1)}{m_1E_1}$$

$$(\sigma r_2)x = a = -p$$

Case 3: Reinforced Cement Concrete lining considering that it is not cracked.

(a) Basic equations:

$$\sigma r = \frac{mE}{m^2-1}\left(B(m+1)-\frac{C_1}{x^2}(m-1)\right)$$

$$\sigma t = \frac{mE}{m^2-1}\left(B(m+1)+\frac{C_1}{x^2}(m-1)\right)$$

$$U = Bx + O/x$$

$$\sigma t_3 = \frac{E_3}{a}\left(B_2a \dotplus \frac{C_2}{a}\right)$$

$$\sigma r_3 = \frac{E_sA_s}{a^2}\left(B_2a+\frac{O_2}{a}\right)$$

(b) Limit Conditions and Constants:

(1) At $x=\infty$, $\sigma r_1 = 0$
(2) At $x=b$, $\sigma r_1 = \sigma r_2$
(3) At $x=a$, $\sigma r_2 - \sigma r_3 = -p$
(4) At $x=b$, $U_1 = U_2$

(c) Constants are given by:

$$C_1 = B_2b^2 + O_2$$

$$C_2 = \left(\frac{E_2m_2'm_1+1)}{E_1m_1(m_2+1)}\right)B_2 - \left(\frac{E_2m_2(m_1+1)^2}{E_1m_1(m_2-1)}\right)C_2$$

$$-p = B_2\left(\frac{E_2m_2}{m_2-1}-\frac{E_3A_s}{a}\right)-\left(\frac{E_2m_2}{a^2(m_2-1)}+\frac{E_3A_s}{a^3}\right)C_2$$

Case 4: R.C.C. lining considering that is cracked and that because of Radial cracks it cannot take tangential tensile stress.

(a) Basic equations for rock:

$$\sigma t_1 = \frac{E_1m_1C_1}{(m_1+1)^2\cdot x^2}$$

$$\sigma r_1 = -\sigma t_1$$

$$U_1 = \frac{C_1}{x}$$

(b) Basic equations for concrete:

$$\sigma t_1 = 0$$

$$\sigma r_2 = \frac{a(\sigma r_2)_{x=a}}{x}$$

$$U_2 = \frac{a(\sigma r_2)_{x=a}}{E_2}\log\frac{b}{a}$$

(c) Basic equations for steel:

$$\sigma t_3 = \frac{a\sigma r_3}{A_s}$$

$$\sigma r_3 = \frac{E_3A_s}{a^2}\left(aB_2+\frac{C_2}{a}\right)$$

$$U_3 = \frac{a^2\sigma_3}{E_sA_s}$$

(d) Constants of integration:

$$(\sigma r_2)_{x=a} = \frac{-pam_1E_1E_2}{am_1E_1E_2+m_1E_1E_3A_s\log\left(\frac{b}{a}\right)+(m_1+1)E_2E_3A_s}$$

$$C_1 = \frac{-ab(m_1+1)(\sigma r_2)_{x=a}}{m_1E_1}$$

$$\sigma r_3 = (\sigma r_2)_{x=a+p}$$

6.1.4 *General Requirements, Sequnece of Lining and Placing of Concrete*

The conditions for lining tunnels are very much different from those of other concrete works. Therefore, utmost importance should be given for the requirements of lining concrete on account of curvature, thin sections and difficulties in placement and compaction in restricted space.

On account of curvature, irregularities in rock profile etc., placing of concrete is generally done through a placer pump. The diameter of the placer pipe thus governs the minimum thickness of the lining. Enough space should be available at the tunnel crown to accommodate the placer pipe. Generally, the maximum size of stone is restricted to 40 mm and for this, the minimum size of placer pipe works out to 200 mm dia approximately. Since the concrete is required to be placed by pump, it has to be flowy to avoid segregation and to ensure proper filling. To ensure this, the minimum slump of concrete shall be 100 mm and the cement content shall be between 350 to 400 kg/m^3 to ensure adequate workability and to gain required strength. Sometimes, air entraining agents may have to be added upto about 4 percent for improving the workability of concrete.

Sequence of Concreting

Depending upen the shape and size of tunnel and the type of form work available, the sequence of concreting shall be one of the following:

(a) For horse-shoe and D-shaped tunnels, the kerb shall be concreted first to serve as a base for erection of form work. After the kerb is constructed and form work is erected, lining of sides and the arch is done simultaneously. Finally, after the removal of form work the invert concreting is done.

(b) For circular and narrow bottom tunnels, the invert concreting is done first and a regular base for the erection of form work for sides and arch is thus obtained making further work easier.

(c) In tunnels through weak strata or squeezing and/ or swelling rock, it is necessary to concrete the kerbs, sides and arch simultaneously.

The concrete should be thoroughly vibrated after placing by means of vibrating needle, flexible shaft immersion type vibrators. In addition, the concrete must also be vibrated by external form vibrators to avoid segregation of concrete.

The forms used for tunnel lining are generally of steel. The various types of forms used in tunnel lining are rib and plate type, rib and lagging type, travelling shutters telescopic or non-telescopic. The inside surface of the forms is generally coated with oil to prevent the concrete from sticking to it. The forms are removed after 16 to 24 hours and the tunnel surface is cured by keeping it constantly wet for at least 21 days.

6.2. Steel Lining

General

As briefly described earlier, steel lining is provided where the tunnel has to withstand high pressures. Steel lining consists of a steel plate of adequate thickness provided to the inner surface of the tunnel and serves the following purposes:

(a) to prevent water loss from the tunnel;
(b) to resist the bursting pressure of water carried by the tunnel;
(c) to provide protection from seepage of water from the surrounding mass like rock, concrete etc. and
(d) to provide a smooth surface for flow of water.

Generally, a circular section of the tunnel is found to be most suitable for steel lining. However, D-shaped and rectangular section have also been steel lined in exceptional circumstances as has been done for the Chukha silt flushing tunnel. I.S. Code has recommended a maximum velocity of 9 m/sec. through a steel lined tunnel. The material commonly used for steel lining is the same as that used for steel penstock—i.e., boiler and pressure vessel quality steel.

6.1.1 *Structural Design*

The structural design of steel lining includes the provision of adequate thickness of the steel plate to ensure safe handling and safety against internal as well as external pressure.

6.2.1.1 Thickness Required from Handling Considerations

The minimum thickness required to provide the rigidity required during fabrication and erection is given by the formula:

$$t=\frac{d+50}{400}$$

where = minimum handling thickness of plates in cms.
d = internal diameter of the tunnel in cms.

6.2.1.2 Design for Internal Pressure

The steel liner shall be designed to withstand the stress developed in closing the initial gap between the liner and the surrounding concrete plus the stress developed in the liner due to remainder of the pressure less the portion of the pressure carried by surrounding concrete and rock. The portion of the internal pressure transferred to the rock is given by the formula:

$$\lambda=\frac{\frac{r^2}{E}-\frac{y_o}{p}}{\frac{r^2}{E}-\frac{r}{E_r}(1+\mu)+\frac{r_c^2-r^2}{2r_cE_c}+\frac{r}{2r_cd}\left(\frac{d^2-r_c^2}{E_r}\right)}$$

or

$$\lambda=\frac{\sigma_t-\frac{Ey_o}{r}}{\sigma_t-p\left[\frac{E}{E_r}(1+\mu)+\frac{E}{2E_c},\frac{r}{r_c}(r_c^2-r^2)+\frac{Er^2}{2E_rr_cd}(d^2-r_c^2)\right]}$$

where λ = portion of internal pressure transferred to rock;
E = modulus of elasticity of steel;
E_r = modulus of elasticity of rock;
E_c = modulus of elasticity of concrete;
r = inside radius of steel liner;
r_c = outside radius of concrete lining;
d = radius to the end of radial fissures;
σ_t = allowable stress in the steel liner;
y_o = initial gap between steel liner and concrete;
p = internal pressure in the conduit; and
μ = Poisson's ratio of rock.

The ratio of rock participation for various types of rock with different modulus of elasticity can also be obtained from Figure 6.5.

The thickness of steel lining should also be determined for hoop stress and longitudinal stress. The thickness of steel liner is given by the formula.

$$t=\frac{pD}{2f_se}$$

where t = shell thickness in cm;
p = maximum internal pressure after rock participation in kg/cm^2

D = internal dia. of finished surface of tunnel in cm;
f_s = design stress in steel liner in kg/cm^2; and
e = efficiency of longitudinal joint.

6.2.1.3 Design for External Pressure

The steel liner shall also be designed to withstand the external pressure due to ground water and grout pressure. The ground water level should be taken upto ground level unless otherwise specified or established by geophysical explorations. On the assumption that there would be a radial gap between steel and surrounding concrete, the critical stress in the liner is given by the solution of the following two equations by Armstutuz:

$$\left(\frac{\sigma_t}{E^*}+\frac{y_o}{r}\right)\left[1+\frac{3K^2\sigma_t}{E^*}\right]^{3/2} = 1.68K\left(\frac{\sigma_y-\sigma_t}{E^*}\right)\left[1+\frac{K}{4}\cdot\frac{\sigma_y^\bullet-\sigma_t}{E^*}\right]$$

and

$$1-\frac{PK}{2\sigma_t}=0.175\,\frac{K}{E^*}\,(\sigma_y^\bullet-\sigma_t)$$

where

$$E^*=\frac{E}{1-\mu^2},\quad \sigma_y^\bullet=\frac{\sigma_y}{1-\mu-\mu^2}\text{ and } K=\frac{2r}{t}$$

where σ_t = allowable stress in material;
σ_y = yield stress of material;
E = modulus of elasticity of the material;
y_o = initial gap between the liner and concrete;
t = thickness of the pipe in cms; and
μ = Poisson's ratio

For different values of y_o/r and D/t a family of curves is shown in Figure 6.6 which gives critical external pressure for a material with yield stress of 32000 psi.

A value of 3×10^{-4} for initial gap is recommended and Figure 6.7 gives a family of curves which gives critical external pressures for a gap of $y_o/r=3\times10^{-4}$ for various types of steel generally used for steel liners.

6.2.1.3.1 *Anchors for Resisting External Pressure:* The steel liner is anchored to the rock around the opening by welding short angle irons (30 cm to 50 cm long) all along the circumference and embedded in concrete. The function of anchors is to stitch the steel liner into concrete so that there is no shearing off, should the steel liner try to slip along the concrete.

Spacing of the anchors along the periphery of the circular steel liner is given by Backte's formula

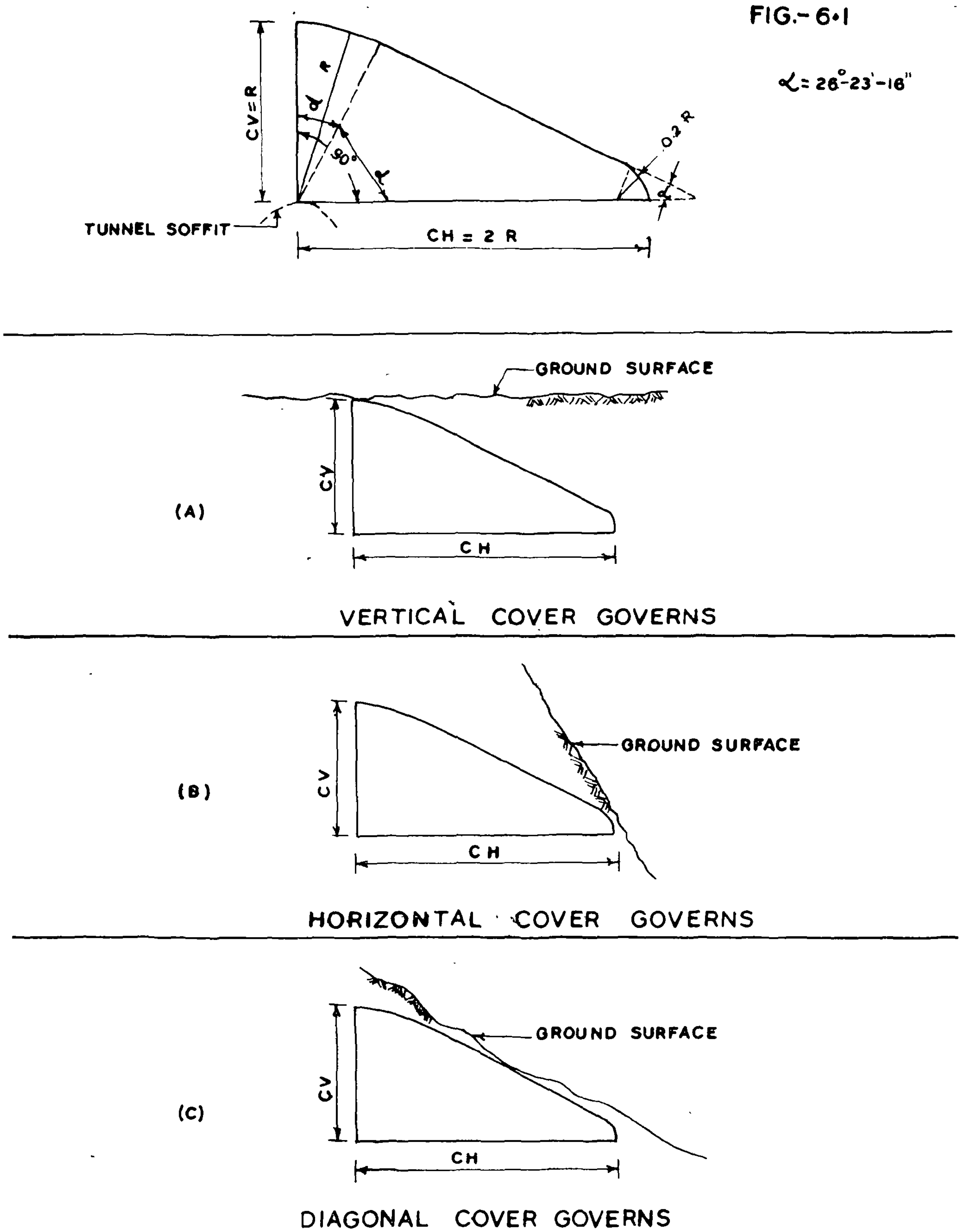
$$L=2aR$$

$$a=\frac{(Fe_1)^{3/2}\cdot E\cdot t^4}{P_c^{5/2}\cdot R^4}$$

where L = spacing of the anchors in cms;
Fe_1 = stress at lower yield point of steel in kg/cm^2;
E = modulus of elasticity of steel in kg/cm^2;
t = thickness of the shell in cm;
P_c = critical external pressure on the lining in kg/cm^2; and
R = internal finished radius of the tunnel in cm.

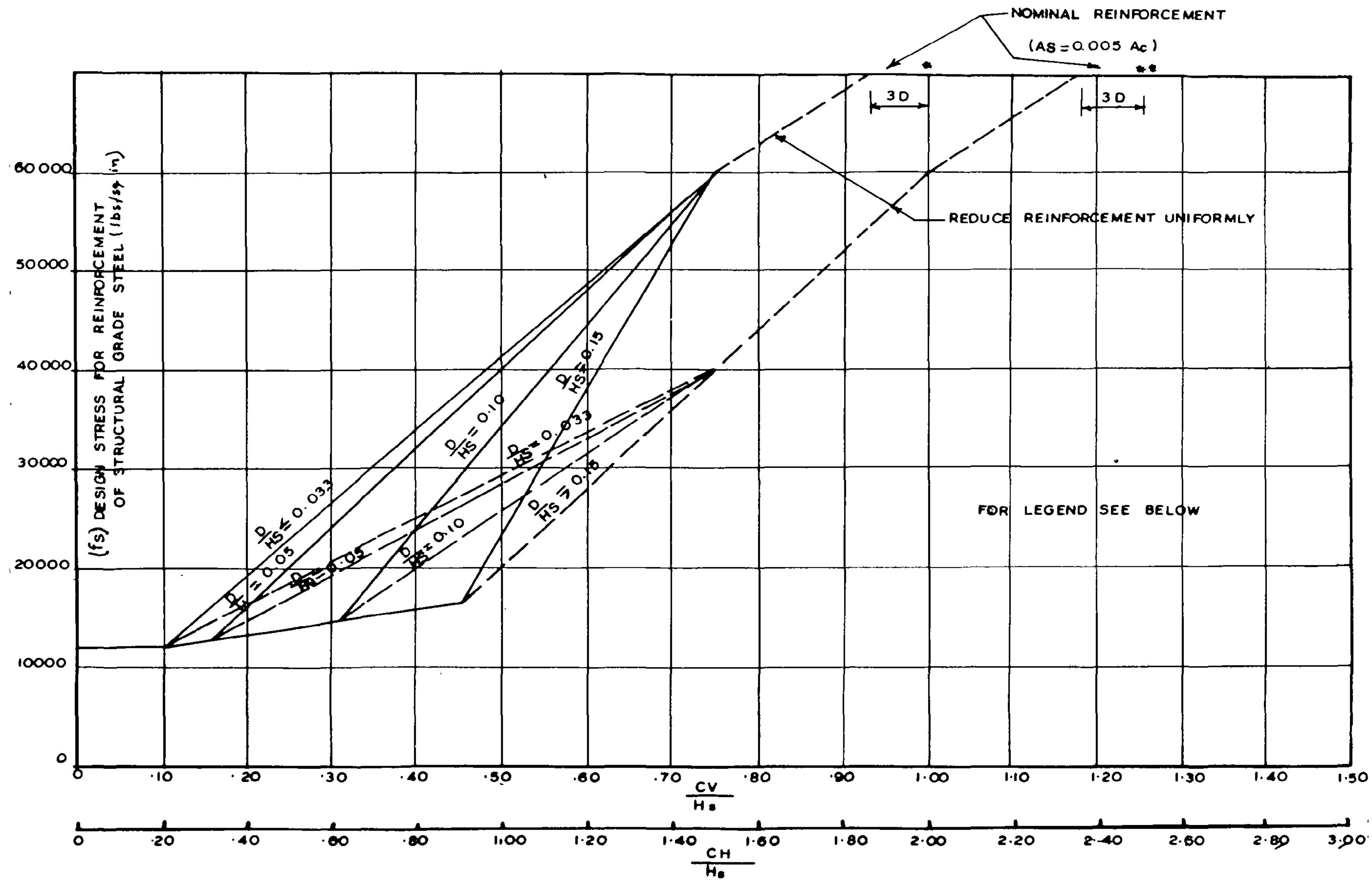
The spacing of the anchors in the longitudinal direction is kept the same as that along the circumference.

FIG.- 6·1



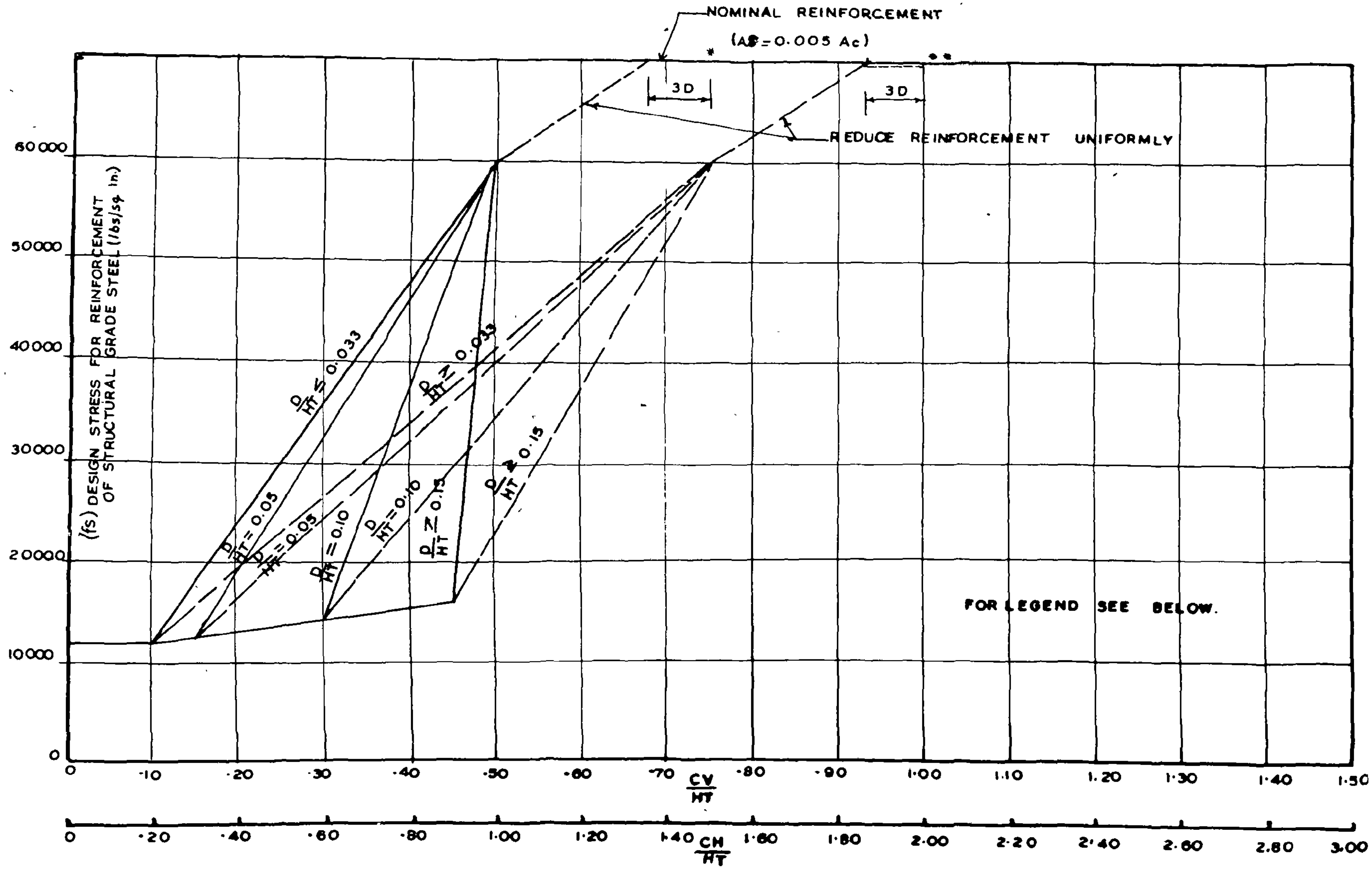
CONCRETE LINED TUNNELS
ESTIMATE OF EFFECTIVE ROCK COVER

FIG. 6·2

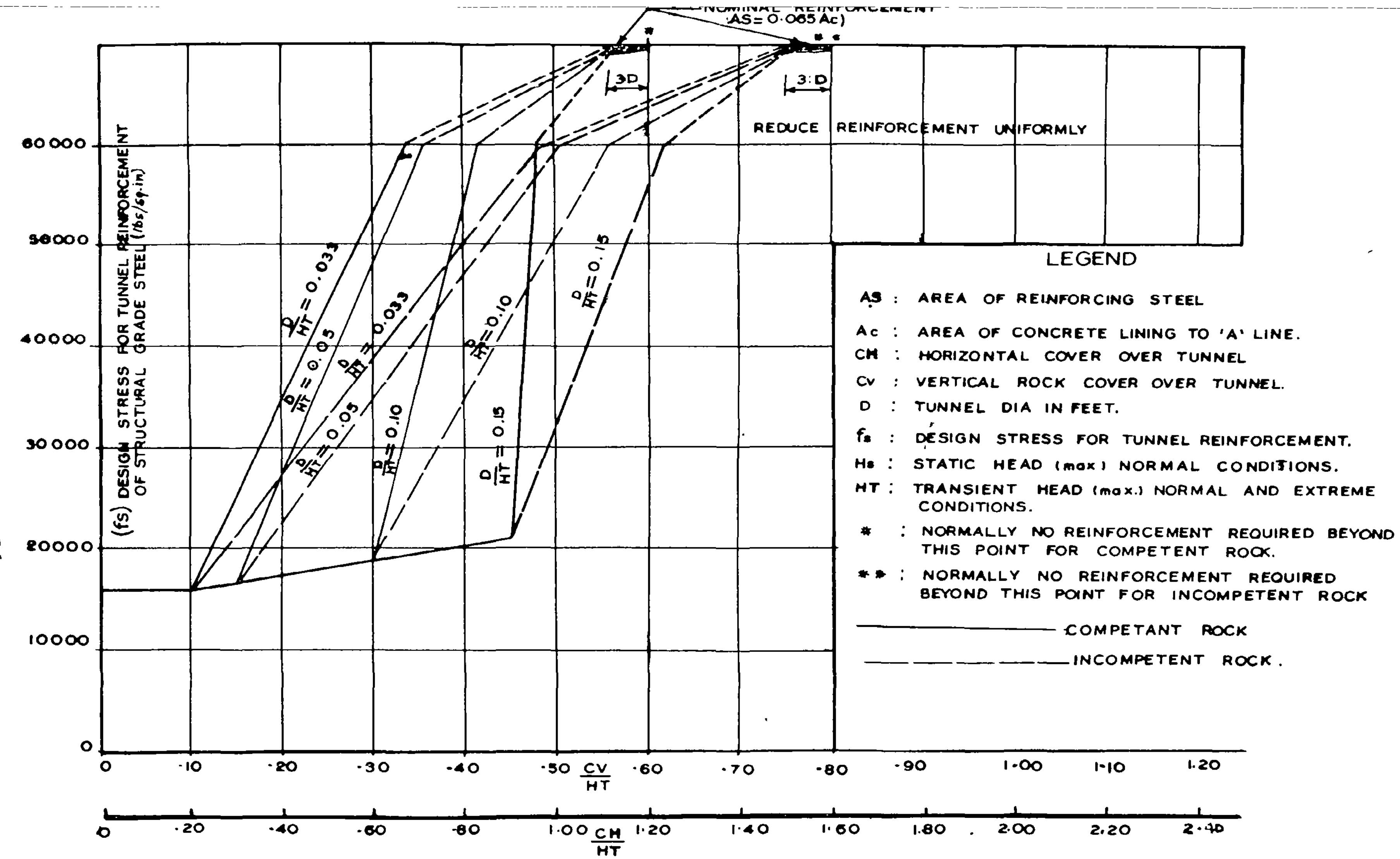


NORMAL DESIGN LOADING CONDITION
(STATIC HEAD)

LEAKAGE IMPORTANT



NORMAL DESIGN LOADING CONDITION
(TRANSIENT HEAD)
LEAKAGE UNIMPORTANT



EXTREME DESIGN LOADING CONDITION
LEAKAGE UNIMPORTANT

CONCRETE LINED TUNNEL

CONCRETE LININGS SUBJECT TO INTERNAL PRESSURES DESIGN CRITERIA

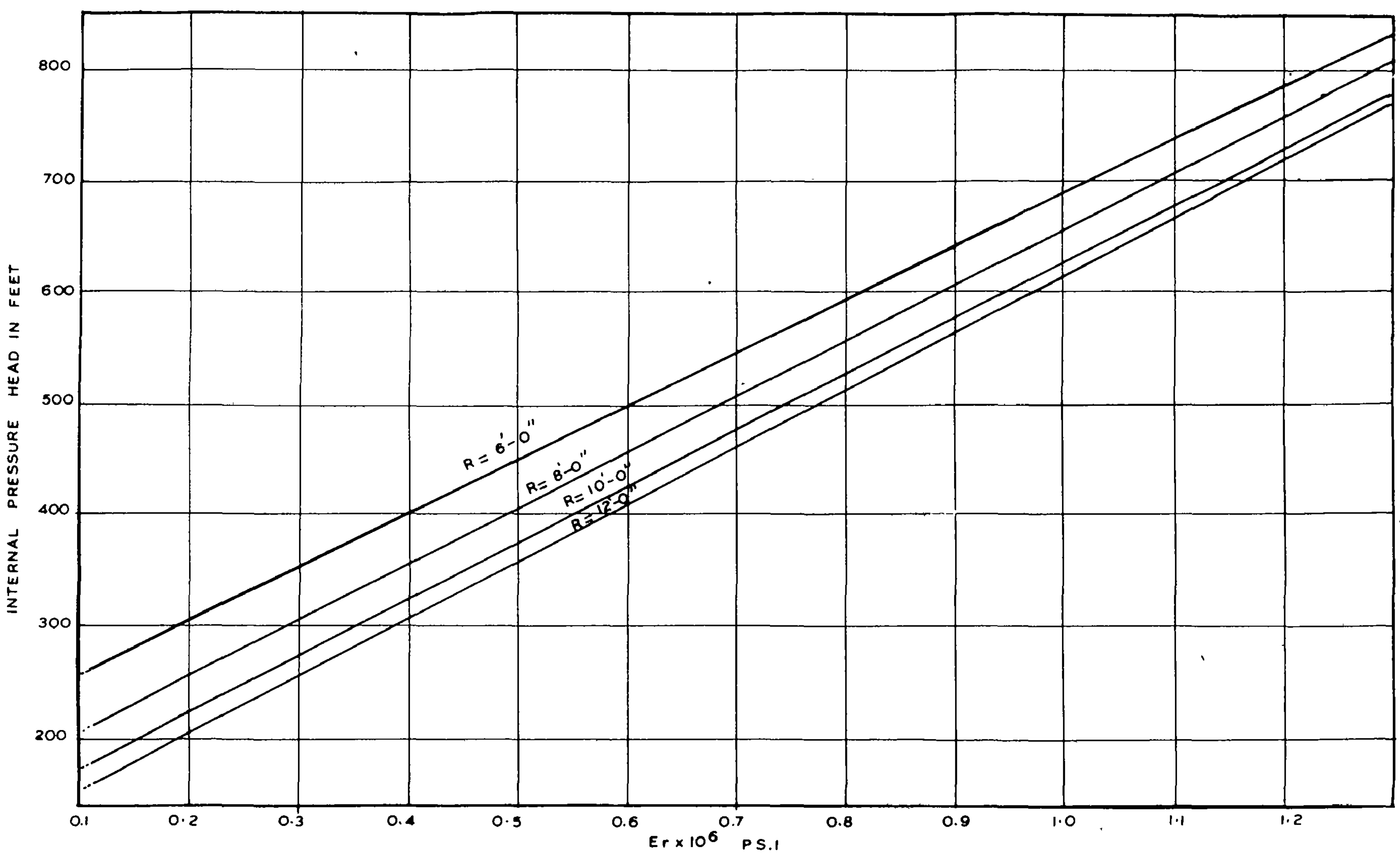


FIG. 6·3 RELATIONSHIP BETWEEN
MODULUS OF DEFORMATION OF ROCK "Er" & INTERNAL
PRESSURE FOR AS = 4.82 SQ IN. AND Fs = 16000 P.S.I.

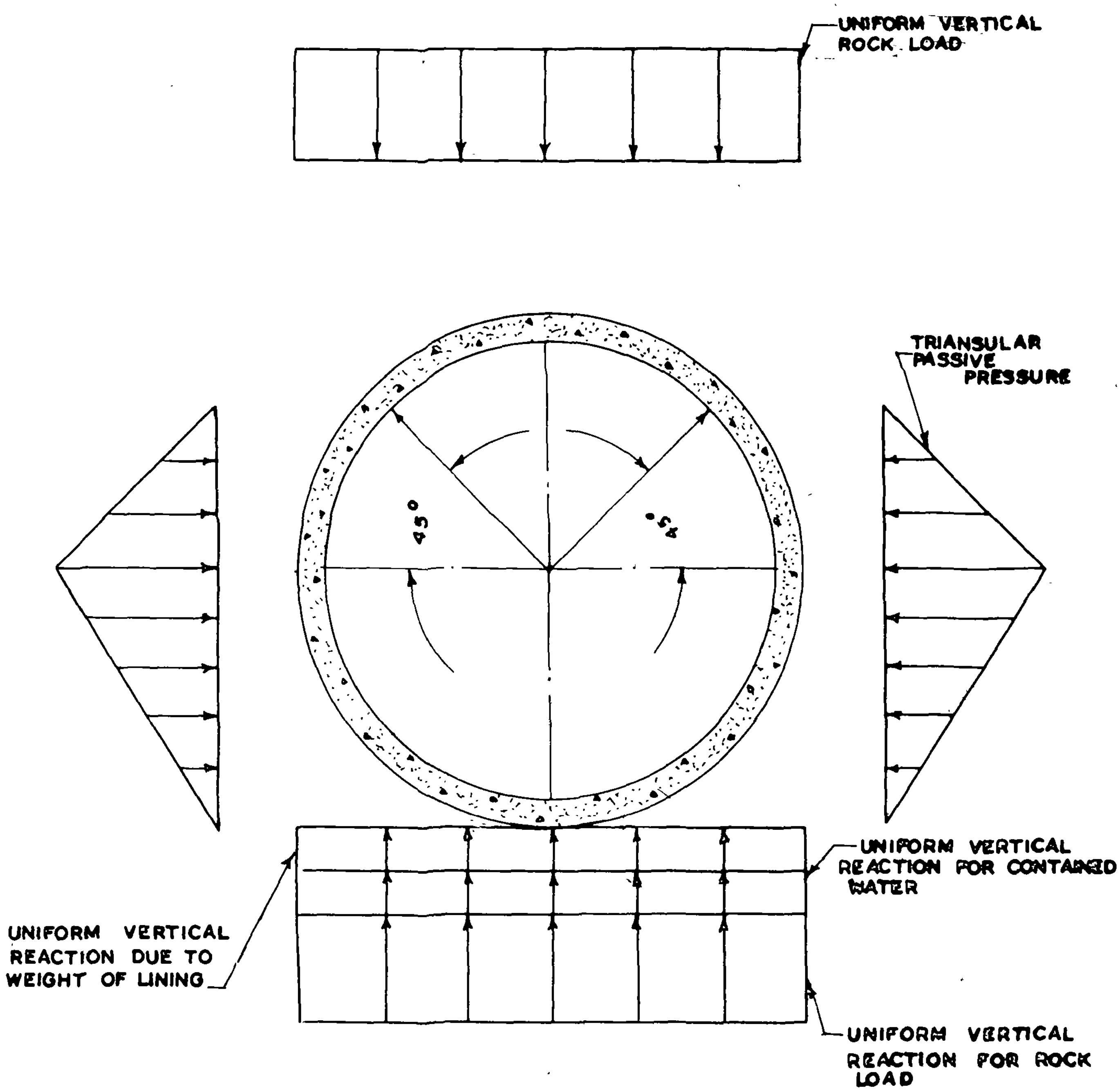
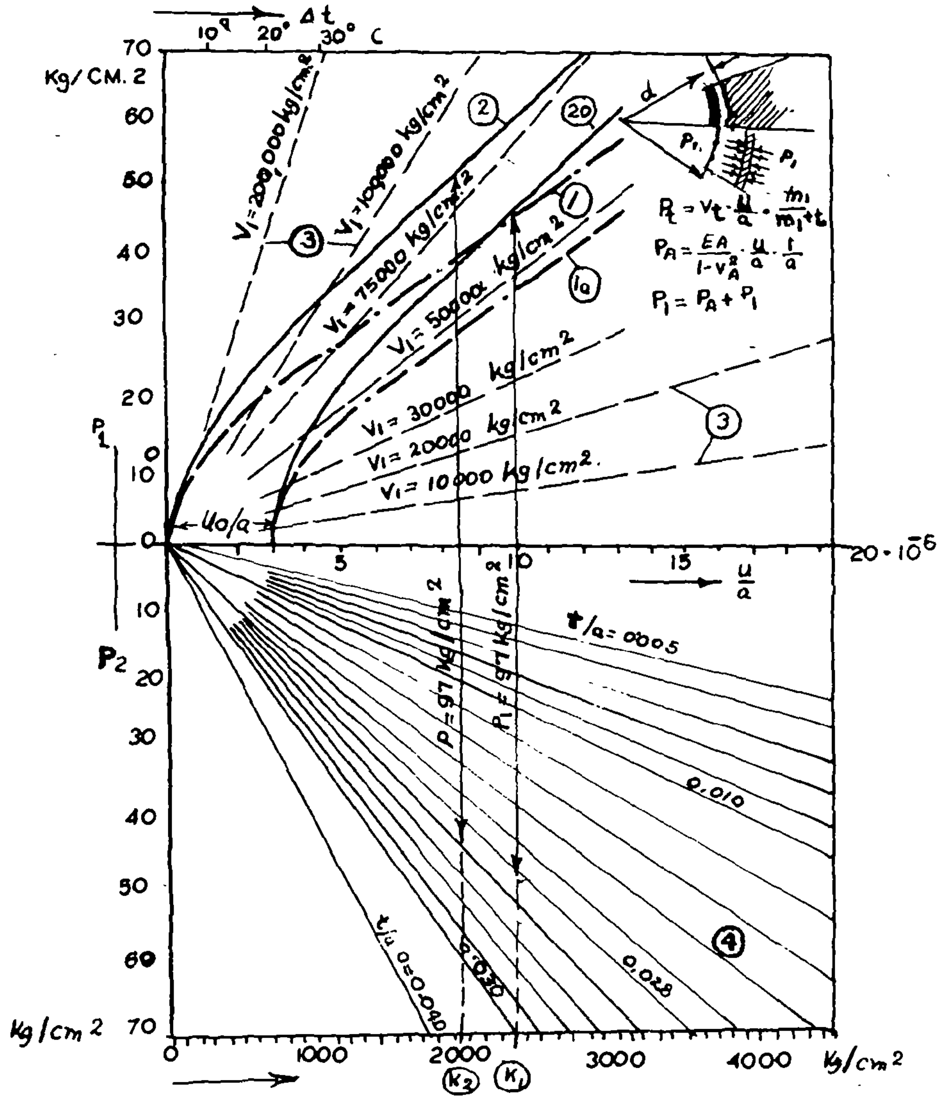


FIG.-6.4 EXTERNAL LOADS ON LINING

FIG.-6.5



DESIGN TABLE FOR STEEL LINING

1 DEFORMATION-LINE FOR NON GROUTED LAMINATED LIMESTONE ACCORDING TO TEST ZONE F.II

2. DEFORMATION—LINE FOR GROUTED LAMINATED LIMESTONE ACCORDING TO TEST ZONE F. II

(1a), (2a) PARALLELLY DISPLACED DEFORMATION LINE CONSIDERING A GAP u_g.

3. DEFORMATION-LINE FOR A CONSTANT MODULUS OF DEFORMATION V

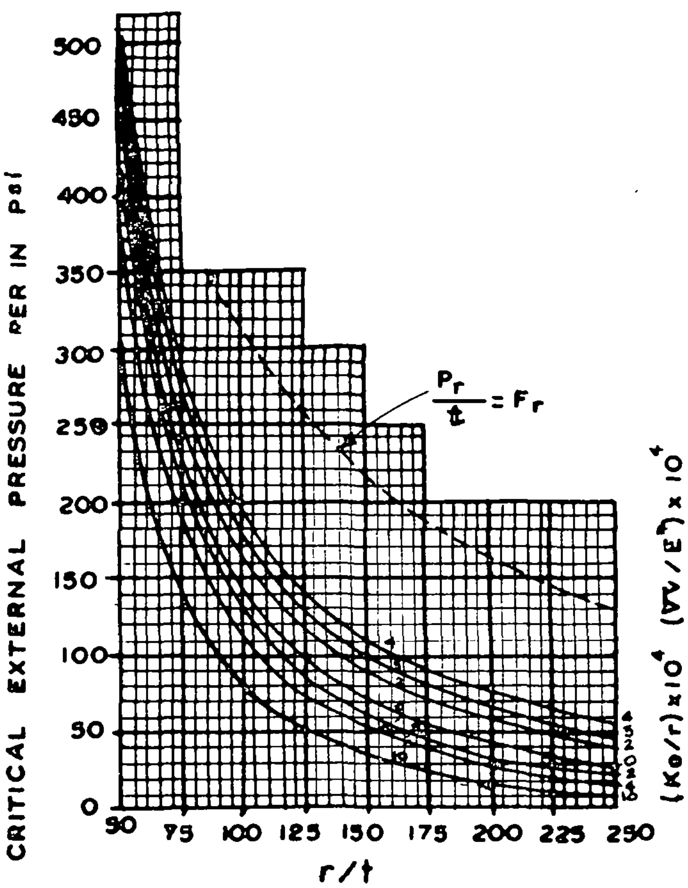
4. PORTION OF INTERNAL PRESSURE TAKEN UP BY THE LINING FOR DIFFERENT PROPORTIONS OF THICKNESS TO INTERNAL RADIUS $\frac{t}{a}$

(K_1) (K_2) EXAMPLES OF DIMENSIONING.

FIG.-6.6

ASTM-A 515 F_r = 32 000 psi

THEORY OF E. AMSTUTZ SEE <u>SCHWEIZERISCHE BOUZEITUNG</u> APRIL 18 1955



E = MODULUS OF ELASTICITY OF STEEL
= 25×10^4 psi

E^* = $E/(1-u^2) = 30.9 \times 10^6$ psi

U = POISSON'S RATIO = 0.25

F r = YIELD STRESS OF STEEL

t = THICKNESS OF STEEL LINER

r = RADIUS OF STEEL LINER

p = EXTERNAL PRESSURE

pcr = CRITICAL EXTERNAL PRESSURE

ko = INITIAL RADIAL GAP BETWEEN STEEL LINER AND CONCRETE

σv = CIRCUMFERENTIAL PRESTRESS IN LINER.

CRITICAL EXTERNAL PRESSURE FOR PLATE STEEL LINER

FIG.-6.7

EQUATION :—

$$\left(\frac{Y_o}{R} + \frac{\sigma_N}{E^*}\right)\left(1 + \frac{3K^2\sigma_N}{E^*}\right)^{3/2} = 1.68\,K\left(\frac{\sigma_Y^* - \sigma_N}{E^*}\right)\left(1 - \frac{K}{4}\,\frac{\sigma_Y - \sigma_N}{E^*}\right)$$

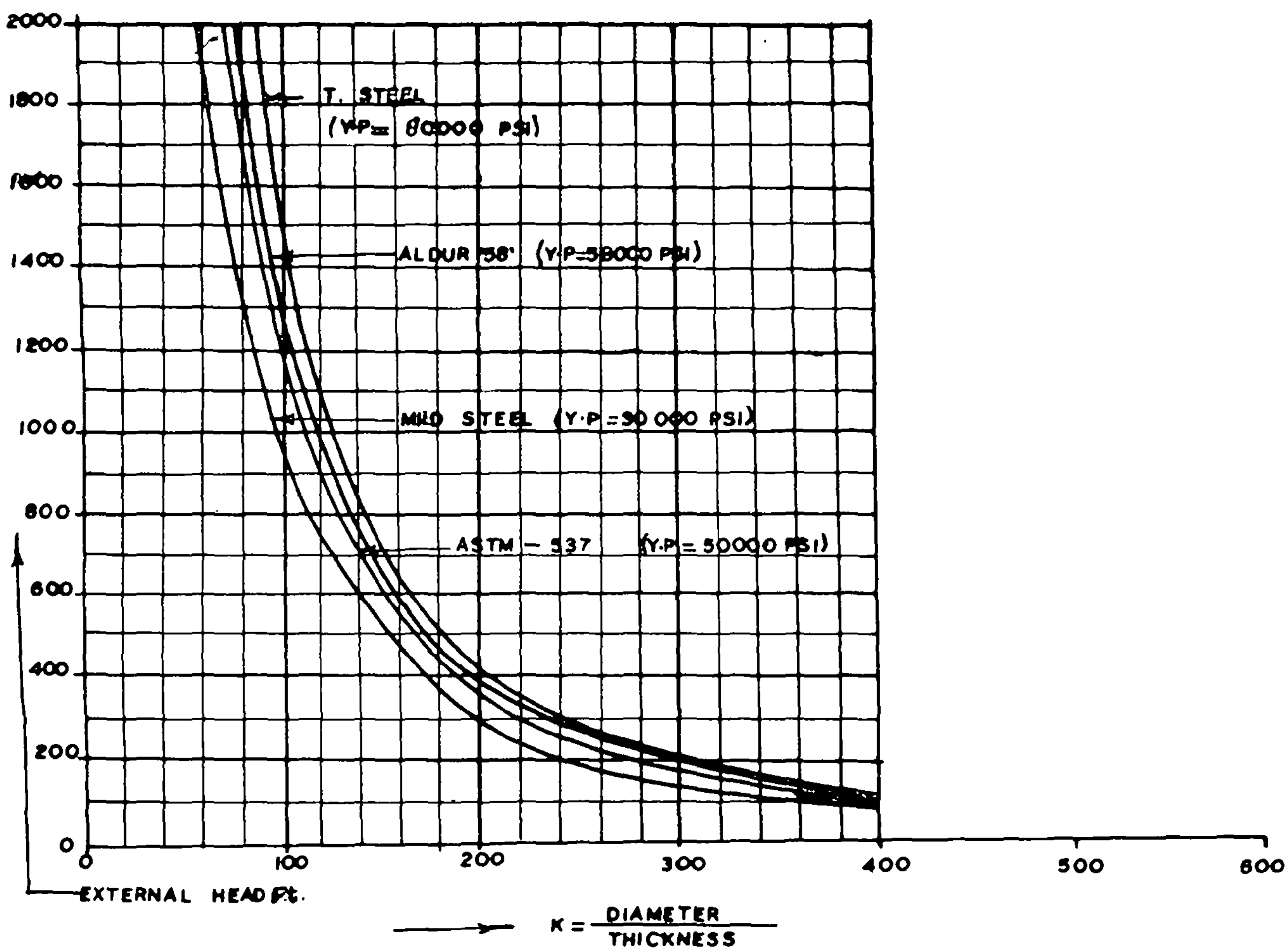
AND

$$1 - \frac{PK}{2\,\sigma_N} = 0.175\,\frac{K}{E^*}\left(\sigma_Y^* - \sigma_N\right)$$

$$E^* = \frac{E}{1-u^2} \qquad \sigma_Y^* = \frac{\sigma_Y}{\sqrt{1-u+u^2}}$$

$$K = \frac{D}{t}$$

$$\frac{Y_o}{R} = 3\times10^{-4} \quad \& \quad E = 30\times10^6\ \text{PSI}$$



CRITICAL BUCKLING PRESSURE
AMSTUTZ CURVE
FOR VARIOUS TYPES OF STEEL

CHAPTER 7

TUNNEL GROUTING

7.0 General

As mentioned in Chapters 5 and 6 on tunnel supports and tunnel lining respectively, the job of tunnel support or tunnel lining does not end with the installation of a support system (steel ribs, rock bolts, shotcrete, etc.) and provision of lining (P.C.C., R.C.C. or steel). For arresting the further displacements and movement of the strata around an excavated and supported underground opening it becomes absolutely necessary to backfill the space between the supporting system and the excavated surface. As explained in Chapter 6, this backfilling is done by means of placing tightly packed concrete behind the supports. Experience shows that whatever cares might have been taken to place the backfill concrete tightly, there are always certain interstices left in the concrete and also between the concrete and the excavated rock surface. In order to fill up these interstices and also to provide a proper bond between the concrete and the rock surface, grouting is recommended. In the case of steel lined tunnels, grouting is done in order to fill up the gap formed between the steel liner and the backfill concrete because of shrinkage of concrete, in addition to the requirements listed above.

7.1 Types of Grouting

Grouting in tunnels is of two types:

(a) Backfill or Contact grouting; and
(b) Pressure grouting or consolidation grouting.

Each of the above two types of grouting has its own specific functions and should be done judiciously.

7.1.1 *Backfill or Contact Grouting*

The main purpose of the contact grouting is that it fills up all voids and cavities between the concrete lining and the rock. Contact grouting is also sometimes known as low pressure grouting or backfill grouting as it shall normally be done at pressures not exceeding 5 kg/cm^2. The normal range of pressure varies from 2 kg/cm^2 to 5 kg/cm^2.

Backfill grouting should be done after the concrete in lining has gained strength. The period of waiting ranges from 21 to 28 days.

Backfill grouting is limited to the arch portion of a D-shaped tunnel or cavity whereas in the case of vertical shafts and steel lined tunnels, it should be done all along the full periphery. The depth of grout holes for backfill grouting shall be such that at each location, the holes extend 30 cm, beyond the concrete lining into the rock. The location of grout holes and the sequence of grouting adopted for the tunnels of Chucka H.E. Project (Bhutan) are shown in Figure 7.1. The exact location of the holes may be varied slightly or additional holes provided depending upon the actual excavation profile at any section. The spacing of the sections shall normally be 3 m, centres but the exact spacing may be varied depending upon site conditions.

7.1.2 *Consolidation or Pressure Grouting*

In the conventional drilling and blasting method of tunnel excavation, the rock around the cavity gets shattered to a certain depth depending upon the depth of blast holes and the type of rock. The aim of consolidation grouting, as the name suggests, is to consolidate the shattered rock by filling up the joints and discontinuities in the rock which got opened out during blasting operations. As a result of pressure grouting, the rock quality gets improved thus increasing the resistance of the rock to carry internal water pressure. Thus, pressure grouting is of utmost importance so far as hydraulic tunnels are concerned. However, IS: 5878 (Part VII) 1972 also recommends the use of pressure grouting around large permanent openings like surge shafts, power houses, access tunnels etc. The consolidation grouting is done after the backfill grouting is completed in a length of atleast 60 m ahead of the point of grouting. Pressure grouting shall be done all around the cavity and for a uniform radial distance equal to atleast 0.75 times the finished diameter of tunnel from the finished concrete face. The spacing of holes should neither exceed 3 m nor the depth of the holes and should be the lesser of the two. The grout holes should be staggered in alternate sections, the spacing of the sections being 3 m centres. In zones requiring special treatment as in the case of crushed rock, the spacing of the sections as well as the number of grout holes could be altered. Depending upon the rock formations and the grout intake, the consolidation grouting should be done in one or more stages with increasing pressures. As a thumb rule, the grout pressure shall not exceed twice the depth of rock cover on the supports so as to ensure safety against uplift of the overburden. The normal range of pressure grouting shall be 7 to 10 kg/cm^2.

In most of the recent tunnels where new tunnelling methods have been used and rock cover is greater than the head of water, depth of grouting holes provided is less than the recommended depth of 0.75 times the finished tunnel diameter. In such cases, the contact and consolidation grouting has been combined in one operation by drilling holes about 50 cm into the rock and grouting them. Such holes are provided on the top half of the tunnel where voids are likely to be present. Figure 7.2 shows the typical location of grout holes and the sequence of grouting adopted for Chukha Project tunnels.

7.2 Process of Grouting

The process of grouting consists of the following operations :

(a) Drilling holes
(b) Cleaning and Washing holes
(c) Testing holes
(d) Grouting holes
(e) Testing of grouted zone for efficacy of grouting.

7.2.1 *Drilling Holes*

It is recommended that drilling through the lining should be avoided to the maximum possible extent. This is generally feasible also since the pattern ofgrouting is fixed before hand and G.I. pipes are placed in position while concreting. This would ensure that the holes are located as per the design and no unnecessary damage is done to the concrete lining. The nominal size of grout holes is kept as 40 mm and the size of G.I. pipes placed in the concrete lining shall be 50 mm internal dia.

It is desirable that a drill hole is grouted before drilling the adjoining holes so as to avoid the blocking of holes by the flow of grout if the adjoining holes are interconnected. In all types of grouting, it is mandatory that the side holes are drilled and grouted first before drilling and grouting the crown holes.

7.2.2 *Cleaning and Washing Holes*

The holes after being drilled to the desired depth shall be cleaned by blowing air through compressors and then washed by flushing water under pressure.

7.2.3 *Testing Holes*

The holes are tested for water intake to ascertain the efficacy of the grouted holes later on.

7.2.4 *Grouting Holes*

Grouting is done by injecting the grout mixtures through grout pumps into the grout holes. In the semi-automatic type of grouting pumps where the control of grouting pressure is done manually, a return line equipped with a pressure relief valve must be provided on the manifold as a precautionary measure against the application of excessive grout pressures.

The normal range of grout mixtures shall be between 5 : 1 to 0.5 : 1 (ratios of weight of water and cement) depending upon the site conditions and the results of the water tests conducted earlier. Generally, the mixture used for contact grouting shall be thickest (in the range 0.5 : 1 to 1 : 1) whereas thin mixtures in the range 1 : 1 to 5 : 1 shall be used for pressure grouting. It is advisable not to use any additives in the grout mixtures which consists of cement and water only. However, on economical considerations inert material like puzzolana, fine sand, clay, bentonite, stone dust etc. may be added to keep down the cost of grouting operations if the intake is heavy.

Once the grouting of a hole is commenced it should be continued without interruption until completion. In general, grouting should be considered complete when the intake of grout at the desired limiting pressure is less than 2 litre per minute averaged over a period of 10 minutes for pressures more than 3.5 kg/cm^2 and 1 litre per minute for pressures lower than 3.5 kg/cm.^{2}

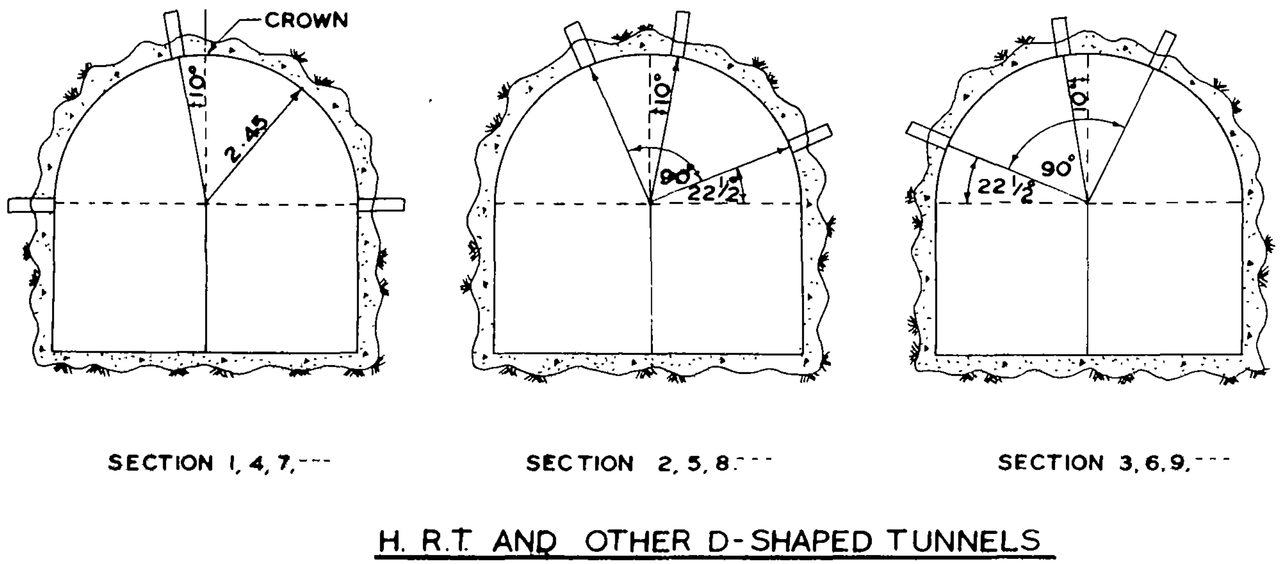
After the completion of grouting operation, the holes should be closed by means of a valve to maintain the grout pressure for a sufficient period to prevent the escape of the grout due to back pressure and flow reversal. The period of closing may range from 1 to 2 hours depending upon the type of strata and the consistency of grout.

7.2.5 *Testing for Efficacy of Grouting*

This testing shall be done by drilling holes in between the two successive grouted sections and by performing water intake test in these holes and comparing it with the results of the tests conducted prior to grouting. If the water intake has not come down sufficiently, further grouting may be considered necessary by increasing the number of grouting planes.

7.3 Maintenance and Upkeep of Grouting Equipment

After each day's job the grout pump, the inlet manifold, the delivery pipe, the return pipe and various valves and fittings should be thoroughly cleaned with water. The nozzles of manifold, pipes and valves etc. should be preferably oiled or greasd to prevent any sticking of cement mixture.



NOTES

1. DIA OF HOLES = 40mm.
2. DIA OF G.I. PIPE = 50 mm
3. DEPTH OF HOLES = 30 Cms BEYOND CONCRETE LINING,
4. GROUT PRESSURE = 2 KG/CM^2 TO 5 KG/CM^2
5. GROUT MIX.= 0.5 : 1 TO 1:1 (RATIO OF WEIGHTS OF WATER AND CEMENT)
6. SPACING OF SUCCESSIVE SECTIONS = 3M. C/C.

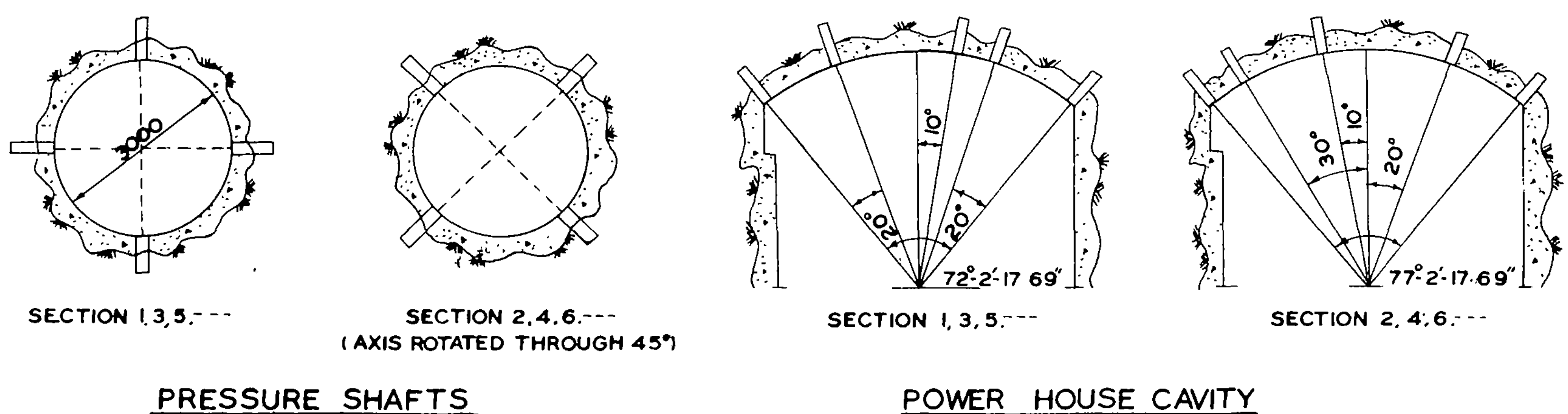


FIG. 7.1 LOCATION OF GROUT HOLES FOR CONTACT GROUTING

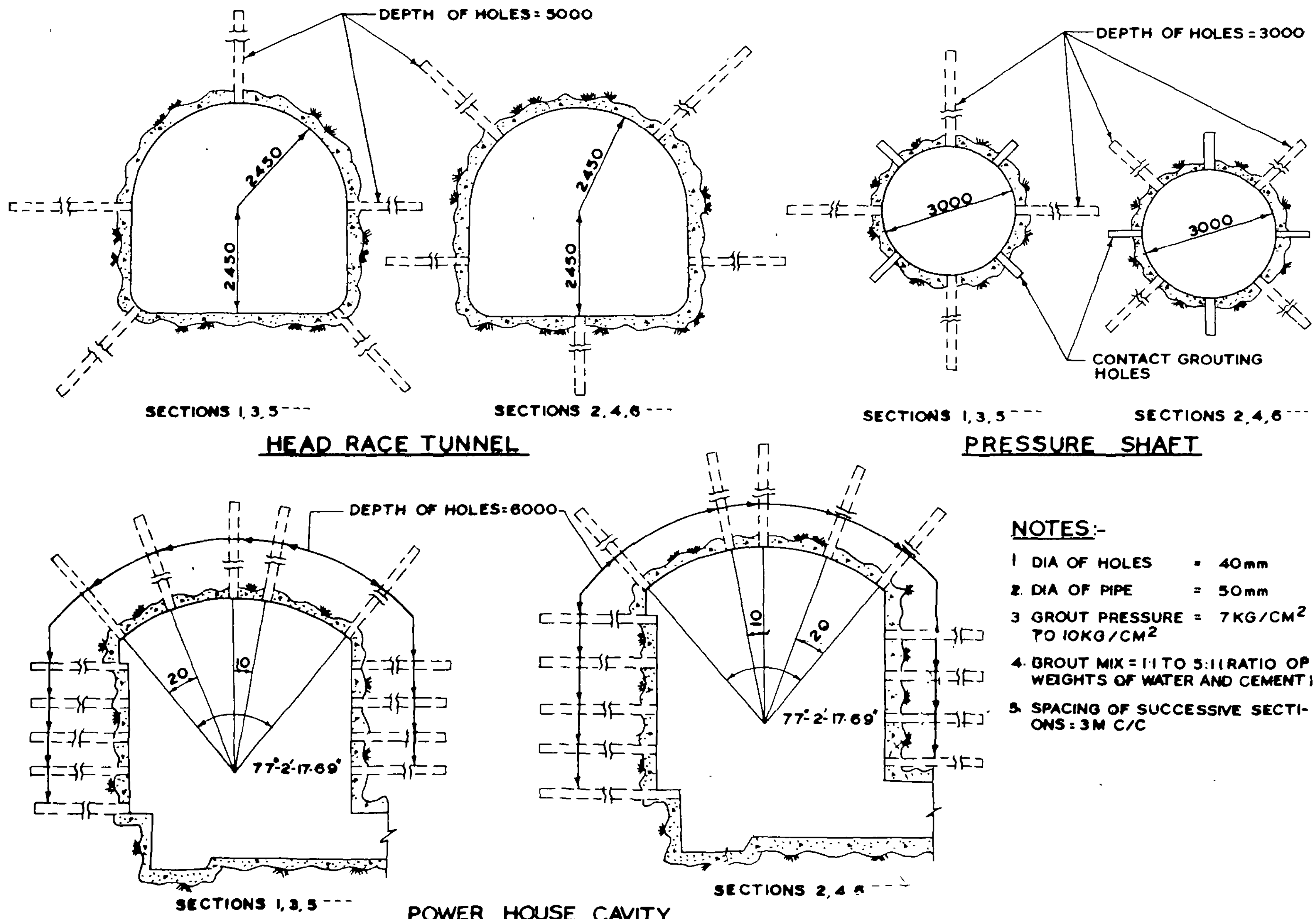


FIG. 7.2 LOCATION OF GROUT HOLES FOR PRESSURE GROUTING.

CHAPTER 8

TUNNEL PORTALS AND PLUGS

8.0 General

After finalising the alignment of the tunnel it becomes necessary to decide the number of working faces for that tunnel. In each and every case of tunnelling, at least two working faces—one on each end of the tunnel—are always available. Depending upon the length of tunnel and the urgency for its completion, the tunnel may be provided with one or more intermediates. Each such adit would yield two more working faces through which tunnelling could progress. For ease of mucking it is desirable to have the intermediate adits in a horizontal plane as has been for Chukha Head Race Tunnel and diversion tunnel, Salma penstock tunnel and many more hydraulic tunnels. However, depending upon the topography of the area, in certain cases it is possible to provide only vertical intermediate adit as has been done in the case of Loktak Head Race Tunnel, Beas-Satluj Link Tunnel etc.

The horizontal adits which have no hydraulic function after the tunnel has been completed are plugged with mass concrete. The vertical adits often play a very important hydraulic function—that of acting as small surge shafts which help in controlling surges in the tunnel and hence are never plugged.

From the foregoing discussion it is evident that the tunnel designer is posed with two problems at the start of any tunnelling operation and one problem at the end of such operations. To begin with, the designer is required to provide detailed drawings and design of a suitable portal at each working face. A similar portal may also be required at each junction of the tunnel with the adit or any other tunnel. Finally, the designer has to provide a suitable plug in the adit so that it is not ruptured or sheared-off even during the course of worst hydraulic conditions. The above aspects have been discussed in detail in the following paras of this Chapter.

8.1 Tunnel Portals

As stated earlier, whenever tunnel operation is to be started, a portal has to be constructed at the working face. Thus, it is obvious that an approach road has first to be constructed to reach the working face. It has been seen that generally rocks near the working face are highly weathered and the first few metres are nothing but loose overburden. This loose overburden is first of all removed and thus a working platform at the invert level of the proposed tunnel is made available. The outline of the tunnel face is then marked on the exposed rock face and an R.C.C. or steel framed portal is constructed around the periphery of the proposed tunnel. The main function of the portal is to provide a well defined access to the tunnel and to protect the tunnel face from loose overburden falling above the tunnel opening. The structural design of the portal is thus very simple. Generally the load coming over the beam of the portal is the self-load of the beam plus a uniformly distributed live load calculated by assuming 45° dispersion above the beam. Annexure-IV gives the typical calculations for a tunnel portal designed for the surge shaft expansion gallery portal for Chukha H.E. Project (Bhutan). Figure 8.1 shows the details of the portal.

Sometimes, when the tunnelling conditions warrant, a portal is required to be provided at the junction of the adit tunnel with the main tunnel or in places where tunnel intersection and branching takes place. In such conditions, the load coming from the larger tunnel is required to be supported by the portal provided at the junction. Figure 8.2 shows the typical details of such a portal at the junction of main and branch tunnels for Salma Dam Project (Afghanistan). The load coming from the ribs of the major tunnel can be considered to be acting uniformly on the portal beam. The beam moments and the column moments are worked out using simple moment distribution methods. For the Salma Dam Project, both the alternatives viz., R.C.C. as well as steel portal frame were designed as shown in Figure. 8.2. The steel portal, wherever provided could later be embedded in concrete.

8.2 Tunnel Plugs

When all the tunnelling operations are over and the tunnel is ready for filling, the various intermediate adits are plugged with suitable concrete plugs. Similarly, the diversion tunnels are also plugged after the completion of the dam, unless these are envisaged to be used later as spillways. In a few instances hemispherical or elliptical shaped bulk head type steel tunnel plugs have also been envisaged. However, it is recommended that such steel plugs should not be used as permanent plugs in view of the inhernt weakness of steel against water.

The concrete plug should be so designed that it is safe in shearing along its periphery due to the maximum thrust expected during the worst hydraulic conditions. If adequate length of the plug is not provided, the contact area between the periphery of the plug and the rock perimeter in the adit tunnel is less and the plug might fail in shear. The friction coefficicnt "μ" between the rock and concrete could be assumed to have a value varying from 0.25 to 0.35. To being with, the length of the plug is taken equal to the diameter of the tunnel to be plugged. Since the cross-sectional area of the plug is also known, the weight W of the plug is calculated. The total frictional resistance force F_1 provided by the plug thus works out to "μW".

Alternatively, the value of force F_1 could be estimated if field tests are carried out to determine the value "$\tan \phi$"—the internal friction factor of the rock mass. The force F_1 would then be $W \tan \phi$.

Along with the frictional resistance due to the weight of the plug, another resisting force due to the bond developed between the rock and the plug concrete also helps in resisting the hydraulic thrust "T". For calculating the force due to bond, tests are carried out in the tunnel to be plugged and the value of shear coefficient "C" of the rock is obtained. The force F_2 due to the bond resistance could then be calculated by multiplying "C" with the contact area of the plug. Having calculated the forces F_1 and F_2, the factor of safety is calculated as under :

$$F.S. = \frac{F_1 + F_2}{T}$$

It is usual to provide a high value for the factor of safety because of the uncertainty in the values of F_1 and F_2 calculated from field test data. Usually the plug is considered to be safe if the factor of safety lies between 4 to 5. If the factor of safety works out to less than 4, length of plug is increased and the calculations are repeated till the value of factor of safety falls between 4 and 5.

Using the above procedure, designs of various plugs for Bhaledh feeder tunnel of Baira-Siul H.E. Project were worked out. Figure 8.3 brings out the details of various plugs provided on the tunnel.

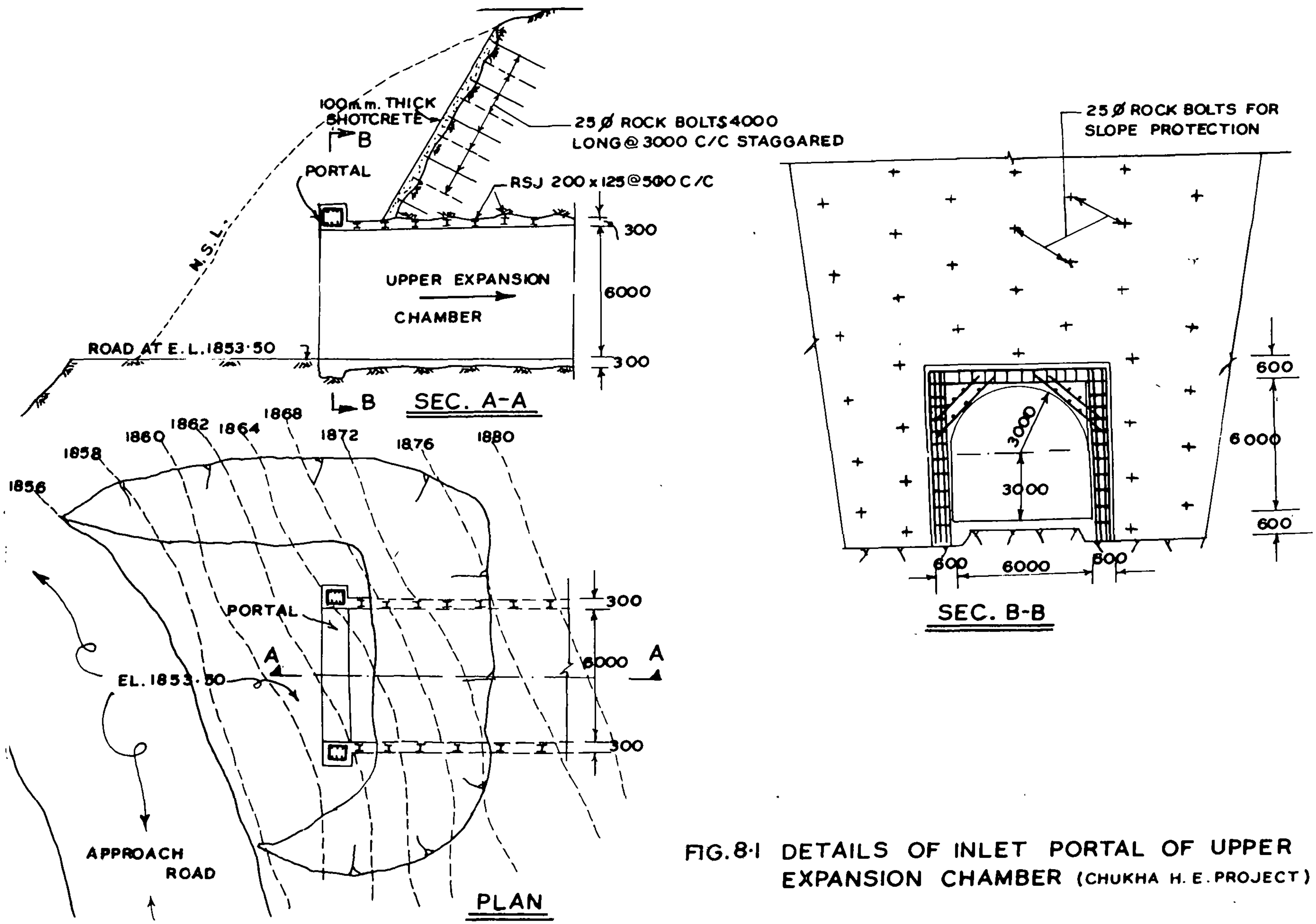


FIG. 8·1 DETAILS OF INLET PORTAL OF UPPER EXPANSION CHAMBER (CHUKHA H. E. PROJECT)

FIG.-8·2 PORTAL AT JUNCTION OF DIVERSION TUNNEL & POWER TUNNEL (SALMA PROJECT) AFGHANISTAN

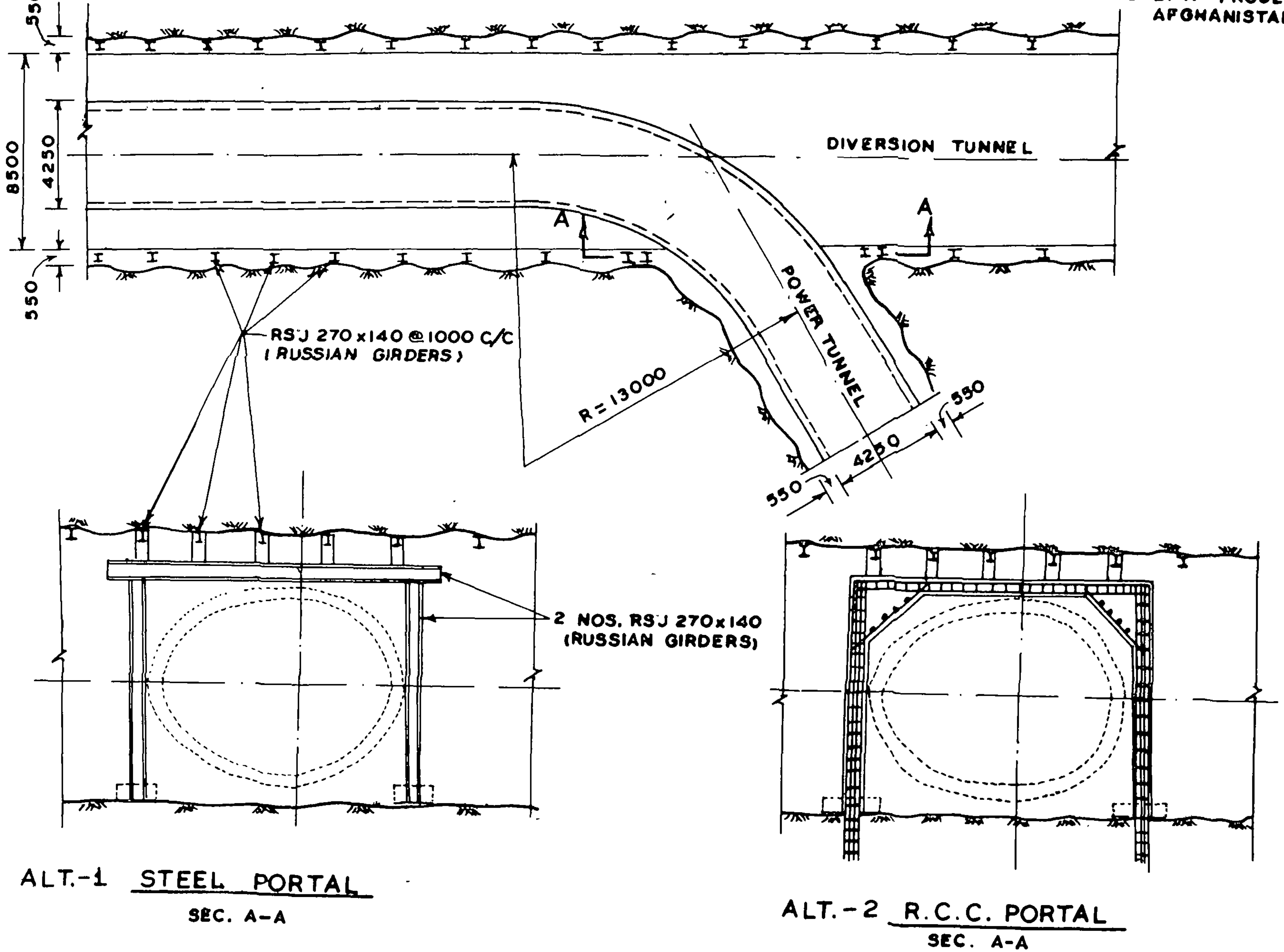
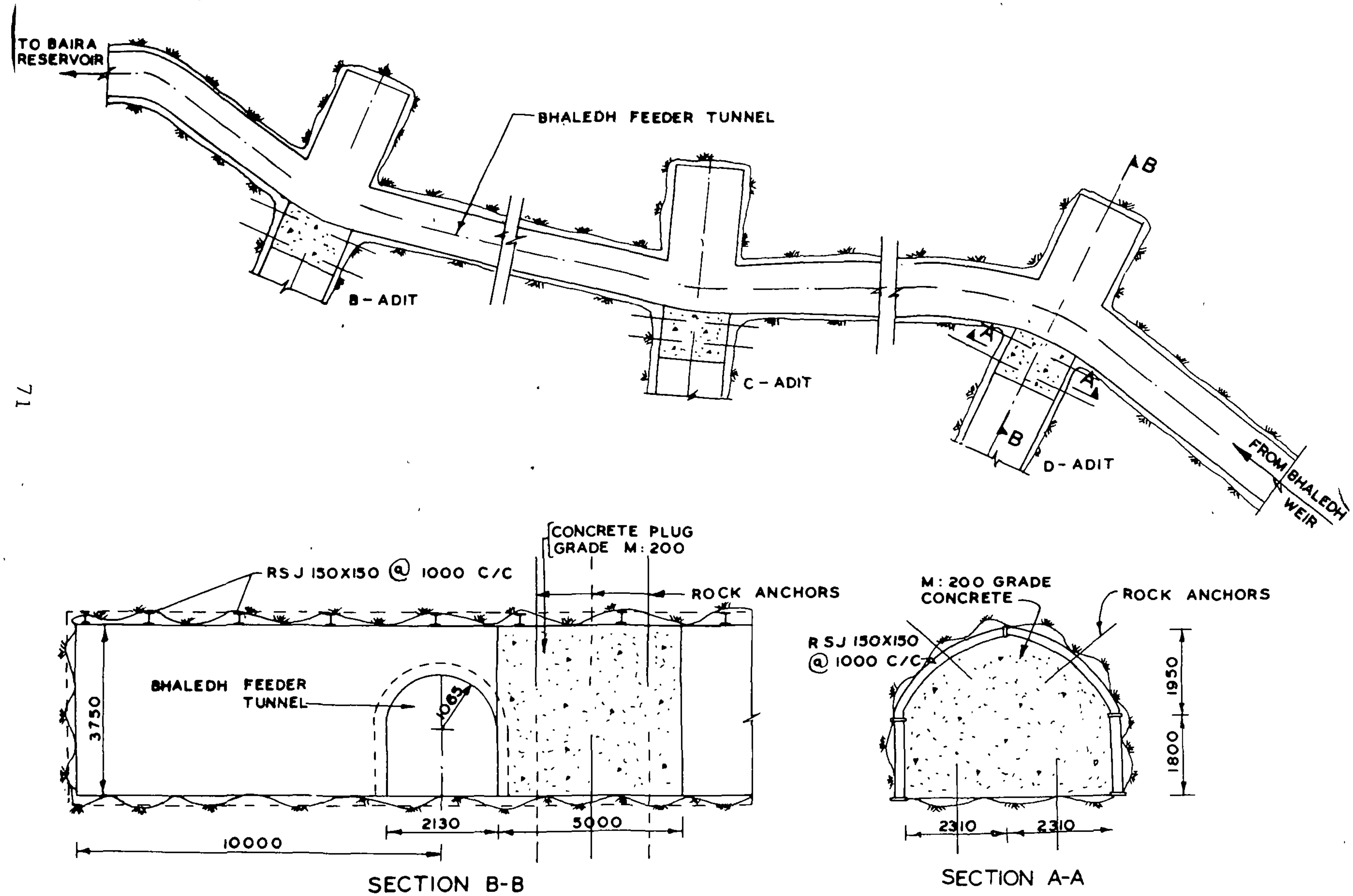


FIG. 8·3 TYPICAL DETAIL OF CONCRETE PLUG FOR BHALEDH FEEDER TUNNEL (BAIRA-SIUL H.E. PROJECT)



CHAPTER 9

SPECIAL PROBLEMS IN TUNNELLING

9.0 General

In the preceding chapters, the descriptions applied more or less to the normal conditions in rock tunnelling. The rock was excavated by blasting; it was strong enough to bridge the span of the opening at least upto the ventilating time; the side pressures were completely absent or were relatively low and the bottom of the tunnel was firm enough to support the footings of the ribs.

However, now and then one encounters such rock conditions also which differ from the normal conditions listed above in more than one respect. Sometimes the rock is so thoroughly crushed or decomposed that it does not require blasting and can be easily excavated by the use of pneumatic or hydraulic shovels. Sometimes, a rock tunnel may intersect buried valley filled with soft, clayey sediment. At times a tunnel may intersect a saturated strata having perched water which might lead to tunnelling under heavy seepage or flowy conditions. A most tricky problem might arise if a tunnel intersects strata having "Squeezing" or "Swelling" conditions. The term "Squeezing rock" is applied to rock formation which squeezes into the excavated opening unless held back by an effective support system. Generally such a condition occurs when the rock deformations are excessive under the action of external loads—both vertical as well as horizontal. Any material which is intermediate between rock and clay constitutes "Squeezing rock". If the clay content has montmorillonite as its main component (which has a great affinity for water) the squeez is chiefly due to volume expansion and the rock is termed as "Swelling rock". Thus, swelling rock coupled with heavy seepage conditions yields the most difficult tunnelling operations.

Sometimes vertical or inclined shafts are required to be constructed to serve as construction adits or as permanent works, such as access tunnels, surge shafts, horizontal and inclined pressure shafts, cable tunnels, ventilation tunnels etc. for underground power stations. Tunnelling methods adopted for such special features are also discussed in this chapter.

9.1 Tunnelling in Firm Ground

Soft and stratified rocks such as sand stone, shale, cemented sand and gravel, hard clays etc. which have a bridge action period just enough to install supports within the mucking period fall under this category. The methods of excavation depend upon the size and shape of the tunnel, availability of equipment and the actual stand up time. Such ground can be excavated either by the conventional drilling and blasting method or by the use of the tunnelling machines.

9.1.1 *Conventional Methods*

All the methods discussed in Chapter 5 under the conventional drilling and blasting method can be used if the size of the tunnel is small and reasonably enough stand-up time is available. However, as the tunnel bore increases and as the quality of rock deteriorates, the methods best suited are the Side Drift Method and Multiple Drift Method.

9.1.1.1 SIDE DRIFT METHOD

This method is particularly suited for large size tunnels through bad rock which require support before mucking. As shown in Figure 9.1, the side drifts are driven ahead of the main excavation for some convenient distance. The support of these drifts includes the main tunnel supports and wall plates. Just prior to shooting the main bore, the drift support is removed leaving the main posts and wall plates in position. These project above the muck pile thus permitting erection of the main arch ribs before mucking out.

9.1.1.2 MULTIPLE DRIFT METHOD

This method is a combination of side drifts and top drift. It is found suitable for large openings in crushed rock formations in fault zones which may behave like earth, even though the rock is compacted enough to require light blasting.

As shown in Figure 9.2, a side drift is driven first on each side. A concrete side wall is placed in each drift with adequate provisions for free drainage of seepage water. If the height of the side walls happens to be too great to build in a single side drift, another side drift may be driven immediately above and the concrete side walls carried on upto the spring line as shown in Figure 9.2.

A top centre drift is then driven through, with the roof support far enough above the future position of the main tunnel ribs to provide space for crown bars over the ribs. A small section of the drift roof is blocked on the crown bars and the drift side posts are removed. The top drift is then slowly widened by means of short shots to connect with the roofs of the side drifts. The main arch ribs are erected on the concrete side walls, lagged and thoroughly packed. The crown bars supporting the roof members of the top centre drift are securely blocked to the ribs, where upon the next advance can be made.

It is preferable to provide a 2-piece continuous rib type steel support for the main tunnel.

9.2 Tunnelling through Soft and Running Ground

Sometimes a rock tunnel might suddenly pass from good rock conditions to material with no cohesion such as clean sand and gravel. Such soft material is termed as running ground. Such lengths of tunnel require the use of forepoling method in combination with steel supports.

Figure 9.3 illustrates the use of this method. The boards which are driven ahead to support the ground ahead of the last rib are known as spiles. They act as contilevers which carry the weight of the ground until their forward ends are supported by installing the next rib. The spiles are installed as far down around the sides of tunnel is necessary. Then the top breast boards are removed, the exposed ground is excavated and the breast boards are reinstalled ahead as shown by dotted lines in Figure 9.3. This process is continued until the excavation arrives at floor level whereupon the next rib can be installed. At sometimes afterwards the tails of the spiles are cut-off. This method was successfully used in driving a 10 feet finished dia. horse-shoe shaped Sierra Madre tunnel near California, U.S.A. and Kudremukh Iron Ore Project—slurry pipe line tunnel.

If the running ground is a bit hard it may be advantageous to use steel spiles instead of wooden spiles. These steel spiles serve the same purpose as sheet piles in an open excavation. This method of forepoling with steel spiles was used in 8′ 6″ finished dia. horse-shoe shaped sewer tunnel for Ward's Island sewage treatment works in New York City.

9.3 Tunnelling through Soft, Water Bearing Strata

The seewage towards tunnels located below the water-table increases very considerably the difficulties and cost of construction. It also shortens or even eliminates the bridge-action period, reduces the bearing capacity of the tunnel bottom which might cause its heaving up. In soft, porous ground it may create the danger of loss of ground due to soil being washed into the tunnel.

Such adverse conditions in rock tunnels may seldom exceed a few hundred metres but they cause untold harm to the progress of work and the Engineer might even be compelled to divert the tunnel alignment. Hence, if the geological studies carried out during investigations indicate the possibility of such lengths of tunnel, adequate arrangements must be made before hand to cope with the problem.

Free draining ground such as clean sand or gravel cannot be tunnelled through unless it is previously drained or grouted. Materials having a low permeability such as silty sand are more easier to tackle as the quantity of water which seeps into the tunnel out of such material is relatively small and therefore does not interfere with the tunnelling operations.

It will be interesting to mention here the collapse that took place in the 4.9 m dia. D-shaped head race tunnel of the Chukha Project due to the presence of the perched water which got suddenly released as soon as the face of the tunnel at RD 638 m was removed. There was a very heavy flow of the order of 20 cusecs initially which got reduced to about 3 cusecs after about 48 hours. The tunnel was filled with rock debris upto about RD 570 m as shown in Figure 9.4. On examination of the rock debris which consisted of large fragments of gougy material and after carrying out further geological investigations it was suspected that a shear zone existed which was acting as a barrier against the flow of perched water in the hill mass. As soon as the shear zone was punctured at RD 638, the water got released bringing along with it huge quantities of gougy material which filled up the tunnel stalling any further progress temporarily. The steel ribs provided between RD 605 m and 638 m were all collapsed.

After the discharge had come down considerably, attempts were made to restart the work. This was done by removing the collapsed debris slowly, first from the sides followed by instantaneous erection of steel rib posts. Suitable drainage holes sufficiently deep were driven, one at the crown and one each on the sides to tap any further seepage water. The inner most material was removed last of all and steel ribs were erected immediately, supporting them on the side posts erected previously. The ribs were thoroughly lagged and back packed with concrete immediately. The progress of tunnelling which was going on prior to the collapse @ 75 m to 90 m per month came down to just 10 m to 12 m per month.

9.4 Tunnelling through Squeezing Ground

Squeezing ground exerts pressure into the tunnel support from all the sides. Therefore, if such ground is encountered, it is highly desirable to convert the external cross-section of the tunnel to a full circle even if it means enlarging considerably to maintain the finished internal diameter. The steel ribs then act as rings and are subject only to thrust, and not bending thereby deriving the greatest resistance per unit weight of steel used. Since all the steel in the ribs is cold worked there is an increase in the safe stresses that may be considered in design. The full circle ribs should be closely spaced and a tight lagging of sufficient strength to transfer the estimated load onto the ribs should be placed on the outside flanges as shown in Figure 9.5. The ribs should be back filled and grouted as early as possible so as to be completely effective.

9.5 Tunnelling through Swelling Strata

If the rock shows a moderate swelling the method of supporting shall be the same as that mentioned for squeezing rock conditions under para 9.4. On the other hand, if the rock has a high swelling properly, it should be allowed to expand into the tunnel for a considerable period before putting final concrete lining. This could be achieved by providing either a system of yielding lagging with rigid supports or a system of yielding ribs and lagging.

9.5.1 *Yielding Lagging with Rigid Supports*

The basic principle of this method is to instal heavy full circle at a wide spacing of about 1.2 m centres and to let the ground swell and squeez between the ribs till it relieves itself. The method is particularly suitable where the strength of the rock is low enough to squeez through without overstressing the ribs.

The method consists of installing light gauge liner plates between the widely spaced ribs and blowing gravel behind. The ribs should not be blocked at any point but the blocking should be placed behind the liner plates. When the rock swells, gravel is trapped between the rock and the outside flange of the ribs thereby loading the ribs uniformly. The liner plates will start to buckle inward as shown in Figure 9.6, at which time they can be unbolted from the webs of the ribs. The plates below the springing line can be removed permanently and the crown plates can be left resting on the inner rib flange to protect from debris falls as shown in Figure 9.7. The progress of swelling should be watched carefully and the plates should be taken out now and then to remove the fallen muck. The entire thing should be embedded in concrete lining when in the opinion of the engineer and the geologist, the swell is complete.

9.5.2 *Yielding Ribs and Lagging*

If the swelling rock is so firm that it would arch from rib to rib and thus overstress the ribs, it would be advisable to use a yielding rib to enable the rock to make its initial movement. As the rock swells into the tunnel, its strength decreases until it becomes soft enough to squeeze between the ribs.

In this method also the ribs are widely spaced about 1.2 m centres. Each circular rib is made out of several segments. Between the butt plates of each two adjacent segments a Crush Lattice made of soft flexible wood is inserted as shown in Figure 9.6. The space between the ribs is bridged by light gauge liner plates and the lining is backfilled with 12 mm size gravel.

The purpose of crush lattice detailed in Figure 9.6 is to allow the diameter of the ribs to decrease and thereby permit the rock to move towards the tunnel. After the diameter of the ribs has decreased a few liner plates are removed to inspect the condition of rock. If the rock has become soft enough to squeeze through the space between the ribs the subsequent procedure is the same as described under para 9.5.1. If, on the other hand, the rock is still too stiff to squeez through the gaps between the ribs, all the liner plates are removed and rock between the ribs is excavated as shown in Figure 9.7 leaving rock ribs between grooves and the liner plates are laid back on the inner flanges in the crown portion of the tunnel to prevent rock spalls from dropping down.

As the rock swells further, the rock ribs adjoining the outer flanges of the steel ribs are crushed and the debris fills the spaces behind the liner plates. After the spaces are filled the liner plates are removed and new grooves are excavated. Thus, it is possible to permit even in a very stiff swelling rock a large amount of squeeze into the tunnel while the ribs remain intact, they do get slightly deformed. Hence, the initial dia. of the ribs should be such that at the end of the service period, the ribs are still located at or beyond their allocated position in the design drawings.

9.6 Conversion to Circular Sections

As mentioned earlier, in soft running ground and in squeezing or swelling rock conditions, tunnel supports with full circle ribs are most suitable. Although the conversion from the normal type of tunnel section to circular section involves some extra labour and material these points are likely to be unimportant compared to the difficulties and delays associated with adhering to the normal types of tunnel supports in such stretches.

In addition to the fact that full circle ribs are better suited than any others to meet the pressure conditions they also serve as a reinforcement for the permanent concrete lining. Since. the lining is cylindrical the bending moments in the lining itself are very small and the strength of the construction material is thus fully utilized.

Generally, it is enough to change the extrados of a tunnel to circular shape while maintaining the inner finished shape in order to avoid head losses in transitions etc. and also to save the cost of additional form work required in case the intrados is also changed.

Figure 9.8 shows a typical conversion of horse-shoe shaped tunnel to a circular shape in squeezing ground conditions.

9.7 Tunnelling Methods by use of Machines

In many advanced countries excavation of tunnels is being done with the help of machines. The most commonly adopted method of tunnelling through clayey ground is by the use of Road heading machines. These methods shorten the time of tunnelling and permit the use of support methods other than the conventional steel rib supports. However, these machines have been very rarely used in India on account of their high capital cost and non-availability of indigenous cutters.

9.7.1 *Shield Method of Tunnelling*

In shallow underground excavations where the chances of collapse are feared this method is successfully used. It consists of a circular shield made of thick steel plates backed up by adequate stiffners. The face is excavated by pneumatically operated spades. The shield is then pushed into the excavated section by hydraulic jacks, pressing against the previously erected lining. The lining could be of precast R.C.C. or P.C.C. blocks or it could be of steel or cast iron blocks about 60 cm long. The lining segments are held in place by bolt both longitudinally and transversely. As soon as the lining segments are erected and bolted up, the gap between the lining and excavated surface is filled up with 12 mm size gravel and packed with cement grout. The shield affords complete safety and where the face is stable and does not require breasting, rapid progress can be attained. Shield method of tunnelling has been successfully used at Tarbela dam diversion tunnels in Pakistan. It was suggested for use in excavating the headrace tunnel of Loktak H.E. Project in Manipur for squeezing or swelling ground conditions. However, on carrying out further studies, it was found by John Golzer, the Austrian tunnelling expert, that if used at Loktak, the Shield would sink into the ground and therefore the programme to adopt shield method of tunnelling at Loktak was abandoned.

9.7.2 *Tunnelling by the use of Road Heading Machines*

The road heading machine has been commercially developed under the name of Alpine Miner. Basically, it consists of cutting the rock face by rotation of a drum or a circular disc or a circular arm on which are mounted the cutting teeth or wheels or tips. These are of special tungsten carbide or industrial diamonds. The lateral force to press the cutters against the face is provided by Hydraulic systems. The machine cuts the rock and the muck is conveyed out of the machine by a system of conveyor belts. Alpine Miners have developed two types of machines—AM 50 and AM 100. The AM 50 issued for cutting rocks having a crushing strength of upto 400 kg/cm^2 where AM 100 has the capacity of cutting rocks having a crushing strength of about 1000 kg/cm^2. Both these machines are available with the option of a provision of shield also. The speed of tunnelling is very rapid and depending upon the quality of rock, could be as high as 30 m per day. The machines facilitate speedy excavation, a clean cut with practically no overbreaks and considerably less vibrations of the strata.

9.7.3 *Full Face Tunnel Boring Machines*

The use of tunnel boring machines dates back to 1882; but their full industrial development can be assumed to be as recent as 1954. Over the past ten years or so, the use of TBM has become widespread. With this increasing use, major improvements have been achieved in the design of cutters and machines. Further, the overall reliability of the various components of the TBM has been increased to reduce "down time" which was a major problem. As a result, TBM can now be used in rocks with compressive strengths upto 30,000 psi (2200 kg/cm^2). With the use of TBM, it is possible to achieve advance rates of approximately 40 m per day which is nearly 10 times that for drill and last method.

A full face TBM consists of a wheel cutter head fitted with teeth or rollers to cut or spall the rock. The wheel is slightly smaller than the bore of the tunnel and is equipped with gaff cutters to produce the designed bore. The wheel may consist of spokes or of a solid disc with slots to allow the muck to pass through. The wheel is rotated at speeds varying from 4 to 10 rpm. The speed varies according to diameter by means of electrical or hydraulic disc motors. The wheel is forced against the tunnel face by hydraulic jacks which apply a thrust varying between 200,000 and 5,000,000 pounds according to the strength of rock and the tunnel diameter. The wheel is attached at the end of a tube called the "backbone" which contains the electrical controls, the hydraulic pump and the drive motors. The backbone is enclosed within a structural steel framework. This framework is equipped with two or four vertical legs reaching to the floor and to the roof of the tunnel and four horizontal arms that can be jammed against the tunnel walls. These legs and arms provide vertical and horizontal control of the grade and alignment of the machine and, in addition, reaction to combat the torque and thrust of the cutting head. As excavation proceeds, the rock cuttings passing through the slots in the cutting head are picked up in buckets attached around the rim of the wheel and are discharged on to a conveyor belt incorporated in the machine. To advance the machine, the cutter head and backbone can be moved forward from 24 to 48 inches within the frame by hydraulic jacks.

At present, there are about 20 types of TBM on the market, each having special design features. With the rapid development of TBM, and variety of rock characteristics on which such machines are being used at present, it is considered that an existing TBM becomes obsolete within 3 to 5 years.

9.8 Excavation of Vertical Shafts

In most of the hydro-electric projects involving the construction of long head race tunnels, vertical shafts either for the purpose of construction adits or ventilation or surge shafts are invariably provided.

Vertical shafts may be sunk either from top to bottom but when access at the bottom is available, they may also be excavated from the bottom. A small pilot shaft of about 2 m to 3 m dia may be first excavated from top to bottom and then the shaft may be widened throwing the muck down the pilot shaft which may be removed from the bottom access tunnel. Alternatively, the vertical shaft may be excavated by providing burn type pattern of long holes drilled throughout the entire depth of the shaft by wagon drills. After the drilling, these holes may be blasted by charging them from top and blasting progressively upwards.

9.9 Excavation of Inclined Tunnels

Inclined shafts or tunnels may be sunk from top to bottom or if access is available, from bottom to top or from both top and bottom as is being done for the Chukha inclined pressure shafts. If the depth of shaft is more than 300 m an intermediate adit may be provided for opening additional working faces. If seepage water is expected in the strata, adequate arrangements should be made to divert the water away from the inclined tunnel.

9.9.1 *Excavation of Inclined Shafts from Bottom Upwards*

In this method, the inclined tunnel is raised from the bottom and as such the muck after taking a blast rolls down by gravity. This method, therefore, needs special arrangements to be made for the protection of water mains, ventilation pipes etc. This can be easily accomplished by dividing the shaft into two parts. The services are kept in the upper half while the muck rolls down the lower half.

The method used for excavating the inclined pressure shafts of Chukha and some other projects consists of installation of an Alimak Raise Climber chamber in a horizontal tunnel located at the lower end of the inclined pressure shaft as shown in Figure 9.8. The Alimak consists of an enclosed chamber which can be hoisted up the slope through a telescopic type extending arm to an angle of upto 55° from the horizontal. The enclosed chamber can house workers who operate the drills while sitting in the chamber. After drilling and loading the holes, the Alimak brings the chamber out of the inclined shaft. It is taken away into the horizontal tunnel (refuge bay) at a safe distance and blasting is then carried out. The loosened muck rolls down the slope. After defuming the Alimak again enters the inclined shaft and workers while sitting in the enclosed chamber scale out the loose muck and drill and load further holes.

This main advantages of this method are (i) the muck slides down by the force of gravity from where it is loaded in wagons and taken out of the tunnel; and (ii) if there is underground water no pumping is required. The method, however, suffers from a disadvantage that the ventilation is difficult as the gases after blasting have the tendency to go upward and hence take more time to be driven out.

9.9.2 *Excavation of Inclined Shafts from Top Downwards*

In this method, all operations are carried out from top downwards as in the case of sinking of vertical shafts. The sinking of inclined shafts is more difficult than sinking a vertical shaft. The general arrangement is shown in Figure 9.9.

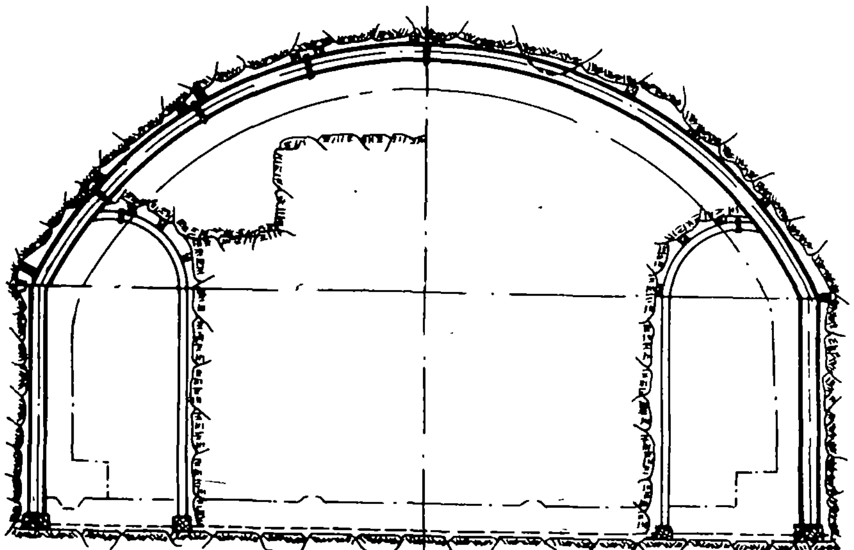
The main advantages of this method are :

(i) Going down the shaft and taking equipments is easier.
(ii) Regular trolly track can be laid at the bottom so that material and men can be easily hauled up after every operation.
(iii) Ventilation is much easier and quicker.
(iv) Services are not liable to be damaged by blasting and mucking.

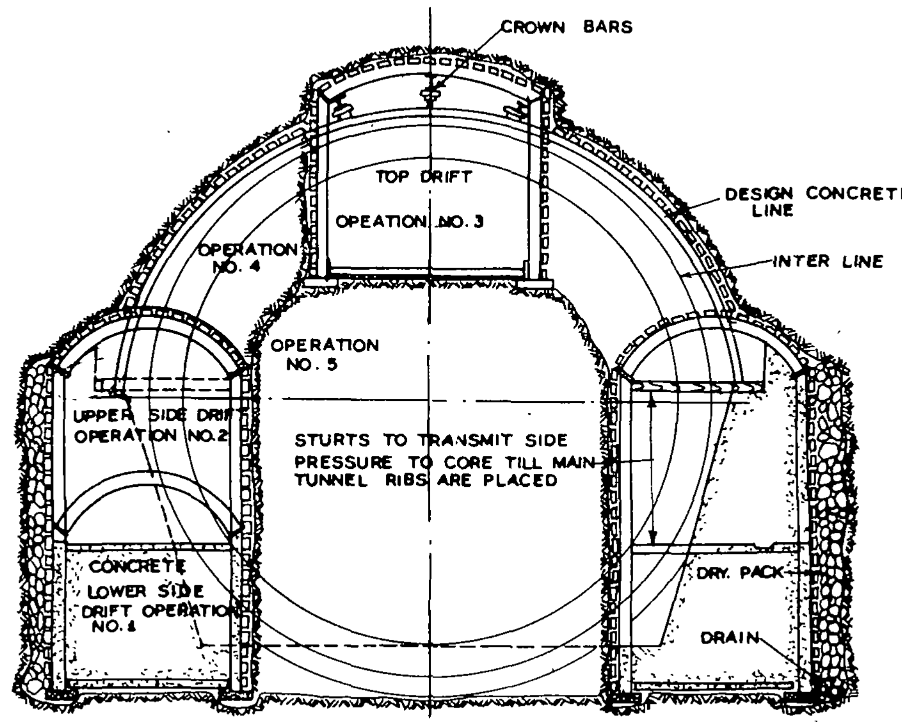
The disadvantages are :

(i) Since the muck is to be loaded manually, the progress is much slower.
(ii) If underground water is met with, pumping has to be done.

FIG= 9.1.



SIDE DRIFT METHOD



MULTIPLE DRIFT METHOD

FIG. 9·3

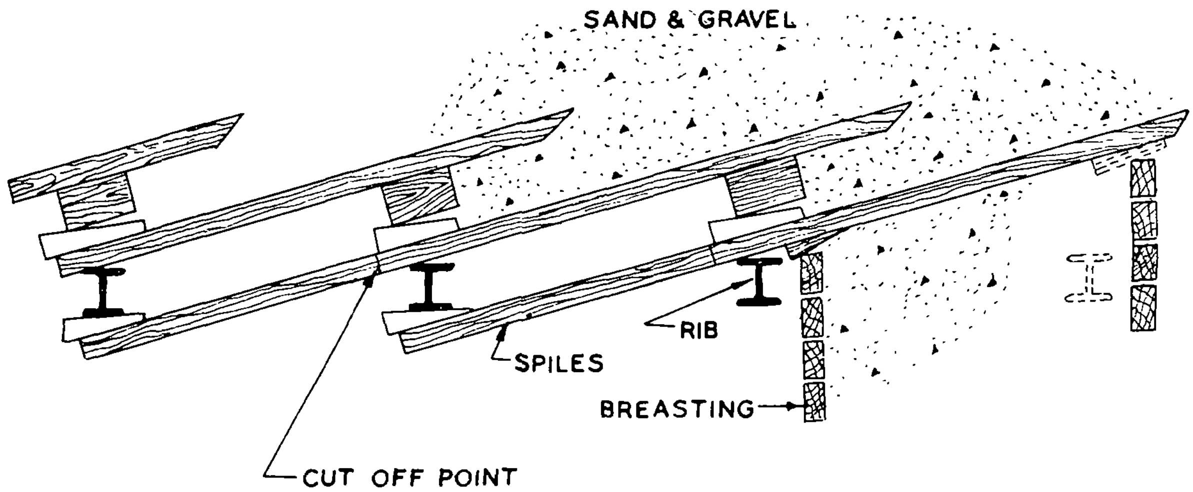


DIAGRAM OF FOREPOLING METHOD OF SUPPORTING RUNING GROUND

THE SPILES ARE DRIVEN AHEAD BEFORE MINING OUT. THE BREAST BOARDS ARE REMOVED, ONE BY ONE. THE SPACE AHEAD EXCAVETED AND THE BREASTING IN ITS NEW FORWARD POSITION. THUS ONLY A SMALL PORTION OF THE FACE IS OPEN AT ANY ONE TIME. THE TAILS OF THE SPILES ARE CUT OFF PRIOR TO CONCRETING.

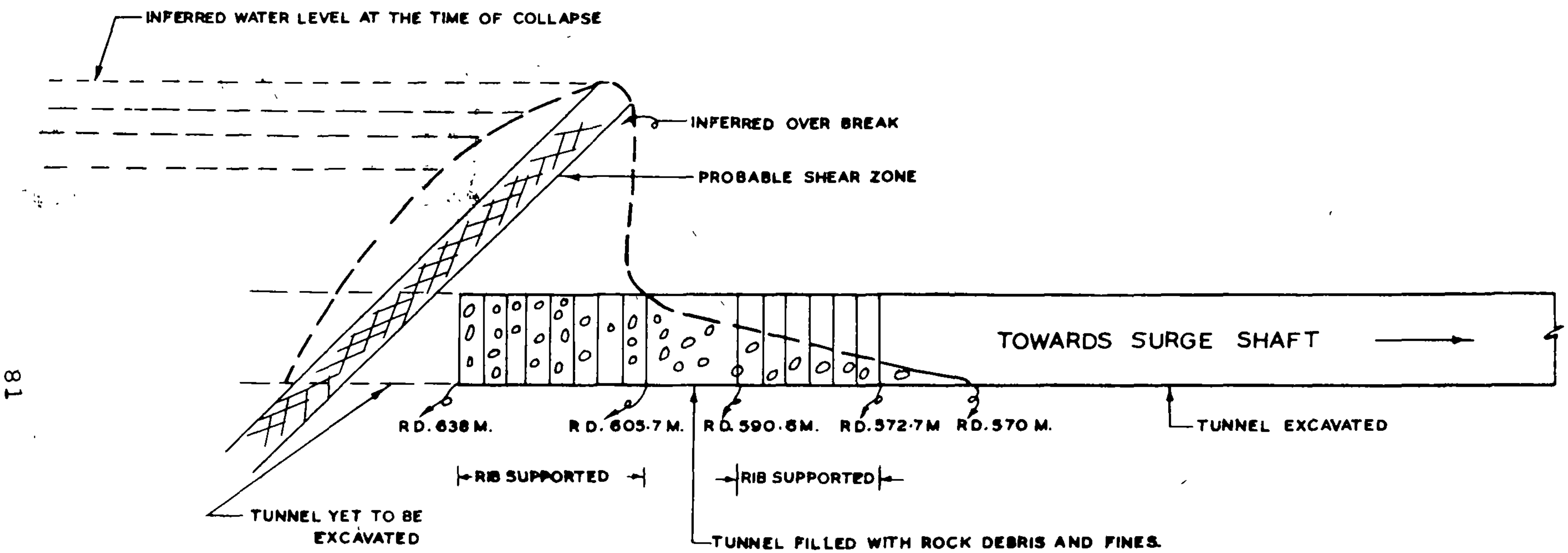


FIG. 9.4 HEAD RACE TUNNEL COLLAPSE CHUKHA H.E. PROJECT (BHUTAN)

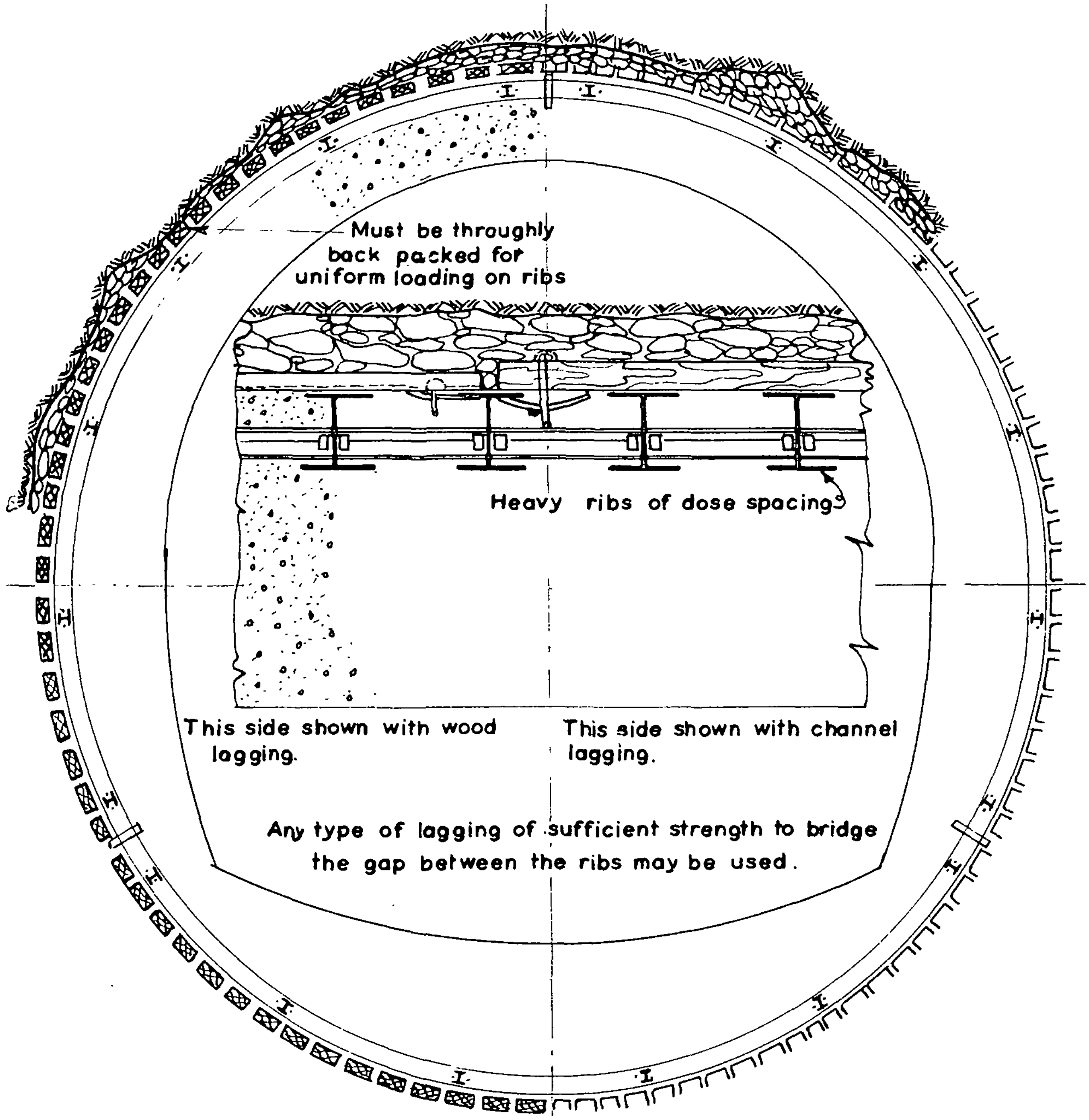


FIG.- 9·5

FULL CIRCLE RIBS CLOSELY SPACED AND HEAVILY LAGGED
FOR HEAVY LOAD ASSOCIATED WITH
SQUEEZING CONDITION

YIELDING LINING FOR SWELLING ROCK

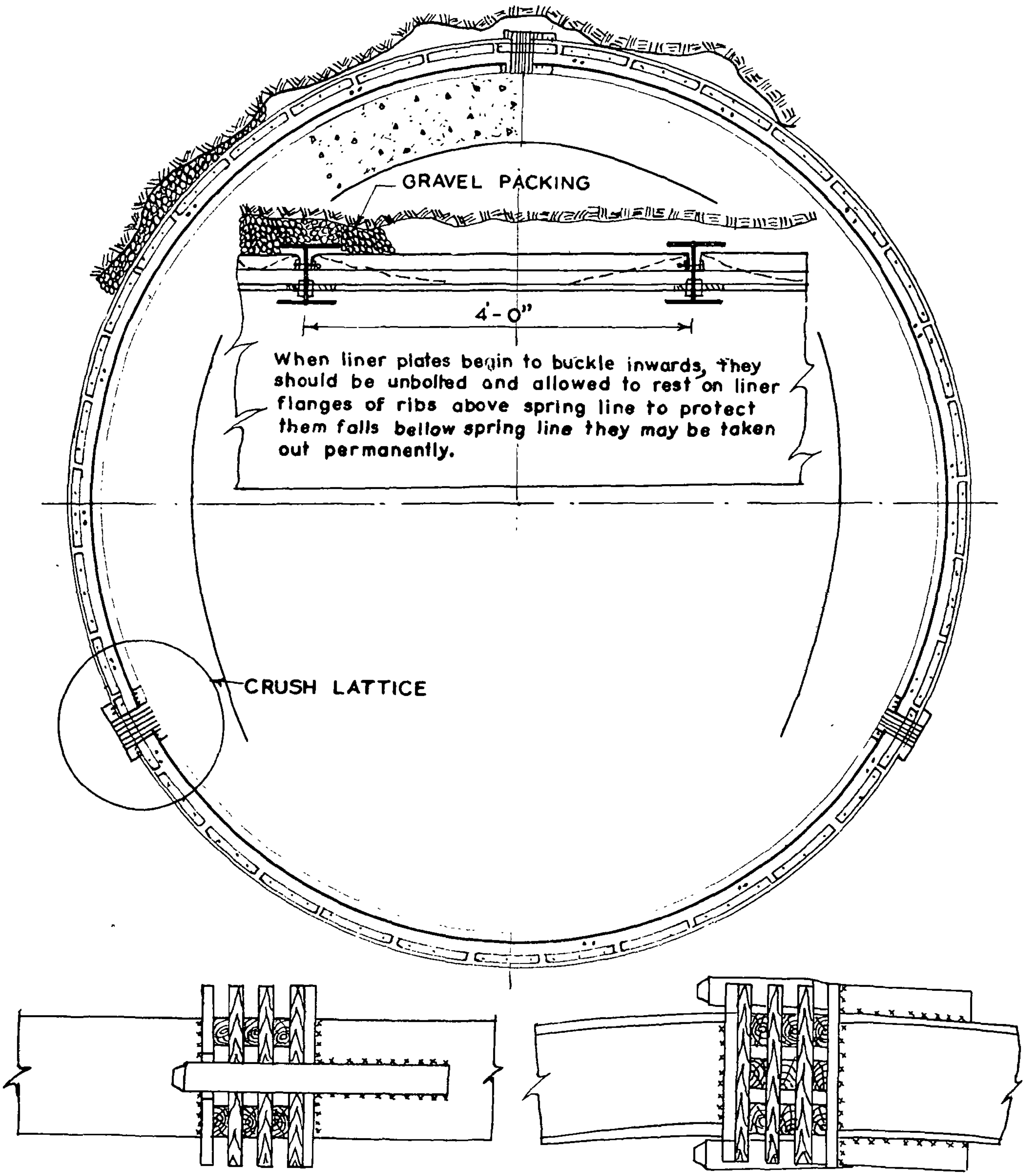


FIG.- 9·6

DETAIL OF CRUSH LATTICE

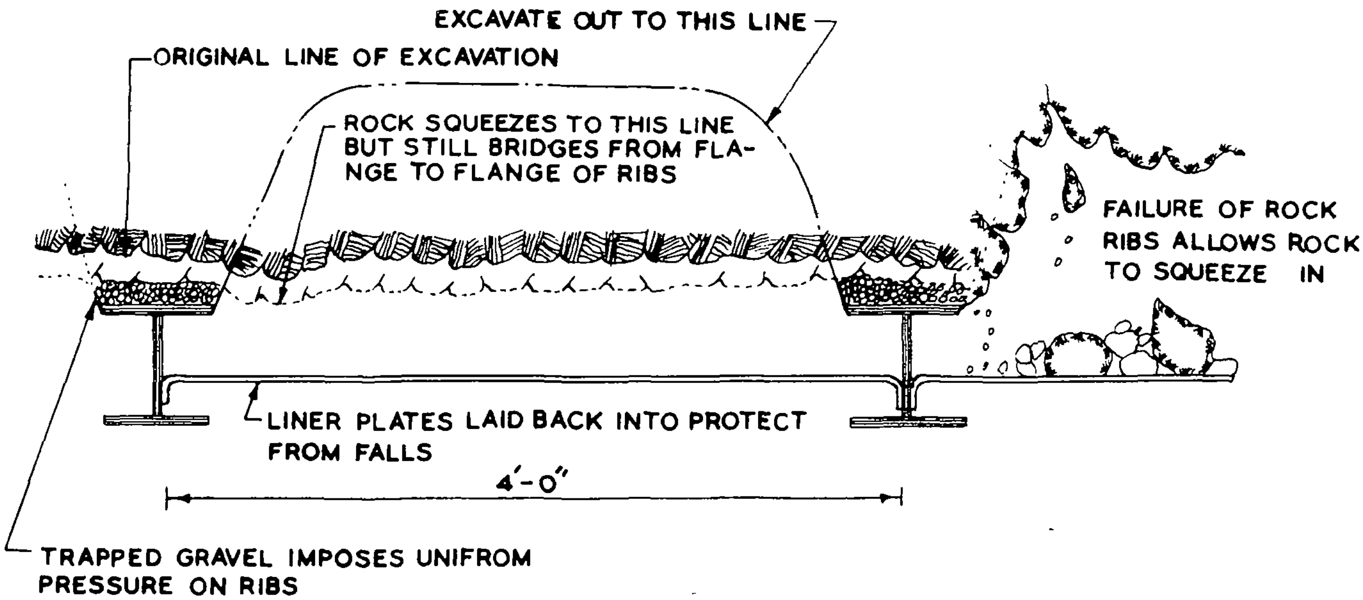


FIG. 9.7

OVER-MINING IN STIFF SWELLING GROUD TO INDUCE SOFTENING.

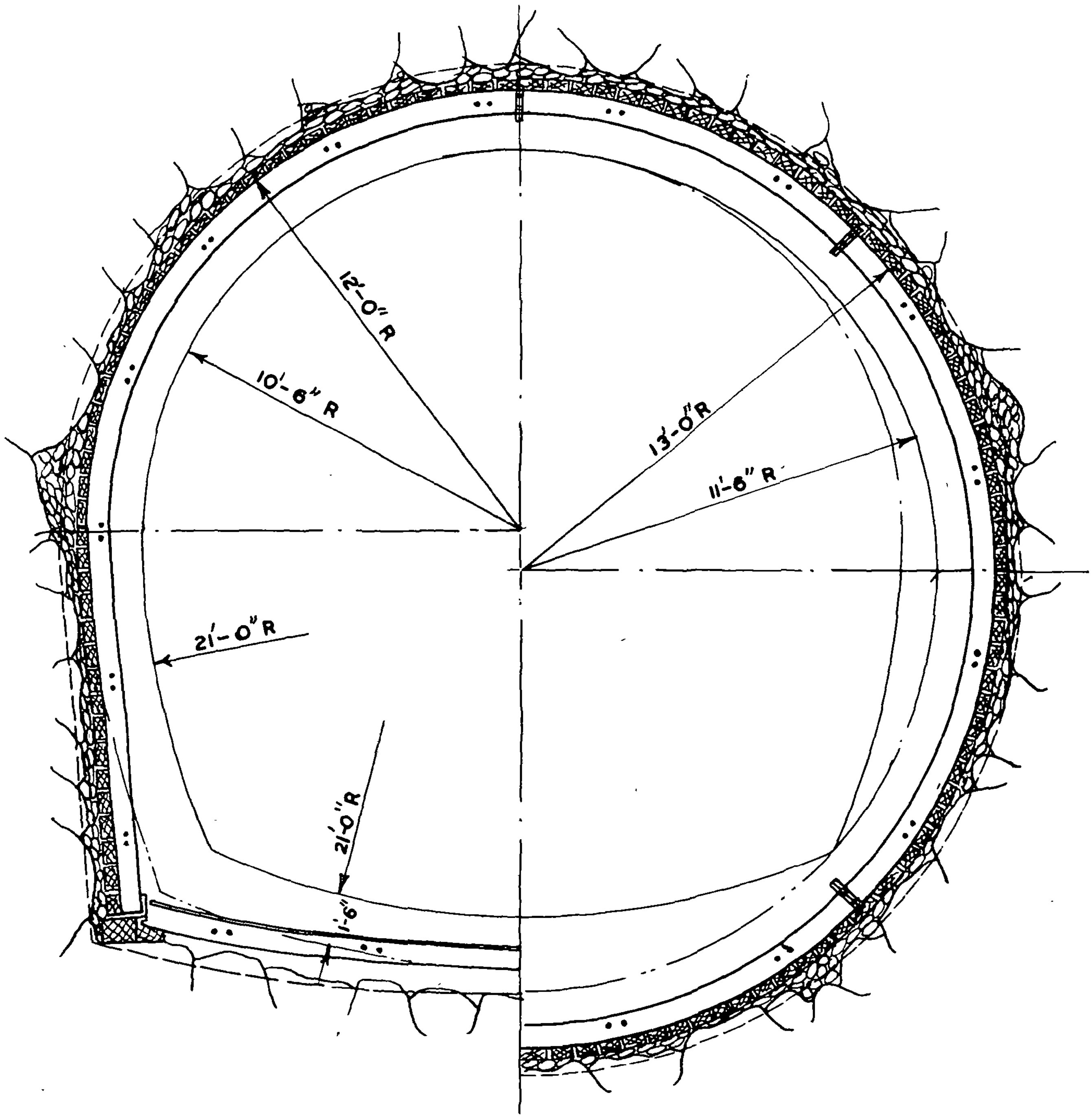


FIG.- 9.8

CONVERSION OF A HORSESHOE SHAPED FLOW TUNNEL TO A CIRCULAR SHAPE IN SQUEEZING GROUND

CHAPTER 10

RECENT ADVANCES IN TUNNELLING

10.0 General

In the past few years significant progress has been achieved in the support and lining systems for underground openings by way of rock reiuforcement rather than rock supporting. With the advancements made in the state of the art of rock mechanics, it is now considered as an alternative or atleast a partial alternative to reinforce the rock rather than support it with steel supports. This rock reinforcement is achieved by means of rock bolts and/or shotcrete in conjunction with other accessories like wire mesh etc.

The design of a support system or the assessment of rock reinforcement for an underground opening should be based on a qualitative basis of rock mass classification rather than a quantitative basis of its classification. While arbitrary classification systems such as whether the rock is good, fair or poor can be adopted, it is much more satisfactory to firstly specifically define the parameters to be included in rock assessment and secondly to specify how these various parameters are to be weighted and combined to give an overall classification of rock mass, i.e. a rating of the likely tunnelling conditions. Two such systems of rock mass classifications which are presently being followed all over the world are the C.S.I.R. classification system developed by Bieniawski and the N.G.I. classification system developed by Barton.

10.1 Rock Bolts

10.1.0 During construction of a tunnel, before the concrete is placed rock bolts are also used, either on their own or in conjunction with steel ribs. The early view was that the work of rock bolts was to "pin" or "nail" loose blocks or slabs of rock to the sounder rock behind them. Rock bolts have been so used for a long time, but their use for this purpose without understanding can be dangerous. The hazard is greatly increased where "spot bolting" is used, i.e. where bolts are used individually or with relatively large spacing between the bolts.

10.1.1 Rock bolt consists of a steel bar inserted in a hole drilled into the rock. The end away from the rock surface has a device which permits it to be firmly anchored in the hole. The projecting end is fitted with a plate which bears against the rock surface. The bolt is placed in tension between the anchor and the plate, thereby exerting a compressive force on the rock between the ends of the bolt.

10.1.2 *Types of Rock Bolts and their Effects*

Although there are many varieties of rock bolts in use, all have in common the following elements ;a bar or shank, an anchoring device at one end of the bar (the anchorage or "A" end), and a tensioning device at the other end of the bar (the tensioning or "*T*" end).

The rock bolts in common use are shown in Figure 10.1 and are classified in three groups by the type of anchorage used :

(a) Dive-set or Slot and Wedge type,
(b) Torque-set or Expansion type, and
(c) Grouted type.

The most commonly used bolts are the expansion anchorage type. However, the slot and wedge type can also be used except in soft rock and can be easily and quickly manufactured at site. Epoxy type resins for anchoring the bolts are also being used-particularly in water bearing strata. The use of an explosive charge in a hollow chamber at the anchorage end of the bolt to expand the chamber and provide an adequate bond between the bolt and the sides of the drill hole has also been suggested by the U.S. Bureau of Mines.

The rock bolts if properly installed have the following effects :

(i) A zone of compression is created in the rock around the excavation by the tension in the bolts.
(ii) The compressed material between the ends of the bolts tends to expand laterally. At this tendency is restrained by the rock outside the bolted areas, compressive forces are induced at right angles to the direction of the rock bolts.
(iii) The deformation of the surface of the excavation is restrained i.e. the rock at and near the surface is prevented from moving in towards the cavity, particularly if the bolts are placed very soon after blasting. In other words, the interaction of the rock bolts when installed in an appropriate pattern prevents the dilation of the surface of excavation which otherwise would take place as a result of the relaxation of the original stress and strain around the opening.

These effects imply that in the vicinity of free surface, the bolts form a diaphragm of material which behaves as a structural member. The thickness of this diaphragm is somewhat less than the length of the rock bolt and it has properties which can be assessed and behaviour which can be designed for.

10.1.3 *Strength of a Rock Bolt*

The strength of a rock bolt is determined by its anchorage. If the required tensile load in the bolt cannot be maintained because of continuing anchorage slip then the bolt is considered to have "failed".

10.1.4 *Grouted Rock Bolts*

For permanent rock structures such as dams, hydropower works, road cuttings etc. the bolts used for rock reinforcement must have a life span comparable with that of the other permanent parts of the work. Over a period of time, the ordinary rock bolt, anchored at one end in a hole in the rock, and tensioned with a nut and bearing plate at the other end, is subject to various factors that can reduce its effectiveness. These aggressive factors include ground water, slip of anchorage either because of blasting operations and/or by dynamic forces rock movements etc.

The above difficulties can be overcome by completely filling the bolt hole with cement grout, epoxy resin or other equally competent material thus providing not only corrosion protection, but also ensuring that the bond between the shank, the grout and the rock remains effective and the tension in the bolt is not lost. Typical grouted bolts are shown in Figure 10.2. Grouted bolts also offer shear resistance to lateral movements and prevent the shank being "pinched". Hence for all permanent works, the rock bolts must be grouted.

10.1.5 *Stresses Caused by Rock Bolts*

The compressive stress in the rock between ends of an isolated rock bolt is quite localised and the bolt has only a small effect on the rock more than one bolt length from it. For this reason bolts used as rock reinforcement should be installed on a set pattern and not individually. Investigations carried out with models and photoelastic analysis have shown that in order to obtain interaction between adjacent bolts the ratio of bolt length to spacing should preferably be not less than 2. The stress distribution in rock resulting from a systematic bolt pattern is shown in Figure 10.3 which shows typical trajectories for the principal stresses and maximum shear stresses caused by the bolting. From these figures it will be noticed that near the surface between two successive bolts, there is a zone of tensile stress. It is in this area that fallout will occur in fractured rock and start a revelling process which, if not checked, could lead to general collapse. The location of the boundary between this tension zone and the compression zone can be approximated by taking a surface at 45° to the bolt axis, as shown in Figure 10.4. Fallout from these tension zones can be effectively prevented by using wire mesh singly or in conjunction with shotcrete against the surface of the rock.

With fallout prevented the zone of compression caused by the rock bolt tension is approximately shown in Figure 10.4. This zone created by the bolts acts as a structural element and is effective in stabilizing rock excavations.

10.1.6 *Installation of Rock Bolts*

It is very desirable that rock bolts be installed ns soon as possible after the new surface is created and as close to the face as practicable. This not only greatly reduced the rock deformations and provides a more effective reinforced rock member, but also greatly increases the safety of workmen.

The installation of rock bolts is carried out in the following steps: drill the hole; place bolt and seat anchorage, install bearing plate; place bevel washers and torque nut; and tension the bolt. Grouting can be done immediately or later.

For efficient use of the bolts it is desirable to have the tension in the bolt as high as possible but not greater than the yield strength of the shank. A typical assembly at the tension (T) end of the bolt using a torque nut for tensioning the bolt is shown in Figure 10.5. In this method a nut is placed on threaded T-end of the bolt, together with necessary washers and tightened to a predetermined torque with a suitable torque wrench thus giving the required tension in the bolt shank.

In practice the bearing surface of the rock at a bolt is rarely perpendicular to the axis of the bolt. Also, the surface of the rock is never smooth and steel bearing plates are used to bridge the irregularities in the rock face and provide a firm bearing for the taper or bevel washer, which ensure that the nut has an even bearing. In addition, a hard washer immediately under the nut ensures less friction and more uniform conditions.

As shown in Figure 10.5 the bearing on the rock is essentially a "point" loading on several asperities of the rock surface. These crush until the area of contact with the bearing plate can support the tension load in the bolt. With the vibrations caused by nearby blasting, the crushed rock material at the "point" bearings tend to come loose, and at time spalling of the rock under the plate occurs. In these circumstances the bolt must be retensioned by tightening the nut. Hence, a mere installation of the bolts is not enough and a constant watch and routine testing and checking is a "must".

10.1.7 *Emperical Relationships for Rock Bolt Design:*

Table on page 89 gives the recommended empirical rules for the adoption of rock bolts.

10.2 Shotcrete in Rock Tunnelling

Shotcrete in tunnelling practice has been defined by the American Concrete Institute as pneumatically applied concrete. It is being vastly used in Europe, U.S.A., Scandenavia, Japan and Hong Kong in mines and tunnels to protect and support zones of fractured, crushed, disintegrated or spalling rock, and to preserve and prevent further deterioration caused by the action of water or the atmosphere or time. Shotcrete can be very effectively and safely used in situations where conventionally placed concrete or other material is impossible to be used. Shotcrete has been found to be very effective in areas where ground water is discharging from the freshly exposed surface and in filling up and effectively controlling wide seams. The recent works done by Rabcewicz, Pacher,

Sl. No.	Parameter	Empirical Rules
1.	Minimum Length	Greatest of : (a) Two times the bolt spacing. (b) Three times the width of critical and potentially instable rock blocks. (c) For elements above the spring-line: (i) Spans less than 20 feet : one—half of span. (ii) Spans 20 feet to 60 feet: Interpolate between 10 ft to 15 ft lengths. (iii) Spans from 60 feet to 100 feet: one-length of span. (d) For elements below the spring-line: (i) For openings less than 60 feet high use lengths as determined in (c) above. (ii) For openings greater than 60 feet high: one-fifth of the height.
2.	Maximum Spacing	Least of : (a) One-half the bolt length. (b) 1.5 times the width of critical and potentially unstable rock blocks. (c) 6 feet.

Note :—Spacing greater than 6 feet would make attachment of surface treatment such as chain-link fabric difficult.

Golser and others have proved that shotcrete in conjunction with wire mesh and rock bolts can be used to built up structural members which can permanently support even very large size tunnels under adverse geological conditions at very low costs.

10.2.1 *Design Parameters for Shotcrete*

Depending upon the rock conditions and the effective span to be bridged, the design of shotcrete shall incorporate the following parameters :

(*i*) Thickness of the layer,
(*ii*) Size and gradings of aggregates,
(*iii*) Amount of cement to be used,
(*iv*) Amount of sand and coarse aggregates to be used,
(*v*) Water cement ratio, and
(*vi*) Use of accelerators.

10.2.1.1 Thickness of Shotcrete Layer

Depending upon the quality of rock and the span to be bridged between two successive rock bolts, the thickness of shotcrete shall vary from 30 mm to 100 mm. For obtaining a 100 mm thickness, the shotcrete shall have to be sprayed in two layers each of 50 mm thickness. Experience has shown that for obtaining 100 mm thickness of shotcrete, a single layer would result in an enormous rebound of the shotcrete thus increasing the overall cost.

10.2.1.2 Size and Grading of Aggregates

All aggregates should be uniformly well-graded and shall conform to IS : 515 gradations. The gradation of the combined coarse and fine aggregate mixture shall be within the limits shown in the Table below.

Standard Sieve Size	Percentage Passing by Weight	
	Gradation No. 1	Gradation No. 2
25.4 mm (1")	100	100
19.05 mm (3/4")	90-100	—
12.7 mm (1/2")	75-95	100
9.55 mm (3/8")	65-88	95-100
No. 4	48-74	72-85
No. 8	34-57	52-73
No. 16	22-44	36-55
No. 30	12-31	20-38
No. 50	5-20	7-20
No. 100	2-10	2-12
No. 200	0-5	0-5

10.2.1.3 Amount and Type of Cement Used

The amount of cement used in shotcrete per cubic yard of dry mix is normally 650 to 660 lbs (400 kg per cu m). Since high early strength is a pre-requisite in shotcrete, only best quality Portland cement should be used. The use of Pozzolana cement shall be strictly prohibited.

10.2.1.4 Amount of Coarse Aggregates

According to the recommendations of ACI, the amount of sand and aggregates per cubic yard of dry mix is normally 3250 to 3300 lbs. The sand and aggregate shall be uniformally graded as stated in para 10.2.1.2.

10.2.1.5 Water Cement Ratio

Since water is mixed just before application and since the shotcrete has a high degree of compaction due to pneumatic application, the water-cement ratio is kept low. It varies from 35 to 50—increasing with decreasing maximum size aggregate.

10.2.1.6 Additives Used as Accelerators

Since the main purpose of shotcrete is to prevent and arrest further loosening up of the rock mass it is essential that high strength is attained as early as possible. This is achieved by adding a calculated quantity of accelerators to the cement and aggregates.

The chemical composition of a given type of cement may vary somewhat from factory to factory. As a consequence of this, a given accelerator may be incompatible with a particular cement, resulting in a drastic loss in quality of the shotcrete. It is, therefore, essential that the compatibility of cement and accelerator be tested in advance.

Apart from imparting high early strength, the accelerator will prevent sagging and sloughing of the shotcrete during application thus reducing the rebound and increasing the plasticity of mix which allows shotcrete to deform without cracking.

The accelerators could be either in powder form or in fluid form. Powder accelerators are added in an amount of 3 to 6 percent of cement weight (i.e., 20 to 40 lb. accelerate per cubic yard of mix). Fluid accelerators in volume ratio to water range from 1 : 20 (very low) to 1 : 1 (very high).

It may be noted here that the addition of accelerators results in a loss of strength of the final set shotcrete—it may be 300 kg/cm^2 only with accelerators as compared to 350 kg/cm^2 without accelerators. The 28 days compressive strength of shotcrete should not be less than 350 kg/cm^2.

10.2.2 *Application of Shotcrete to a Rock Surface*

Shotcrete mix can be brought to the nozzle for application either as a dry mix or as a wet mix. The more commonly used method is the dry mix method. Cement, sand and coarse aggregate are brought to an open dry mix hopper where powder accelerator is added. Via a pressure chamber, this mix is fed continuously through a hose to the nozzle under pneumatic pressure which is in the range of 5 to 6 kg/cm^2. The water is added at the nozzle at a pressure of about 4 to 4.5 kg/cm^2. The entire arrangement is shown in Figure 10.6.

The entire success of shotcrete is dependent upon the nozzleman. The amount of water added is controlled by him, and he, therefore, has a crucial influence on the quality of the final mix. Nozzleman's competence is equally important when it comes to actual application of shotcrete. Figure 10.7 shows the effect of correct distance and angle on the amount of rebound and the quality of shotcrete in place.

10.3 Tunnels Design Criteria using C.S.I.R. and N.G.I. Classification

10.3.1 *Bieniawski or C.S.I.R. Method*

The various steps involved in arriving at a suitable tunnel support system in this method are as below:

(*i*) Assess the various rock parameters.

(*ii*) Assess the rating of each of the above parameters using Table 10.1.

(*iii*) Once the basic, rock mass value has been arrived at, adjust for the joint orientation.

(*iv*) Arrive at the total rock mass rating and classify the rock mass accordingly.

(*v*) Having assessed the rock mass classification, determine the expected stand up time using Figure 10.8.

(*vi*) After determining the rock mass classification and the stand up time, arrive at the support requirement as given in Table 10.2.

10.3.1.1 Design Example

Excavation span of opening 14 metres.

Using Table 10.1

Sl. No.	Parameter	Value	Rating	
1.	Uniaxial Compressive strength	120 MPa (1200 kg/cm^2)	12	
2.	RQD	75% to 90%	17	
3	Spacing of joints	Set 1 : 50-300 mm	10	17 (average)
		Set 2 : 0.3-1 mm	20	
		Set 3 : 0.3-1 mm	20	
4.	Condition of joints	Slightly rough surface, Separation less than 1 mm Soft Joint Wall	12	
5.	Ground Water	Inflow for 10 m tunnel length less than 25 lit/minute Moist only	7	
		Basic Rock Mass Value	65	
6.	Orientation of joints	Set 1 : Horizontal	−10	−12 (Maximum o the three)
		Set 2 : Vertical Parallel to tunnel axis	−12	
		Set 3 : Vertical Perpendicular to tunnel axis	0	
	Total Rock Mass Rating		65−12=53 Fair Rocks (Class No. III)	

Using Figure 10.8

Stand up time = 100 hours (4 days).

Using Table 10.2, three different types of support systems are suggested as given below:

(*i*) *Mainly Rock Bolts*
Provide 3 m long grouted rock bolts spaced 1.0 m to 1.5 m centres. Wire mesh and 30 mm shotcrete may be provided in crown where required.

(*ii*) *Mainly Shotcrete*
Provide 100 mm shotcrete in crown and 50 mm in sides plus occasional wire mesh and rock bolts where required.

(*iii*) *Steel Ribs*
Light R.S.Js. may be provided at a spacing of 1.5 m to 2 m centres.

10.3.2 *Barton's (N.G.I.)*

Various steps involved in arriving at the support requirement using Barton's classification are as given below:

(*i*) Assess the various rock parameters and their corresponding values using Table 10.3.
Find out the value of rock quality index "Q" by using the formula:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

(*ii*) Work out the value of D_e called the equivalent dimension using the relationship

TABLE 10·1 – GEOMECHANIC CLASSIFICATION OF JOINTED ROCK MASSES

A-CLASSIFICATION PARAMETERS AND THEIR RATINGS:–

	PARAMETER		RANGE OF VALUES						
1.	STRENGTH OF INTACT ROCK MATERIAL	POINT-LOAD STRENGTH INDEX	>8MPa	4-8MPa	2-4MPa	1-2MPa	FOR THIS LOW RANGE UNIAXIAL COMPRESSIVE TEST IS PREFERRED		
		UNIAXIAL COMPRESSIVE STRENGTH	>200MPa	100-200MPa	50-100MPa	25-50MPa	10-25MPa	3-10MPa	1-3MPa
	RATING		15	12	7	4	2	1	
2	DRILL CORE QUALITY RQD		30% - 100%	75% - 90%	50% - 75%	25% - 50%		<25%	
	RATING		20	17	13	8		3	
3	SPACING OF JOINTS		> 3M	1-3M	0·3-1M	50-300MM		50MM	
	RATING		30	25	20	10		5	
4.	CONDITION OF JOINTS		VERY ROUGH SURFACE NOT CONTINUOUS NO SEPARATION HARD JOINT WALL ROCK	SLIGHTLY ROUGH SURFACES SEPARATION <1MM HARD JOINT WALL ROCK.	SLIGHTLY ROUGH SURFACES SEPARATION <1MM. SOFT JOINT WALL ROCK	SLICKENSIDED SURFACES OR GAUGE <5MM THICK OR JOINTS OPEN 1-5MM CONTINUOUS JOINTS		SOFT GAUGE >5MM. THICK OR JOINTS OPEN >5MM CONTINUOUS JOINTS	
	RATING		25	20	12	6		0	
		INFLOW PER 10MM TUNNEL LENGTH	NONE		<25 LITERS/MIN	·25-125LTS/MIN.		125 LITERS/MIN.	
5.	GROUND WATER	RATIO $\frac{\text{JOINT WATER PRESSURE}}{\text{MAJOR PRINCIPAL STRESS}}$	0		00-0 2	0 2-0 5		>0·5	
		GENERAL CONDITIONS	COMPLETELY DRY		MOIST ONLY INTERSTITIAL WATER	WATER UNDER MODERATE PRESSURE		SEVER WATER PROBLEMS	
	RATING		10		7	4		0	

B-RATING ADJUSTMENT FOR JOINT ORIENTATIONS:–

STRIKE AND DIP ORIENTATION OF JOINTS		VERY FAVOURABLE	FAVOURABLE	FAIR	UNFAVOURABLE		VERY UNFAVOURABLE
RATING TUNNEL							
		0	-2	-5	-10		-12

C-ROCK MASS CLASSES DETERMINED FOR TOTAL RATINGS:–

RATING		100-81	80-61	60-41	40-21		<20
CLASS NO		I	II	III	IV		V
DESCRIPTION		VERY GOOD ROCK	GOOD ROCK	FAIR ROCK	POOR ROCK		VERY POOR ROCK

$$D_e = \frac{\text{Excavation span, diameter or height}}{\text{Excavation support ratio (E.S.R.)}}$$

The value of E.S.R can be obtained from Table 10.4.

TABLE 10.2

Guide for Selection of Primary Support in Tunnels at Shallow Depth

Size: 5 m to 15 m Construction by Drilling and Blasting

Rock mass class	Alternative Support System: Mainly Rockbolts (20 mm dia. length ½ tunnel width Rasin bonded)	Mainly Shotcrete	Mainly Steel Ribs
I	Generally no support is required		
II	Rockbolts spaced 1.5 to 2.0 m plus occasional wire mesh in crown	Shotcrete 50 mm in crown	Uneconomic
III	Rockbolts spaced 1.0 to 1.5 m plus wire mesh and 30 mm shotcrete in crown where required	Shotcrete 100 mm in crown and 50 mm in sides plus occasional wire mesh and rockbolts where required	Light sets spaced 1.5 to 2 m
IV	Rockbolts spaced 0.5 to 1.0 m plus wire mesh and 30/50 mm shortcrete in crown and sides	Shotcrete 150 mm in crown and 100 mm in sides plus wire mesh and rockbolts 3 m long spaced 1.5 m	Medium sets spaced 0.7 to 1.5 m plus 50 mm shotcrete in crown and sides
V	Not recommended	Shotcrete 200 mm in crown and 150 mm in sides plus wire mesh, rockbolts and light steel sets, seal face. Close invert.	Heavy sets spaced 0.7 m with lagging. Shotcrete 80 mm thick to be applied immediately after blasting.

TABLE 10.3

Classification of individual parameters used in the NGI tunnelling Quality Index

Description	Value	Notes
1	2	3
1. ROCK QUALITY DESIGNATION	(RQD)	
A. Very poor	0-25	1. Where RQD is reported or measured as 10 (including 0), a nominal value of 10 is used to evaluate Q.
B. Poor	25-50	
C. Fair	50-75	
D. Good	75-90	2. RQD intervals of 5, i.e., 100, 95, 90, etc. are sufficiently accurate.
E. Excellent	90-100	
2. JOINT STRUCTURE NUMBER	(J_n)	
A. Massive, no or few joints	0.5-1.0	
B. One joint set	2	
C. One joint set plus random	3	
D. Two joint sets	4	
E. Two joint sets plus random	8	
F. Three joint sets	9	1. For intersections use ($3.0 \times J_n$)
G. Three joint sets plus random	12	2. For portals use ($2.0 \times J_n$)
H. Four or more joint sets, random, heavily jointed "sugar cume", etc.	5	
J. Crushed rock, earth like	20	
3. JOINT ROUGHNESS NUMBER	(J_r)	
a. Rock wall contact, and		
b. Rock wall contact before 10 cms shear.		
A. Discontinuous joints	4	
B. Rough or irregular, undulating	3	
C. Smooth, undulating	2	
D. Slickensides, undulating	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m.
E. Rough or irregular, planner	1.5	2. $J_r = 0.5$ can be used for planner, slickensided joints having lineations, provided the liners one are favourably oriented
F. Smooth, planner	1.0	
G. Slickensided, planner	0.5	
c. No rock wall contact when sheared		
H. Zone containing clay minerals thick enough to prevent rock wall contact	1.0 (Nominal)	
J. Sandy, gravelly or crushed zone thick enough to prevent rock wall contact	1.0 (Nominal)	
4. JOINT ALTERATION NUMBER	(J_a ϕ_r (approx))	
a. Rock wall contact		
A. Tightly healed, hard, non-softening, impermeable filling	0.75	1. Value of ϕ_r the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alternation products, if present
B. Unaltered joint walls, surface staining only	1.0 (25°-35°)	
C. Slightly altered joint walls, non-softening mineral coatings,		

1	2	3
sandy particles, clay-free		
C. Disintegrated rock, etc.	2.0 (25°-30°)	
D. Silty, or sandy-clay coatings, small clay-fraction (Non-softening)	3.0 (20°-25°)	
E. Softening or low friction clay mineral coatings, i.e., kaolinite. Also chlorite, tame, gypsum and graphite etc. and small quantities of swelling clays. (Discontinuous coatings, 1-2 m or less in thickness)	4.0 (8°-16°)	
b. Rock wall contact before 10 cm shear.		
F. Sandy, particles, clay free disntegrated rock etc.	4.0 (25°-30°)	
G. Strongly over-consolidated softening. clay mineral fillings, (continuous 5 mm thick)	6.0 (16°-24°)	
H. Medium or low over-consolidation, softening, clay mineral fillings, (continuous 5 mm thick)	0.8 (12°-16°)	
J. Swelling clay fillings, i.e., montmorillonite, (continuous 5 mm thick). Values of J_a depend on percoat of swelling clay-size particles, and access to water.	8.0-12.0 (6°-12°)	
c. No rock wall contact when sheared.		
K. Zones or bands of disintegrated	6.0	
L. Or crushed rock and clay (see G, H and J for clay conditions)	8.0	
M.	8.0-12.0 (6°-24°)	
N. Zones or bands of silty or sandy clay, small clay fraction, (non-softening)	5.0	
O. Thick, continuous zones or		
P. Bands of clay (see G.H.	10.0-13.0 (6°-24°)	
R. and J for clay conditions)	13.0.-20.0	
5. JOINT WATER REDUCTION FACTOR	J_w	approx. water pressure (kg/cm^2)
A. Dry excavations or minor inflow, i.e., lit/min locally	1.0	1.0

1	2	3
B. Medium inflow or pressure, occasional outwash or joint fillings.	0.66 1.0-2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5 2.5-10.0	
D. Large inflow or high pressure, considerable outwash or fillings	0.33 2.5-10.0	1. Factors C to F are crude estimates. Increase J_w if drainage measures are installed
E. Exceptionally high inflow or pressure at blasting, decaying with time.	0.2-0.1 10	
F. Exceptionally high inflow or pressure continuing without decay.	0.1-0.05 10	
6. STRESS REDUCTION FACTOR Weakness zones intersecting or influencing excavation, which may cause loosening the rock mass when tunnel is excavated.		
A. Multiple occurrences of SRF weakness zones containing rock, very loose surrounding rock (any depth)	10.0	1. Reduce these values of SRF by 25-50% if the relevant shear zones only influence but do not intersect the excavation.
B. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth >50 m)	5.0	
C. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth <50 m)	2.5	
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)	7.5	
E. Single shear zones in competent rock (clay free), depth of excavation >50 m)	5.0	
F. Single shear zones in competent rock (clay free), (depth of excavation <50 m)	2.5	
G. Loose open joints heavily jointed or "sugar cube" (any depth)	5.0	
b. Competent rock, rock stress problems		
H. Low stress, near surface	2.5	
J. Medium stress	1.0	
K. High stress, very tight structure	0.5-2.0	
L. Mild rock burst (massive rock)	5-10	
M. Heavy rock burst (massive rock)	10-20	

1	2	3
c. Squeezing rock; plastic flow or in-competent rock under the influence of high rock pressures		
N. Mild squeezing rock pressure	5-10	
Q. Heavy squeezing rock pressure	10-20	
d. Swelling rock; chemical swelling activity depending on presence of water		
P. Mild swelling rock pressure	5-10	
R. Heavy swelling rock pressure	10-15	

TABLE 10.4

In order to relate their Tunnelling Quality Index Q to the behaviour and support requirements of an underground excavation, Barton, Lien and Lunde defined an additional quality which they call the equivalent dimension D_e of the excavation. This equivalent dimension is obtained by dividing the span, diameter or wall height of the excavation by a quantity called the excavation support ratio ESR.

$$\text{Hence } D_e = \frac{\text{Excavation span, diameter or height (m)}}{\text{Excavation Support Ratio}}$$

The excavation support ratio is related to the use for which the excavation is intended and the extent to which some degree of instability is acceptable. Suggested values for the ESR are as follows:

	Excavation Category	*ESR*
A.	Temporary mine openings	3-5
B.	Vertical shafts,	
	1. Circular section	2.5
	2. Rectangular/square section	2.0
C.	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks) pilot tunnels, drifts and headings for large excavations	1.6
D.	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels	1.3
E.	Power stations, major road and railway tunnels, civil defence chambers, portals, intersections	1.0
F.	Underground nuclear power stations, railway stations, sports and public facilities, factories	0.8

The ESR is roughly analogous to the inverse of the factor of safety used in the design of rock slopes.

(*iii*) Find out the roof support pressure using the following equations:

$$P_{roof} = \frac{2}{J_r} \times (Q)^{-1/3}$$

where, P_{roof} = Permanent roof support pressure
J_r = Joint roughness number
Q = Rock mass quality

(*iv*) Depending upon the value of rock mass quality and the equivalent dimension, Barton has given 38 categories of tunnel support requirement as given in Figure 10.9. The numbers in the trapezium are supported categories and these are presented in *Table 10.5*.

10.3.2.1 Design Example

Tunnel in question is 9.5 m excavated dia. head race tunnel of the proposed Nathpa Jhakri H.E. Project in Himachal Pradesh.

Sl. No.	Parameter	Description	Value (Using Table 12.3)
1.	Rock quality Designation	From core recovery RQD=70%	70
2.	Joint Structure (J_n)	Two joint set random	6
3.	Joint roughness number (J_r)	Smooth undulating	2
4.	Joint alternation number (J_a)	Rock wall contact slightly altered joint walls, non-softening mineral coatings	2
5.	Joint Water Reduction Factor (J_w)	Minor inflow locally	1
6.	Stress reduction factor (SRF)	Competent rock, rock-stress problems medium stress	1

$$\text{Rock quality index } Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

$$= \frac{70}{6} \times \frac{2}{2} \times \frac{1}{1}$$

$$= 11.7 \text{ Good Rock}$$

Using Table 10.4, value of E.S.R. = 1.6
Excavated dia. of tunnel = 9.5 m

$$\text{Equivalent dimension } D_e = \frac{9.5}{1.6} = 5.94$$

$$\text{Now, Roof support pressure } P_{roof} = \frac{2}{J_n} \times Q^{-1/3}$$

$$= \frac{2}{2} \times (11.7)^{-1/3}$$

$$= 0.44 \text{ kg/cm}^2$$

From Table 10.5 support category 13 is required, i.e., provide rock bolts 3 m long @ 1.5 m centres.

10.4 Design of Tunnel Supports Using New Austrian Tunnelling Method

10.4.1 *General*

The New Austrian Tunnelling Method (NATM) was developed by Rabcewicz and Pacher on the experiences gained during the tunnelling operations carried out in the Alps after the Second World War. By that time the use of shotcrete and rock bolts had become quite common in mines and tunnels. However, the shotcrete and rock bolts were being used at random without giving much thought to the well known principles of rock mechanics.

The actual "New" in this method, developed primarily by Rabcewicz and Pacher and later corroborated by Golser and Muller, is not the technique of construction but the approach with which the rock mass is treated. The planning of the works as well as their execution are based upon the knowledge (or a reasonable estimate) of the type and manner to which rock mass react to the different types of outside interference (tunnelling operation). This estimates of rock-mass behaviour is based upon the data made available by various types of instrumentation planned and conceived in NATM.

10.4.2 *Concept of NATM*

The NATM has the following concept:

"The ground (rock or soil) which is surrounding the excavation, will be activated to a load bearing ring, thus enabling the ground to become an important support member in itself".

10.4.3 *Elements of NATM*

The main elements of NATM are:

(a) A so called semi-rigid lining (shotcrete layer(s)).
(b) The reinforcement of rock or soil by systematic rock bolting, and
(c) Insitu observation and measurement of data.

10.4.4 *Basic Principles of NATM*

The NATM is based upon the following basic principles:

(a) Choice of proper tunnel shape,
(b) Avoidance of detrimental loosening,
(c) Careful excavation,
(d) Right sequence of working stages,
(e) Provision of a semi-rigid lining,
(f) Controlled deformations for stress relief, ring closure time, ring closure distance etc., and
(g) In-situ measurements.

10.4.5 *In-situ Measurements*

As mentioned earlier, the success of NATM depends upon accurate and effective instrumentation in section chosen under various geological conditions. Rabcewicz has stated that in-situ measurements on the full size cross-section in the course of construction are much more informative than those made in test drifts. By means of measurements on the full size cross-section, it is possible to recognise within a short time whether and to what extent the chosen type of construction is adequate to stabilize the cavity or how it should be modified to obtain the economically and structurally optimum results.

The measuring sections are placed at suitable distances where the construction starts. The behaviour of both the support and the surrounding rock, the amount of deformation and stresses are exactly watched by measurements. In cases where the deformations are too great and where rock stabilizing means have been under dimensioned possibly resulting in shear cracks, the measurement data obtained indicate well in time when and to what extent additional stabilizing means are to be applied.

The following set of instruments are generally used for in-situ measurements:

Sl. No.	Instrument	Purpose of Measurement
1.	Tape extensometer	Convergency closure measurements
2.	Bore hole extensometer (Single point or Multi point)	Movement of rock surrounding a cavity and relative movement of joints, fissures etc.
3.	Pressure Cell	Radial stress, tangential stress and contact pressure at the interface.
4.	Load Cell	Thrust in steel supports, anchor forces, horizontal and vertical loads.
5.	Inclinometer	Stability of underground structures supervision of slopes, slides movements, supervision of movement of of dam abutments.
6.	Piezometers	Water pressure in rock, lining or at interfaces.

TABLE 10.5

Support measures for rock masses of exceptional "Extremely Good, Very Good and Good quality" (Q Range: 1000-10)

Support Category	Q	Conditional Factors RQD/Jn	Jr/Jn	Span/ ESR(m)	P kg/cm² (Approx.)	Span ESR(m)	Type of Support	See Note No.
1.	1000-400	—	—	—	< 0.01	20-24	sb (utg)	
2.	1000-400	—	—	—	< 0.01	30-60	sb (utg)	
3.	1000-400	—	—	—	< 0.01	46-80	sb (utg)	
4.	1000-400	—	—	—	< 0.01	65-100	sb (utg)	
5.	400-100	—	—	—	0.05	12-30	sb (utg)	
6.	400-100	—	—	—	0.05	19-45	sb (utg)	
7.	400-100	—	—	—	0.05	30-65	sb (utg)	
8.	400-100	—	—	—	0.05	48-88	sb (utg)	
9.	100-40	⩾ 20 < 20	— —	— —	0.25	8.5-19	sb (utg) B (utg) 2.5-3 m	
10.	100-40	⩾ 30 < 30	— —	— —	0.25	14-30	B (utg) 2-3 m B (utg) 1.5-2 m +clm	
11.	100-40	⩾ 30	—	—	0.25	23-48	B (tg) 2-3 m B (tg) 1.5-2 m +clm	
12.	100-40	⩾ 30	—	—	0.25	40-72	B (tg) 2-3 m B (tg) 1.5-2 m +clm	
13.	40-10	⩾ 10 ⩾ 10 < 10 > 10	⩾ 1.5 < 1.5 ⩾ 1.5 < 1.5	— — — —	0.5 — — —	5-14 — — —	sb (utg) B (utg) 1.5-2 m B (utg) 1.5-2 m B (utg) 1.5-2 m +S 2-3 cm	I I I I
14.	40-10	⩾ 10 < 10 —	— — —	⩾ 15 ⩾ 15 < 15	0.5	9-23	B (tg) 1.5-2 m +clm B (tg) 1.5-2 m +S (mr) 5-10 cm B (utg) 1.5-2 m +clm	I, II I, II I, II
15.	40-10	> 10 ⩽ 10	— —	— —	0.5	15-40	B (tg) 1.5-2 m +clm B (tg) 1.5-2 m +S (mr) 5-10 cm	I, II, IV I, II, IV
16. See Note XII	40-10	> 15 ⩽ 15	— —	— —	0.5	30-65	B (tg) 1.5-2 m +clm B (tg) 1.5-2 m +S (mr) 10-15 cm	I, V, VI I, V, VI

TABLE 10.5 (*Contd.*)

Engineering classification of rock masses for the design of tunnel support
Support measure for rock masses of fair and poor quality

Support Category	Q	Conditional Factors RQD/Jn	Jr/Jn	Span/ ESR	P kg/cm² (approx.)	Span/ ESR(m)	Type of Support	See Note No.
17.	10—4	> 30	—	—	1.0	3.5-9	sb (utg)	I
		⩾ 10, ⩽ 30	—	—			B (utg) 1-1.5 m	I
		< 10	—	⩾ 6 m			B (utg) 1-1.5 m +S 2-3 cm	I
		< 10	—	< 6 m			S 2-3 cm	I
18.	10—4	> 5	—	⩾ 10 m	1.0	7-15	B (tg) 1-1.5 m +clm	I, III
		> 5	—	< 10 m			B (utg) 1-1.5 m +clm	I
		⩽ 5	—	⩾ 10 m			B (tg) 1-1.5 m +S 2-3 cm	I, III
		⩽ 5	—	< 10 m			B (utg) 1-1.5 m +S 2-3 cm	I
19.	10—4	—	—	⩾ 20 m	1.0	12-29	B (tg) 1-2 m +S (mr) 10-15 cm	I, II, IV
		—	—	< 20 m			B (tg) 1-1.5 m +S (mr) 5-10 cm	I, II
20.	10—4	—	—	⩾ 35 m	1.0	24-52	B (tg) 1-2 m +S (mr) 20-25 cm	I, V, VI
See Note XII		—	—	< 35 m			B (tg) 1-2 m +S (mr) 10-20 cm	I, II, IV
21.	4—1	⩾ 12.5	⩽ 0.75	—	1.5	2.1-6.5	B (utg) 1 m +S 2-3 cm	I
		< 12.5	⩽ 0.75	—			S 2.5-5 cm	I
			> 0.75	—			B (utg) 1 m	I
22.	4—1	> 10, < 30	> 1.0	—	1.5	4.5-11.5	B (utg) 1 m+clm	I
		⩽ 10	> 1.0	—			S 2.5-7.5 cm	I
		< 30	⩽ 1.0	—			B (utg) 1 m +S (mr) 2.5-5 cm	I
		⩾ 30	—	—			B (utg) 1 m	I
23.	4—1	—	⩾ 15 m		1.5	8-24	B (tg) 1-1.5 m +S (mr) 10-15 cm	I, II, IV, VII
			< 15 m				B (utg) 1-1.5 m +S (mr) 5-10 m	I
24.	4—1	—	⩾ 30 m		1.5	18-46	B (tg) 1-1.5 m +S (mr) 15-30 cm	I, V, VI
See Note XII			< 30 m				B (tg) 1-1.5 m +S (mr) 10-15 cm	I, II,IV

TABLE 10.5 (*Contd.*)

Support measure for rock masses of rock of very poor quality (Q Range: 1.0-0.1)

Support Category	Q	Conditional Factors RQD/Jn	Jr/Ja	Span/ESR (m)	P kg/cm² (Approx)	Span/ESR (m)	Type of Support	See Note No.
25.	1.0-0.4	> 10	> 0.5	—	2.25	1.5-4.2	B (utg) 1 m+mr or clm	I
		⩽ 10	> 0.5	—			B (utg) 1 m+S (mr) 5 cm	I
		—	⩽0.5	—			B (tg) 1 m+S (mr) 5 cm	I
26	1.0-0.4	—	—	—	2.25	3.2-7.5	B (tg) 1 m +S (mr) 5-7.5 cm	VII, X, XI
							B (utg) 1 m+S 2.5-5 cm	I, IX
27	1.0-0.4	—	—	⩾ 12 m	2.25	6-18	B (tg) 1 m + S (mr) 7.5-10 cm	I, IX
		—	—	< 12 m			B (utg) 1 m +S (mr) 5-7.5 cm	I, IX
		—	—	> 12 m			CCA 20-40 cm +B (tg) 1 m	VIII, X, XI
		—	—	< 12 m			S (mr) 10-20 cm +B (tg) 1 m	VIII, X, XI
28	1.8-0.4	—	—	⩾ 30 m	2.25	15-38	B (tg) 1 m +S (mr) 30-40 cm	I, IV, V, IX
				⩾ 20, < 30			B (tg) 1 m +S (mr) 20-30 cm	I, II, IV, IX
				< 20 m			B (tg) 1 m +S (mr) 15-20 cm	I, II, IX
		—	—	—			CCA (sr) 30-100 cm +B (tg) 1 m	IV, VIII, X, XI
29.	0.4-0.1	> 5	> 0.25	—	3.0	1.0-3.1	B (utg) 1 m+S 2-3 cm	—
		⩽ 5	> 0.25	—			B (utg) 1 m+S (mr) cm	—
		—	⩽ 0.25	—				
30	0.4-0.1	⩾ 5	—	—	3.0	2.2-6	B (tg) 1m+S 2.5-5 cm	IX
		< 5	—	—			S (mr) 5-7.5 cm	IX
		—	—	—			B (tg) 1 m +S (mr) 5-7.5 cm	VIII, X, XI
31.	0.4-0.1	> 4	—	—	3.0	4-14.5	B (tg) 1 m +S (mr) 5-12.5 cm	IX
		⩽ 4 ⩾ 1.5	—	—			S (mr) 7.5-25 cm	IX
		< 1.5	—	—			CCA 20-40 cm +B (tg) 1 m	IX, XI
		—	—	—			CCA (sr) 30-50 cm +B (tg) 1 m	VIII, X, XI
32	0.4-0.1	—	—	⩾ 20 m	3.0	11.34	B (tg) 1 m +S (mr) 40-60 cm	II, IV, IX, XI
		—	—	< 20 m			B (tg) 1 m +S (mr) 20-40 cm	III, IV, IX, XI
		—	—	—			CCA (sr) 40-120 cm +B (tg) 1 m	IV, VIII, X, XI

Key to Support Tables :

sb = Spot Bolting
B = Systematic Bolting
(utg) = Untensioned, Grouted
(tg) = Tensioned, (expanding shell type for competent rock masses, grouted post-tensioned in very poor quality rock masses, see note XI)
S = Shotcrete
(mr) = Mesh Reinforced
clm = Chain Link Mesh
CCA = Cast Concrete Arch
(Sr) = Steel Reinforced

Bolt spacings are given in metres (m) Shotcrete, or cast concrete arch thickness is given in centimetres (cm)

TABLE 10.5 (*Contd.*)

Support measures for rock masses of extremely "Poor and Exceptional Poor" quality (Q Range: 0.1-0.001)

Support Category	Q	Conditional Factors RQD/Jr	Jr/Jn	Span/ ESR (m)	P kg/cm² (Approx.)	Span/ ESR (m)	Type of Support	See Note No.
33.	0.1-0.01	⩾ 2	—	—	6	1.0-3.9	B (tg) 1 m +S (mr) 2.5-5 cm	IX
		< 2	—	—			S (mr) 5-10 cm	IX
		—	—	—			S (mr) 7.5-15 cm	VII, X
34.	0.1-0.01	⩾ 2	⩾ 0.25	—	6	2.0-11	B (tg) 1 m +S (mr) 5-7.5 cm	IX
							S (mr) 7.5-15 cm	IX
							S (mr) 15-25 cm	IX
							CCA (Sr) 20-60 cm +B (tg) 1 m	VIII, X, XI
35.	0.1-0.01	—	—	⩾ 15 m	6	6.5-28	B (tg) 1 m +S (mr) 30-100 cm	II, IX, XI
See Note XIII				⩾ 15 m			CCA (sr) 60-200 cm +B (tg) 1 m	VIII, X, XI, II
				⩾ 15 m			B (tg) 1 m +S (mr) 20-75 cm	IX, XI, III
				⩾ 15 m			CCA (sr) 40-150 cm +B (tg) 1 m	VIII, X, XI, III
36.	0.01-0.001	—	—	—	12	1.0-2.0	S (mr) 10-20 cm	IX
							S (mr) 10-20 cm +B (tg) 0.5-1.0 m	VIII, X, XI
37.	0.01-0.001	—	—	—	12	1.0-6.5	S (mr) 20-60 cm	IX
							S (mr) 20-60 cm +B (tg) 0.5-1.0 m	VIII, X, XI
38.	0.01-0.001	—	—	⩾ 10 m	12	40-20	CCA (sr) 100-300 cm	IX
See Note XIII		—	—	⩾ 10 m			CCA (sr) 100-300 cm +B (tg) 1 m	VIII, X, II, XI
		—	—	< 10 m			S (mr) 70-200 cm	IX
		—	—	< 10 m			S (mr) 70-200 cm +B (tg) 1 m	VIII, X, III, XI

Engineering classification of Rock Masses for the Design of Tunnel Support Supplementary Notes for Supporting Tables

I. For cases of heavy rock bursting or "popping tensioned" bolts with enlarged bearing plates often used, with spacing of about 1 m (occasionally down to 0.8 m) final support when "popping activity ceases".

II. Several bolt lengths often used in same excavation, i.e., 3, 5 and 7 m.

III. Several bolt lengths often used in same Excavation, i.e., 2, 3 and 4 m.

IV. Tensioned cable anchors often used to supplement bolt support pressure. Typical support spacing 2-4 m.

V. Several bolt length often used in some excavations, i.e., 6. 8 and 10 m.

VI. Tensioned cables anchors often used to supplement bolt support pressures. Typical spacing 4-6 m.

VII. Several of the older generation power stations in this category employ systematc or spot bolting with the areas of chain link mesh, and a free span concrete arch roof (25-40 cm) as permanent support.

VIII. Cases involving swelling, for instance montmorillonite clay (with access of water). Room for expansion behind the support is used in cases of heavy swelling, drainage measures are used where possible.

IX. Cases not involving swelling clay or squeezing rock.

X. Cases involving squeezing rock, heavy rigid support its generally used as permanent support.

XI. According to the author's experience in case of swelling or squeezing, the temporary support required before concrete or shotcrete arches are formed may consist of bolting (tensioned shall—expension type) if the value of RQD/Jr is sufficiently high (i.e. > 1.5) possibly combined with shotcrete. If the rock mass is very heavily jointed or crushed (i.e., RQD/Jr < 1.5) for example a "Sugar Cube Shear Zone in Quartzite), then the temporary support may consist of upto several applications of shotcrete. Systematic bolting (tensioned) may be added after. Casting the concrete (or shotcrete) arch to reduce the uneven loading on the concrete, but it it may not be effective when RQD/Jn < 1.5, or when a lot of clay is present, unless the bolts are grouted before tensioning. A sufficient length of anchored bolt might also be obtained using quick setting resin anchors in these extremely poor quality rock masses. Serious occurrences of swelling and/or squeezing rock may require that the concrete arches are then right up to the face, possibly using a shield temporary shuttering. Temporary support of the working face may also be required in these cases.

XII. For reasons of safety the multiple drift method will often be needed during excavation and supporting of roof arow. Categories 16, 20, 24, 26, 32, 35 (span) ESR > 15 m only.

XIII. Multiple drift method usually needed during excavation and support of arch, walls and floor in cases of heavy squeezing category 38 (span) ESR > 10 m only.

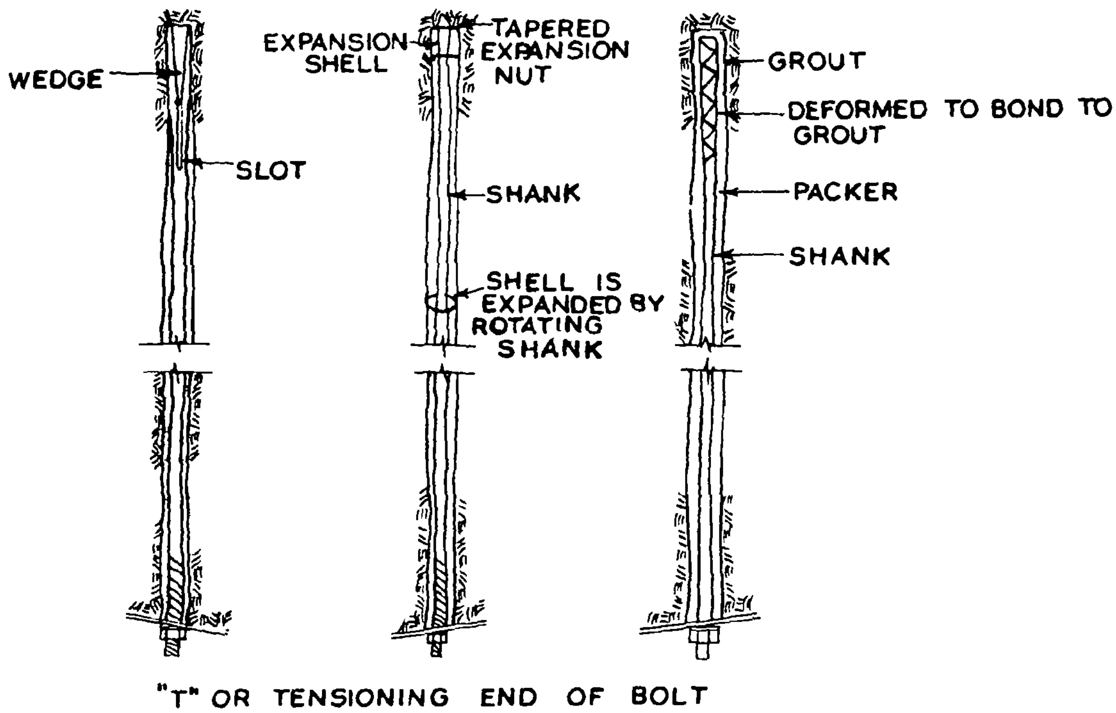
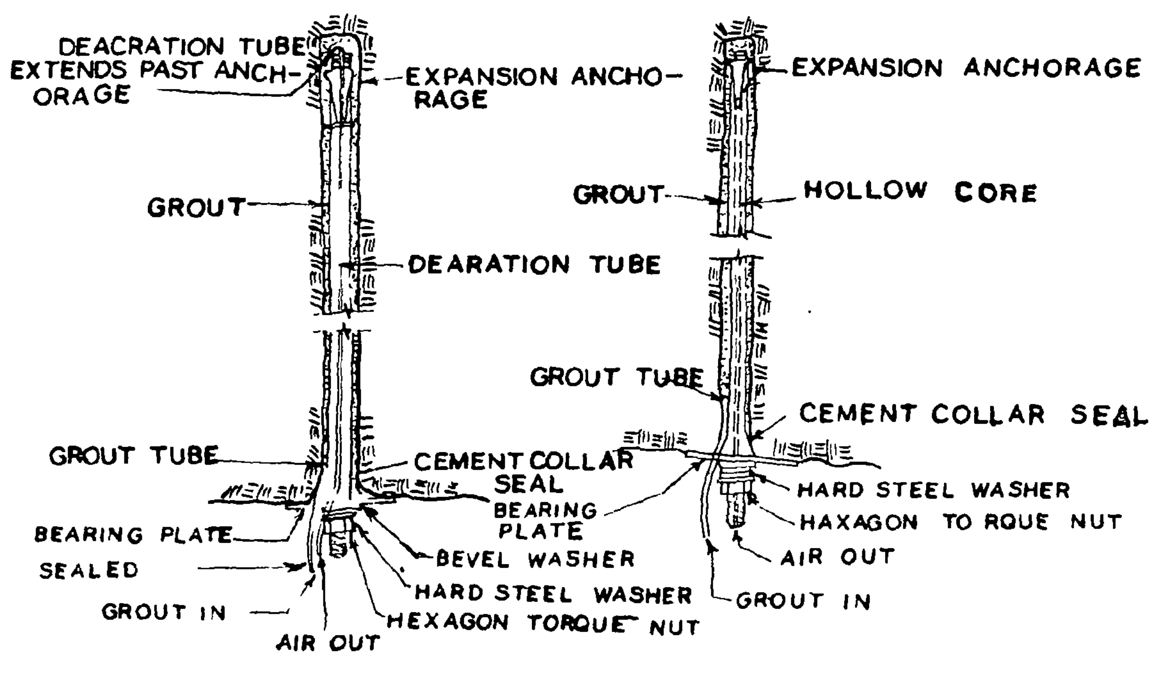


FIG.- 10·1 TYPES OF ROCK BOLTS



(a) GROUTED SOLID EXPANSION ANCHORAGE BOLT
(b) HOLLOW CORE GROUTED ROCK BOLT

FIG.- 10·2 GROUTED ROCK BOLTS

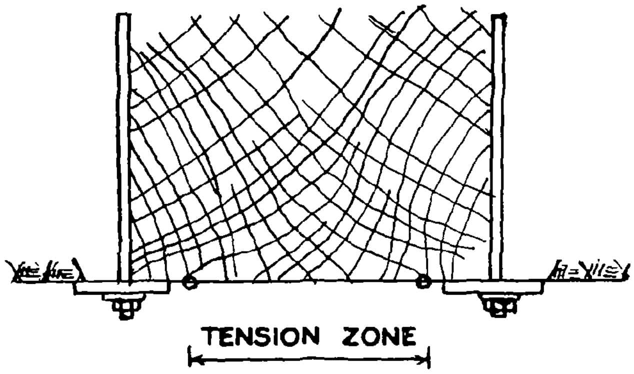


FIG.-10·3 STRESSES CAUSED BY BOLTS

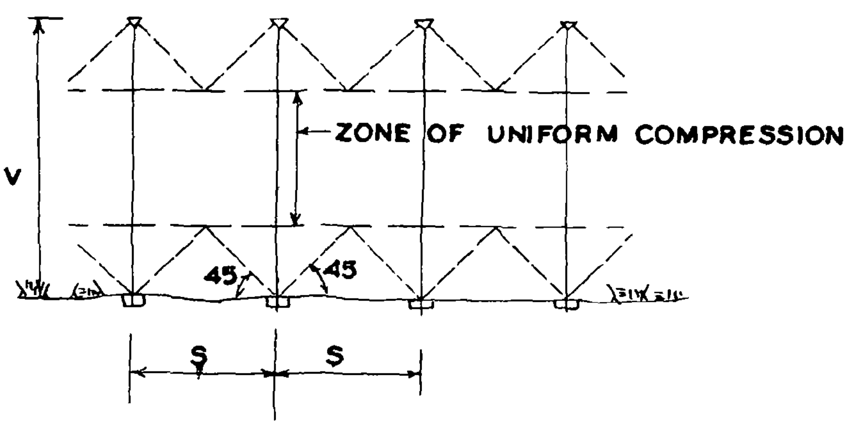


FIG.-10·4 STRUCTURAL MEMBER CONCEPT

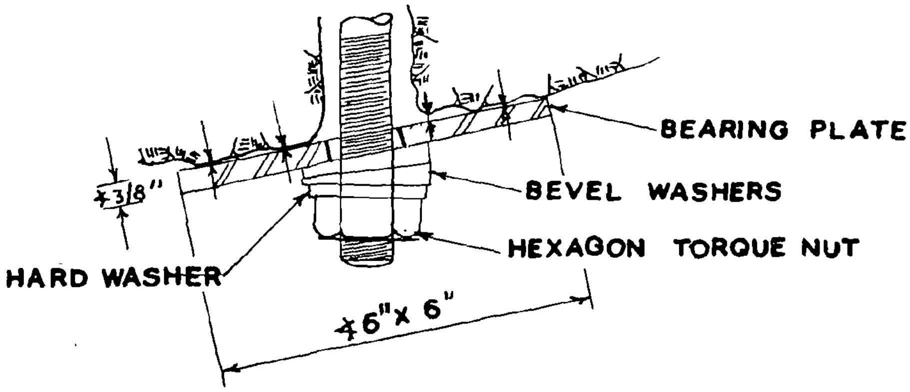


FIG.-10·5 BEARING PLATE ASSEMBLY

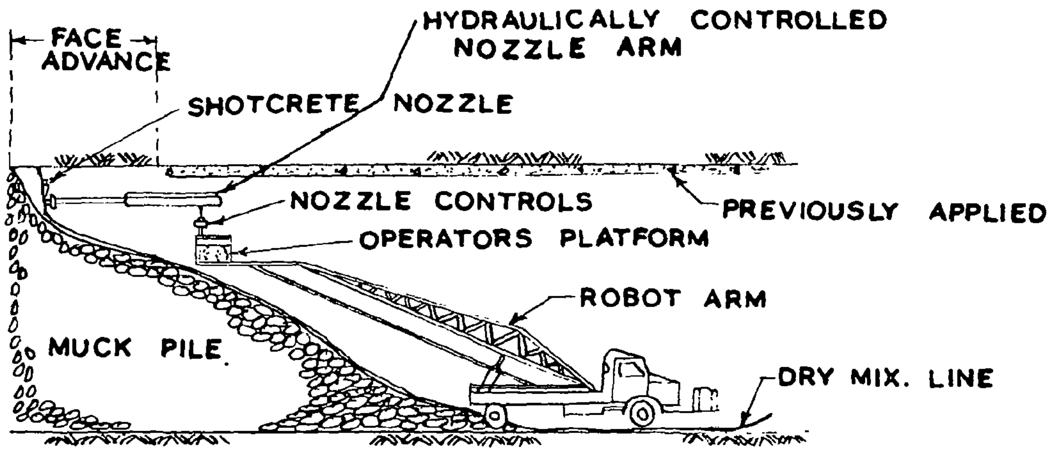


FIG.-10·6 THE SWEDISH ROBOT
(AFTER ALBERTS & BRANNFORS 1965)

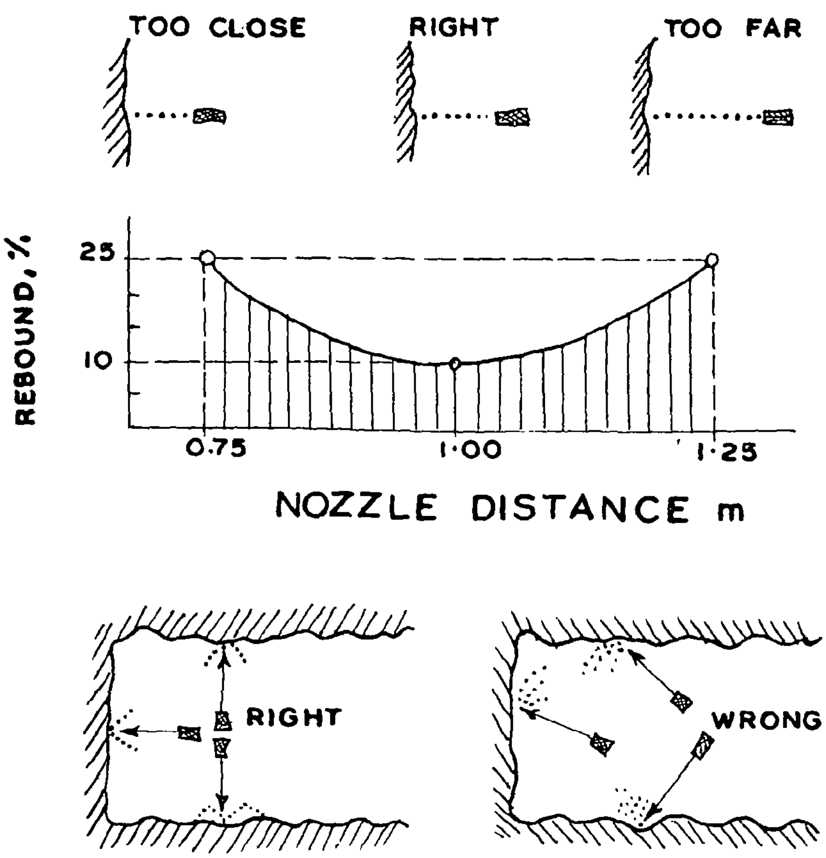
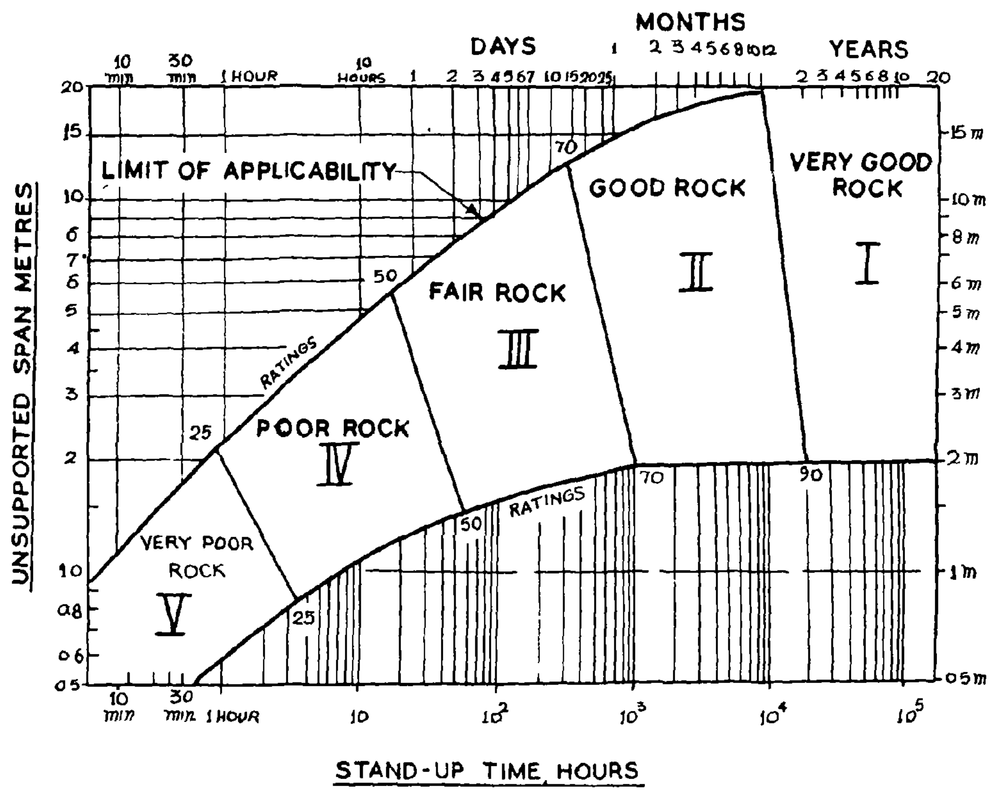


FIG.-10.7 INFLUENCE OF NOZZLE DISTANCE & ANGLE
(AFTER DROGSLER-1961)

FIG. 10.8



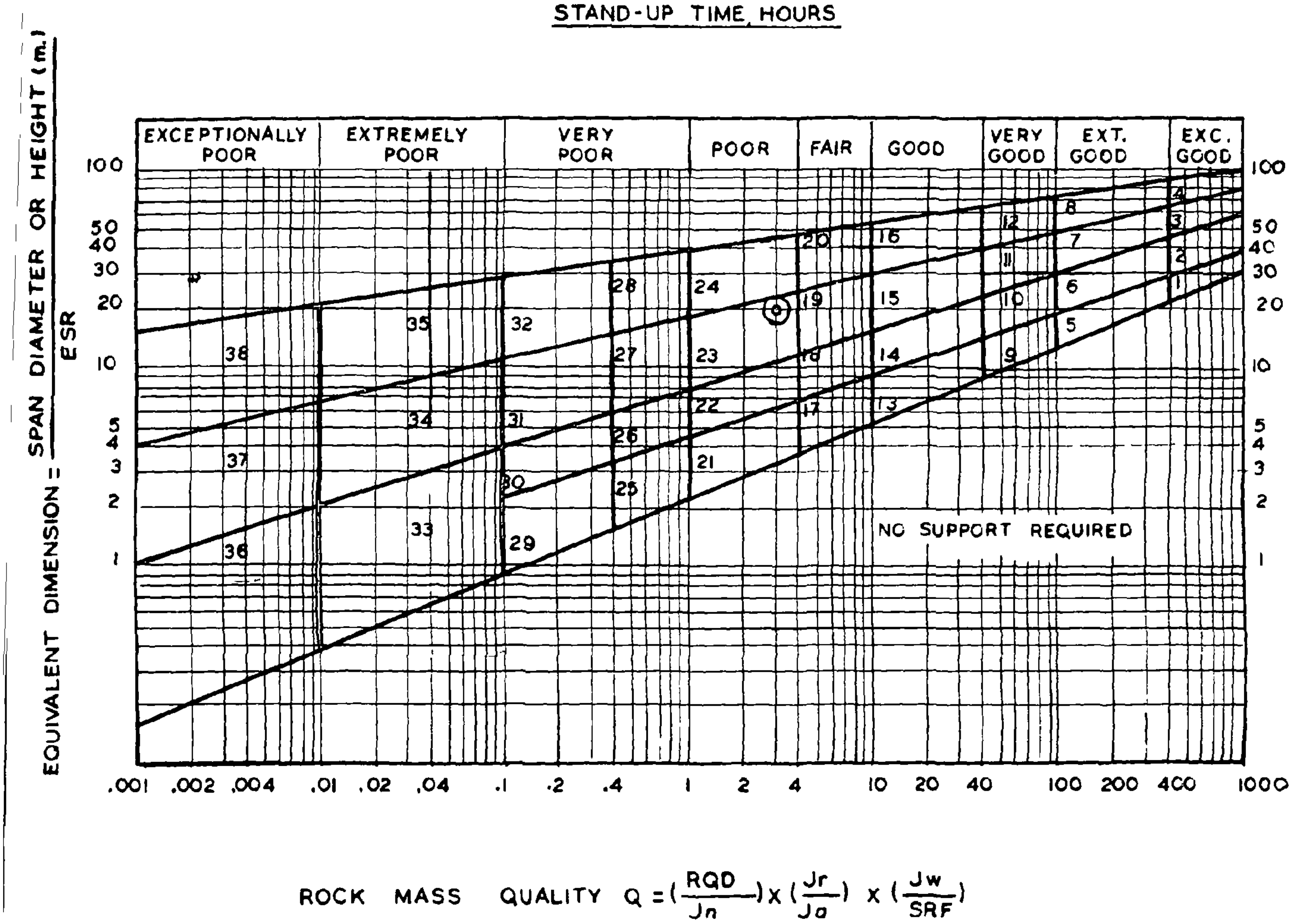


FIG. NO.– 10.9

ANNEXURE—I

DETERMINATION OF ECONOMICAL TUNNEL DIAMETER

The annual charges comprise of fixed costs, cost of operation and maintenance and value of power losses. These can be expressed as a function of Diameter and should be minimum for the Economical Diameter.

Using the following Symbols and Assumptions

D—Tunnel Dia in metres
d—Mean thickness of lining
E—Mean unit price of tunnel excavation = Rs. 250/- per cu m.
L—Unit price of concrete lining = Rs. 450/- per m^3.
G—Mean unit price of grouting = Rs. 200/- per m^3.
A—Total cost of tunnel per m length.
C—Contingencies (percentage of total cost = 5%)
S—Supervision charges (percentage of total cost = 15%)
O—Operation and Maintenance cost = 8%
Y—Life of project in years = 50 years
P—Depreciation factor (For straight line method = 1/50)
N—Rate of interest (7%)
B—Manning's Rugosity co-efficient = 0.014 for concrete lined tunnel
R—Hydraulic mean Radius (D/4 for circular tunnel)
Q—Equivalent discharge = 200 m^3/sec.
U—Value of one unit of power
e—Overall efficiency of plant.

Construction Cost and Fixed Charges

(*a*) Cost of Excavation

$$= \frac{E\pi\,(D+2d)^2}{4}\ \text{Rs/m} = \frac{E(D+2d)^2}{4}$$

(b) Cost of Lining

$$= L\left[\frac{(D+2d)^2 - D^2}{4}\right]\ \text{Rs/m}$$

(c) Cost of Grouting

$$= G(D+2d)$$

∴ Total cost per Linear metre

$$A = \frac{E(D+2d)^2}{4} + L\left[\frac{(D+2d)^2 - D^2}{4}\right] + G(D+2d)$$

$$= \frac{ED^2}{4} + EDd + Ed^2 + Ld^2 + LDd + GD + 2Gd$$

$$= \frac{ED^2}{4} + D(Ed+Ld+G) + Ed^2 + Ld^2 + 2Gd$$

This must be increased by C for contingencies and by S for Supervision charges.

Overall cost = $A(1+C)\cdot(1+S)$ per metre.

Annual charges on tunnel due to Depreciation and interest = $A(1+C)\cdot(1+S)\cdot(P+N)$ per metre

O & M Cost

This can be taken as percentage of the gross annual cost and expressed as O and M Cost = $A(1+C)\cdot(1+S)\cdot O$ per m.

Value of Annual Losses

The main head loss in the tunnel is due to the frictional losses which can be determined by Manning's formula as a function of Dia.

Power loss over a given period is proportional to the product of discharge, head loss and time duration.

If Q_1, Q_2, Q_3, etc. are the discharges which run for time T_1, T_2, T_3 respectively then:

$$Q^3T = Q_1^3T_1 + Q_2^3\,T_2 + Q_3^3\,T_3 + \ldots$$

Loss of Head due to friction per m length

$$h = \frac{n^2V^2}{R^{4/3}} = \frac{n^2Q^2}{\left(\frac{D^2}{4}\right)^2\left(\frac{D}{4}\right)^{4/3}}$$

$$= 101.12\ D^{-16/3}\ n^2Q^2$$

Power loss = $9.8\ Q\,h \times e$

Total No. of hours of operation per year = 8760 hours.
Annual Power loss = $9.8\ Qh\cdot Te$

$$= 9.8\ eTQ\ (101.12\ D^{-16/3}\ n^2Q^2)$$

$$= 1000\ en^2TQ^3D^{-16/3}$$

∴ Cost of annual power

$$= 1000\ en^2TQ^3D^{-16/3}\cdot U.$$

Total Annual Cost

$$T_1 = A(1+C)(1+S)(P+N) + A(1+C)(1+S)O + 1000e\ n^2TQ^3D^{-16/3}\ U$$

where $A = \frac{ED^2}{4} + D(Ed+Ld+G) + Ed^2 + Ld^2 + 2Gd$

For the Economy $\frac{dT_1}{dD} = 0$

Differentiating the above

$$\left[\frac{ED}{2}+d(E+L)+G\right]\cdot(1+C)\cdot(1+S)\cdot(P+N)$$

$$-\frac{1000\times 16}{3}\,n^2Q^3TUeD^{-19/3}=0$$

$$\text{or, } D+\frac{2d(E+L)+G}{E}=\frac{32000}{3}\cdot\frac{n^2Q^3TUeD^{-19/3}}{E(1+C)\cdot(1+S)\cdot(P+N)}$$

$$\text{or, } \frac{32000}{3}e\,n^2Q^3\,\frac{T.U}{E(1+C)(1+S)(P+N)}\cdot D^{-19/3}$$

$$=D+\frac{2d(E+L)+G}{E}$$

or, $mD^{-19/3}=D+K$

or, $\log_e m\ D^{-19/3}=\log_e(D+K)$

or, $\log_e m-\frac{19}{3}\cdot\log_e D=\log_e(D+K)$

By Substituting

$$\log_e D=\frac{2(D-1)}{D+1}$$

$$\log_e(D+K)=\log_e K+\frac{2D}{2K+D}$$

then $\log_e m-\frac{19}{3}\,\frac{2(D-1)}{D+1}=\log_e K+\frac{2D}{2K+D}$

or $\frac{2D}{2K+D}+\frac{19}{3}\,\frac{2(D-1)}{D+1}=\log_e m-\log_e K$

By multiplying $(2K+D)(D+1)$, we obtain

$$2D(D+1)+\frac{38}{3}(2K+D)(D-1)-(2K+D)(D+1)$$

$$\cdot(\log_e m-\log_e K)=0$$

By substituting the values assumed we have

$$m=\frac{32000}{3}\,en^2Q^3\cdot\frac{T.U}{E(1+C)\cdot(P+N)\cdot(1+S)}$$

$$=\frac{32000}{3}\cdot\frac{0.85(0.014)^2\cdot(800)^3\cdot 8760\times 0.10}{250(1+0.03)\cdot(1+0.08)\cdot(0.02+0.07)}$$

$$=4.975\times 10^8$$

$$K=\frac{2d(E+L)+G}{E}$$

$$=\frac{2\times 0.6(250+450)+200}{250}$$

$$=4.16$$

By substituting these values

$$2D(D+1)+\frac{38}{3}(2K+D)(D-1)-(2K+d)(D+1)$$

$$(\log_e m-\log_e k)=0$$

$$2D(D+1)+\frac{38}{3}(2\times 4.16+D)(D-1)$$

$$-(2\times 4.16+D)(D+1)(\log_e 4.975\times 10^8-\log_e 4.16)=0$$

$$2D(D+1)+\frac{38}{3}(8.32+D)(D-1)-(8.32+D)(D+1)$$

$$(20.025-1.4250)=0$$

By solving this equation we will get

$D=8.5$ metres

Economical Dia of tunnel as per Hand Book of Hydro-Electric Engg. By Dr. P.S. Nigam is given by

$$D^{7\cdot 33}=\frac{19.35\,Q^3n^2e\,u\times 10^5}{(E+0.36\,L)\cdot O}$$

where, D—Dia of tunnel
Q—Discharge through tunnel = 800 m³/sec
n—Rugosity coefficient = 0.014
e—Overall efficiency = 85%
u—Cost of power = 10 paise per unit,
L—Unit price of lining = Rs. 450/- per m³,
E—Unit price of tunnel Excavation = Rs. 250/- per m³
O—Annual O and M cost = 8% of total cost

By substituting these values we have

$$D^{7\cdot 33}=\frac{19.35(200)^3\cdot(0.014)^2\times 0.85\times 0.10\times 10^5}{(250+0.36\times 450)\times 0.08}$$

$=8.72$ metres

ANNEXURE—II

DETAILED DESIGN PROCEDURE FOR TUNNEL SUPPORT SYSTEM

Having assessed the rock loads and the spacing of the blocking points, the actual design of supports required is taken up. The design is carried out stepwise in the following manner :

(*i*) Construction of load diagram
(*ii*) Construction of force polygon
(*iii*) Determination of Thrusts, and
(*iv*) Computation of stresses in the arch rib.

The procedure is explained below with the help of a solved example for the design of an underground power house cavity.

Design Example

As per the geological classification the rock likely to be encountered is "Moderately Blocky and Seamy". Hence from Terzaghi's Table of rock loads, the rock load expected is in the range of 0.25 B to 0.35 $(B+H_t)$; B 25.5 m and $H_t = 8.8$ m.

Now,

$$0.25\,B = 0.25 \times 25.5$$
$$= 6.375 \text{ m}$$
$$0.35 = 0.35\,(25.5 + 8.8)$$
$$(B+H_t) = 12.005 \text{ m}$$

Taking the average value of the above two,

$$\text{Design Rock Load } H_p = \frac{6.375 + 12.005}{2} = 9.19 \text{ m}$$

Since a very heavy rock load is to be supported, it is proposed to use *RSJ* 300 × 140 × 46.2 kg/m at 250 mm centre to centre.

Construction of Load Diagram

As shown in the accompanying figure, let us provide 10 blocking points at a spacing of 1.5 m. One blocking point is fixed at the springing level (point 1) and one blocking point near the crown (point 10). Other blocking points are equally spaced in between the above two points.

The vertical rock load coming on the each blocking point is now computed from the load diagram as shown below :

Load per metre width of rib = 1 × 9.19 × 0.25 × 2500 = 5757 kg. Where 2500 kg/m³ is the assumed unit weight of rock.

Load on blocking point No. 1 = 0.6 × 5750 = 3450 kg
Load on blocking point No. 2 = 1.3 × 5750 = 7475 kg
Load on blocking point No. 3 = 1.3 × 5750 = 7475 kg
Load on blocking point No. 4 = 1.35 × 5750 = 7783 kg
Load on blocking point No. 5 = 1.4 × 5750 = 8050 kg
Load on blocking point No. 6 = 1.45 × 5750 = 8338 kg
Load on blocking point No. 7 = 1.45 × 5750 = 8338 kg
Load on blocking point No. 8 = 1.5 × 5750 = 8625 kg
Load on blocking point No. 9 = 1.5 × 5750 = 8625 kg
Load on blocking point No. 10 = 1.25 × 5750 = 7188 kg

Total Load = 75, 327 kg.

The loads W calculated above are then marked on the load diagram to a suitable scale and are resolved into a radial force F and a tangential component Ft if the tangent is inclined less than 25° to the horizontal, otherwise a 25° component.

Next, an upward vertical reaction Rv is shown at the springing level (blocking point 1).

Finally, the chords connecting blocking points are drawn. These chords represent the direction of thrust in each panel.

Construction for Force Polygon

From a common point (pole) draw vertical ray Rv and rays parallel to each chord and label them $T_1 - 2$, $T_2 - 3$ etc.

End up with a horizontal ray Rh. Calculate the sum Rvt of all the roof load. In our case it works out of 75,327 kg. Since the vertical load is made up of two components, the total vertical pressure Rv is smaller than Rvt. To find out its real value, we construct a trial polygon starting from an arbitrarily selected point on the ray Rv. Let us assume this point at a distance of 0.8 Rvt from the pole.

Starting at this point draw a line f_1 parallel to force F_1 on the load diagram to intersect ray T_{1-2}. From this intersection draw a line f_2 parallel to force F_2 to intersect ray T_{2-3} and repeat until the horizontal ray is reached thus completing the trial polygon. In our case Rvt = 75,327 kg and 0.8 Rvt = 60,262 kg.

Now compare f_2 on the polygon with force F_2 on the load diagram, f_3 with force F_3 and so on. It will be seen that atleast at one point the force F on the load diagram will exceed the corresponding trial polygon force f. Transfer this force F to the trial polygon extending the rays as necessary and complete the new polygon. This is the true polygon.

Determination of Thrusts

The length of the rays of the true polygon represent thrust. The longest ray gives the largest thrust on which the design is based. In this example $T=93{,}000$ kg.

Computation of Stresses in the Arch Rib

The following symbols are used in the formulae for the computation of stresses in the arch rib:

C = Chords length between blocking points in cm = 150 cm in our case.

R = Radius of neutral axis of the rib in cm = 2000 cm.

h = Rise of arc between blocking points in cm.

$$=R-\sqrt{R^2-\left(\frac{C}{2}\right)^2}$$

$$=2000-\sqrt{2000^2-\left(\frac{150}{2}\right)^2}=1.4 \text{ cm.}$$

T = Thrust in kg = 93,000 kg.

M_t = Bending moment in the rib in kg cm.

$$=hT$$
$$=1.4\times 93{,}000$$
$$=1{,}30{,}200 \text{ kg cm}$$

$M_{max.}$ = Maximum bending moment in kg cm in the rib.

$$=0.86\ M_t$$
$$=0.86\times 1{,}30{,}200$$
$$=1{,}11{,}972 \text{ kg cm}$$

Z = Section modulus of the rib in cm^3.

= 8,603.6 cm^3 for *RSJ* 300 × 140

A = Sectional area of the rib in cm^2

= 56.76 cm^2 for *RSJ* 300 × 140

fr = stress in the arch portion of rib kg/cm^2.

$$=\frac{T}{A}+\frac{M_{maximum}}{Z}$$

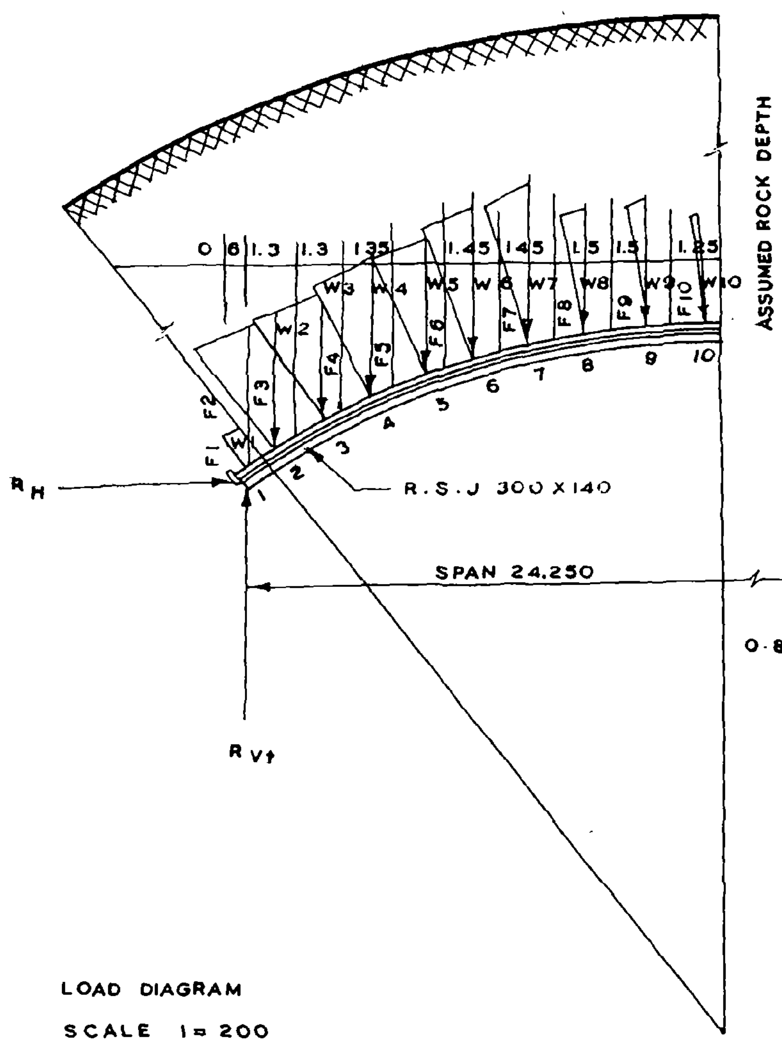
$$=\frac{93{,}000}{56.26}+\frac{1{,}11{,}972}{8{,}603.6}$$

$$=1653.04+13.01$$
$$=1666.05 \text{ kg/cm}^2.$$

Since allowable stress in the rib is 1,690 kg/cm^2 the design is considered to be safe.

ANNEXURE II

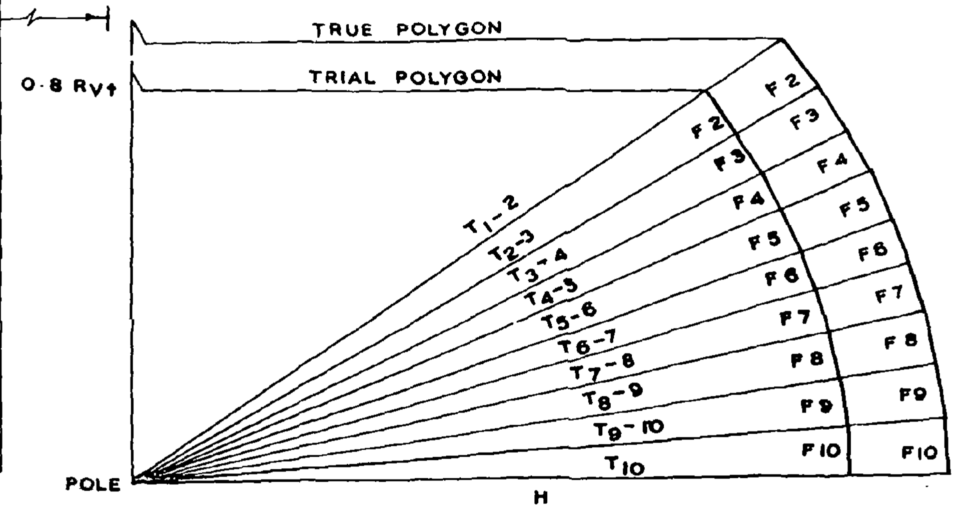
DESIGN OF POWER HOUSE ROOF SUPPORT USING PROCTOR AND WHITE'S GRAPHICAL METHOD



LOAD DIAGRAM

SCALE 1 = 200

1 CM = 4000 KGS



FORCE POLYGON

SCALE 1:1000 1CM = 10,000 KGS

ANNEXURE–III

CHUKHA HEAD RACE TUNNEL

Structural Design of Tunnel Lining

Data: Finished internal dia of tunnel $=4.9$ m

$t=$ Thickness of lining $=0.2$ m

$r=$ Internal radius of tunnel $=2.45$ m

$R=$ Mean radius of tunnel lining $=2.45+0.1$

$=2.55$ m

$E=$ Young's Modulus of lining material (M: 200 Conc).

$$=\frac{2.1\times10^6}{13}\ \text{kg/cm}^2=1.6154\times10^6\ \text{t/m}^2.$$

$$I=M.I.\ \text{of section of lining}=\frac{bd^3}{12}$$

Considering 1 metre strip of lining.

$$I=\frac{1.0\times0.2^3}{12}=0.0007\ \text{m}^4$$

$W=$ Unit wt. of water 1.0 t/m^3

$W_c=$ Unit wt. of conc. $=2.4$ t/m^3

$P=$ Total rock load on mean dia.

In the design, 1.38 m of rock load has been assumed for sound rock as per geological formation.
(Rock load $=0.25$ B $-$ 0.25 B $-0.25\times5.5-1.315$ m)

$$p=1.375\times2.5\times5.1\times1.0=17.53\ t.$$

Radial Shear

ϕ	Due to Uniform Ver. load.	Due to conduit weight	Due to contained water	Total
0°	Zero	Zero	Zero	Zero
45°	$-0.25\,p$ $=-0.25$ $\times17.53$ $=-4.3825$	$-0.8976\,W_ctR$ $-0.8976\times2.4\times0.2$ $\times2.55$ $=-1.0987$	$-0.4488\,Wr^2$ $=0.4488\times1.0$ $\times2.45^2$ $=-2.6939$	$-8.1751\ t$
90°	Zero	$+0.1667\,WctR$ $=0.1667\times2.4$ $\times0.2\times2.55$ $=0.204$	$+0.0833\,W_r2$ 0.0833×1.0 $\times2.45^2$ $=0.500$	$+0.704\ t$
135°	$+0.25\,P$ $=+4.38.25$	$+0.6732\,WctR$ $\pm0.673\times2.4\times0.2$ $\times2.55$ $=0.824$	$+0.3366\,Wr^2$ $=0.3366\times1.0$ $\times2.452$ $=2.0204$	$+7.2269\ t$
180°	Zero	Zero	Zero	Zero

Maximum radial shear $=8.1751\ t$

$$\therefore\ \text{Shear Stress}\ =\frac{8.1751}{100\times20}=4.0876\ \text{kg/cm}^2$$

<5 kg/cm^2 safe.

Bending Moments

ϕ	Due to Vertical Load	Due to Conduit Wt	Due to Contained Water	Total
0°	$+0.125\,PR$ $=0.125$ $\times17.53$ $\times2.55$ $=5.5877\ tm$	$+0.4406\,WctR^2$ $=0.4406\times2.4$ $\times0.2\times2.55^2$ $=1.3752$	$+0.2203\,Wr^2R$ $=+0.2203\times1.0$ $\times2.45^2\times2.55$ $=3.3720$	$+10.3349$ tm
45°	Zero	$-0.0334\,W_ctR^2$ $=-0.334\times2.4$ $\times0.2\times2.55^2$ $=-0.1042$	$-0.0167\,Wr^2R$ $=-0.0167$ $\times1.0\times2.45^2$ $\times2.55$ $=-0.2555$	-0.3598 tm
90°	$-0.125\,PR$ $=-5.5877$	$-0.3927\,WctR^2$ $=-0.3927\times2.4$ $\times0.2\times2.55^2$ $=-1.2257$	$-0.1963\,Wr^2R$ $=-0.1963\times1.0$ $\times2.45^2\times2.55$ $=-3.0046$	-9.8180 tm
135°	Zero	$+0.0334\,WctR^2$ $=+0.1042$	$+0.0167\,Wr^2R$ $=+0.2556$	$+0.3598$ tm
180°	$+0.125PR$ $=+5.5877$	$+0.3448\,WctR^2$ $=+0.3488\times2.4$ $\times0.2\times2.55^2$ $=1.0762$	$+0.1724\,Wr^2R$ $=0.1724\times1.0$ $\times2.45^2$ $\times2.55$ $=2.6388$	$+9.3027$ tm

Now, $m=Qbd^2$ $\quad Q=13,\quad b=100$ cm

$$d=\sqrt{\frac{m}{Qb}}=\sqrt{\frac{10334.9}{13\times100}}=28.19\ \text{cm}$$

Actual thickness of lining provided $=20$ cm
Including Overbreak in rock, total thickness of concrete lining upto the payline $=30$ cm.
Hence, the depth is considered to be adequate.

Normal Thrust

ϕ	Due to Uniform vertical load	Due to conduit weight	Due to contained water	Total
0°	Zero	$+0.1667\,WctR$ $=0.1667\times2.4$ $\times0.2\times2.55$ $=0.204$	$-1.4166\,Wr^2$ $=1.4166\times1.0$ $\times2.45^2$ $=-8.5031$	$-8.2991\ t$
45°	$+0.25\,P$ $=0.25\times17.53$ $=4.3825$	$+1.1332\,W_{ct}R$ $=1.1332\times2.4$ $\times0.2\times2.55$ $=+1.3870$	$-0.7869\,Wr^2$ $=-0.7869$ $\times1.0\times2.45^2$ $=-4.7234$	$+1.0462\ t$
90°	$+0.5\,P$ $=+0.5$ $\times17\ 53$ $=+8.765$	$+1.5708\,W_{ct}R$ $=1.5708\times2.4$ $\times0.2\times2.55$ $=+1.9227$	$-0.2146\,Wr^2$ -0.2146×1.0 $\times2.45^2$ $=-1.2881$	$+9.3995\ t$
135°	$+0.25\,P$ $=4.3825$	$+0.4376\,W_{ct}R$ $=0.4376\times2.4$ $\times0.2\times2.55$ $=+0.5356$	$-0.4277\,Wr^2$ -0.4277×1.0 $\times2.45^2$ $=-2.5673$	$+2.3508\ t$
180°	Zero	$-0.1667\,WctR$ $=-0.204$	$=-0.5834\,Wr^2$ $=-0.5834\times1.0$ $\times2.45^2$ $=-3.5019\ t$	$-3.7059t$

Maximum negative thrust $= -8.2991t$
(which indicates tension at the crown)

Tensile stress in concrete at crown. $= \frac{8.2991 \times 1000}{100 \times 20} = 4.1496$ kg/cm^2 < 5 kg/cm^2 safe

Horizontal Deflection

ϕ	Due to uniform Ver. level	Due to conduit Wt	Due to contained water	Total
0°	ZERO	ZERO	ZERO	ZERO
45°	$0.01473 \frac{PR^3}{EI}$ $=3.77$ mm	$0.05040 \frac{WctR^4}{EI}$ $=0.92$ mm	$0.0252 \frac{Wr^2R^3}{EI}$ $=2.23$ mm	6.92 mm
90°	$0.04167 \frac{PR^3}{EI}$ $=10.67$ mm	$0.13090 \frac{WctR^4}{EI}$ $=2.40$ mm	$0.06545 \frac{Wr^3R^3}{EI}$ $=5.79$ mm	18.86 mm
135°	$0.01473 \frac{PR^3}{EI}$ $=3.77$ mm	$0.04216 \frac{WctR^4}{EI}$ $=0.77$ mm	$0.02108 \frac{Wr^2R^3}{EI}$ $=1.86$ mm	6.40 mm
180°	ZERO	ZERO	ZERO	ZERO

Deflections are within permissible limits.

Vertical Deflection

ϕ	Due to uniform Ver. load	Due to conduit weight	Due to contained water	Total
45°	$+0.02694 \frac{PR^3}{EI}$ $=0.02694 \times \frac{17.53 \times 2.55^3}{1.6154 \times 10^6} \times 0.0007$ $=0.0069$ m $=6.9$ mm	$+0.09279 \frac{WctR^4}{EI}$ $=0.9279 \times \frac{2.4 \times 0.2 \times 2.55^4}{1.6154 \times 10^6} \times 0.0007$ $=0.0017$ m $=1.7$ mm	$0.0464 \frac{Wr^2R^3}{EI}$ $=0.0464 \times 1 \times \frac{2.45^2 \times 2.55^3}{1.6154 \times 10^6} \times 0.0007$ $=0.0041$ m $=4.1$ mm	12.7 mm
90°	$0.04167 \frac{PR^3}{EI}$ $=10\ 67$ mm	$0.13917 \frac{WctR^4}{EI}$ $=2.55$ mm	$0.06958 \frac{Wr^2R^3}{EI}$ $=6.15$ mm	19.37 mm
135°	$0.05640 \frac{PR^3}{EI}$ $=14.44$ mm	$0.18535 \frac{WctR^4}{EI}$ $=3.40$ mm	$0.09268 \frac{Wr^2R^3}{EI}$ $=8.19$ mm	26.03 mm
180°	$0.08333 \frac{PR^3}{EI}$ $=21.33$ mm	$0.2618 \frac{WctR^4}{EI}$ $=4.80$ mm	$0.1309 \frac{Wr^2R^3}{EI}$ $=11.57$ mm	37.70 mm

Deflections are within permissible limits.

ANNEXURE—IV

TYPICAL DESIGN OF PORTAL

Clear Span —6 metres
Height —6 metres

Assuming a slope of 1 in 20 at the face of portal, top width of the column

$$= \frac{6000}{20} + 600 = 900 \text{ mm}$$

But, for calculation purposes we shall take it was 600 × 600. Let us assume size of beam also as 600 × 600. Its back may be refilled with lean concrete as shown in Figure 1.

Loading :

Size of beam $= 600 \times 600$
Self weight $= 0.6 \times 0.6 \times 2400 \times 6.6 = 570.24$ kg

Weight of surcharge assuming 45° dispersion

$$= \tfrac{1}{2} \times 6.6 \times 0.9 \times 3.3 \times 2400 = 23522 \text{ kg}$$

Total load coming over beam

$$= 23522 + 5702 = 29{,}224 \text{ say 30 tonnes}$$

In the portal ABCD shown in Figure 2, I for all the members is same being of same cross-sectional area 600 × 600.

Relative stiffness of members

$$AB = \frac{4\,EI}{L} = \frac{4}{6.3} = 6.3$$

$$BC = \frac{4}{6.6} = 0.60,\; CD = \frac{4}{6.3} = 0.63$$

Distribution factors at joint B for members

$$BA = \frac{0.63}{0.63 + 0.60} = 0.51,\quad BC = \frac{0.60}{0.63 + 0.60} = 0.49$$

Since, total load over beam $BC = 30$ tonnes.

$$FEM_{BC} = FEM_{CB} = \frac{WL}{12} = \frac{30 \times 6.6}{12} = 16.5\ tm$$

Doing moment distribution we can find the moments as shown below:

Joint	A	B	C	D
Member		BA AC	CB CD	
D.F.		0.51 0.49	0.49 0.51	
FEM		− 16.5	+ 16.5	
Dist.		+ 8.415 + 8.085	− 8.085 + 8.415	
C.O.	+ 4.207	− 4.042	+ 4.042	− 4.207
Dist.		+ 2.061 + 1.981	− 1.981 − 2.061	
C.O.	+ 1.030	− 0.990	+ 0.990	− 1.030
Dist.		+ 0.505 + 0.485	− 0.485 − 0.505	
Final Moments	+ 5.237	10.981 − 10.981	+ 10.981	− 5.237

Final B.M. Diagram is shown in Figure 3.

Free B.M. for beam is $\frac{WL}{8} = \frac{30 \times 6.6}{8} = 24.75\ tm$.

∴ Net moment for which steel is to be provided

$$= 24.75 - 10.98 = 13.77\ tm$$

∴ d reqd. $= 43.55$ cm

and At $= \frac{1377000}{2100 \times 0.87 \times 55} = 13.7 \text{ cm}^2$

provide 3 bars of 25 mm dia.

Shear Stress $= \frac{1500}{60 \times 0.87 \times 55} = 5.2 \text{ cm}^2$ safe

Bond stress $= \frac{15000}{0.87 \times 55 \times 23.56} = 13 \text{ kg/cm}^2$

provide 4 bars of 25 mm dia and provide stirrups nominally at a spacing of $0.87 \times d = 0.87 \times 55$ say 400 c/c.

Design of column :

Superimposed Load on each column $= \frac{30}{2} = 15$ tonnes.

Self Load $= 0.6 \times 0.6 \times 6.6 \times 2400 = 5702$ kg

∴ Total load on column $= 15000 + 5702 = 20702$ kgs.

Moment $= 5.237\ tm$.

Assuming $b = 60$, $d = 60$

$$\therefore \quad \frac{1.85\,P}{\sigma_{cu} bd} = \frac{1.85 \times 20702}{200 \times 60 \times 60} = 0.053$$

$$\frac{1.85\,M}{\sigma_{cu}\, bd^2} = \frac{1.85 \times 523700}{200 \times 60 \times 60 \times 60} = 0.022$$

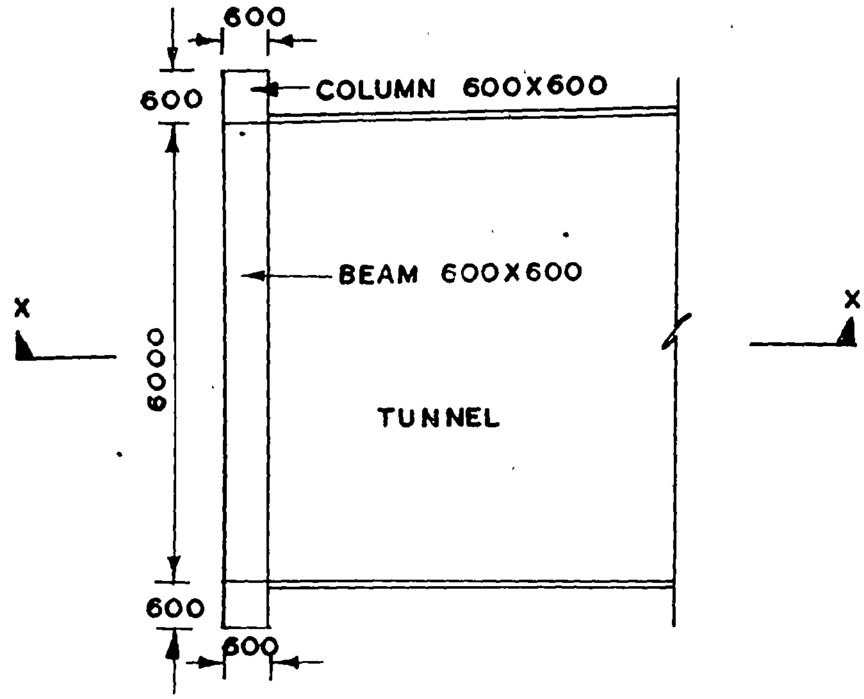
For $\frac{dt}{dl} = \frac{55}{60} = 0.91$ from chart 3.2 of Handbook of Tor-steel we find $\frac{\gamma}{\sigma_{cu}}$ is negligible

Hence, let us provide 0.8% steel.

$$\therefore \quad \frac{60 \times 60 \times 0.8}{100}$$

$$= 28.8 \text{ cm}^2$$

Provide 6 bars of 25 mm dia provide 10 mm dia stirrups of 250 c/c.



PLAN

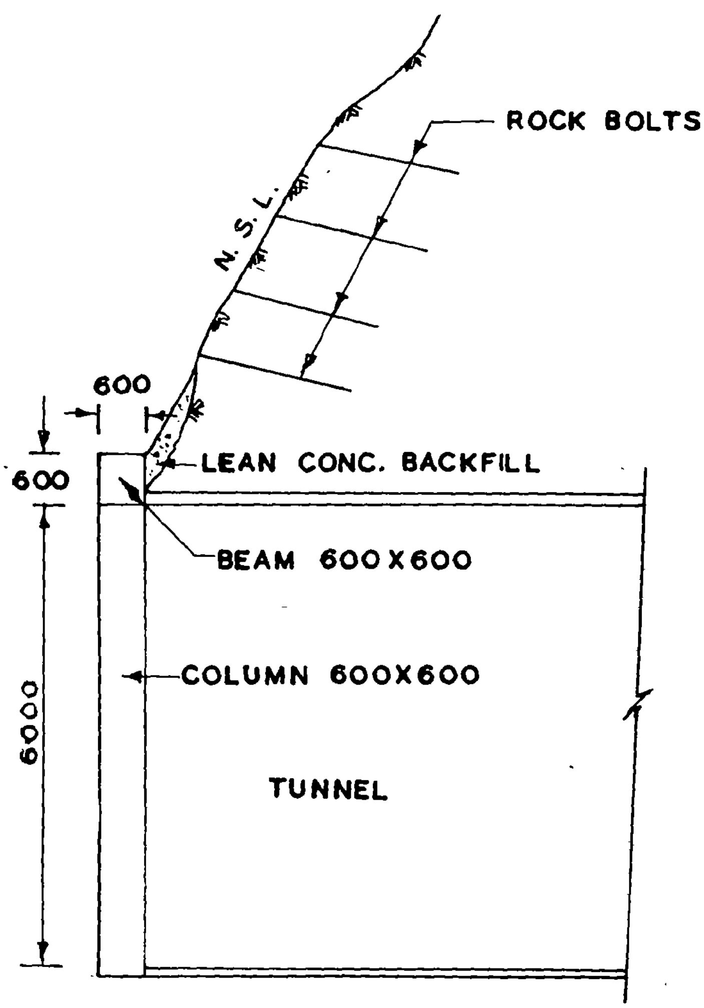


FIG-IV-1 SECTION X-X

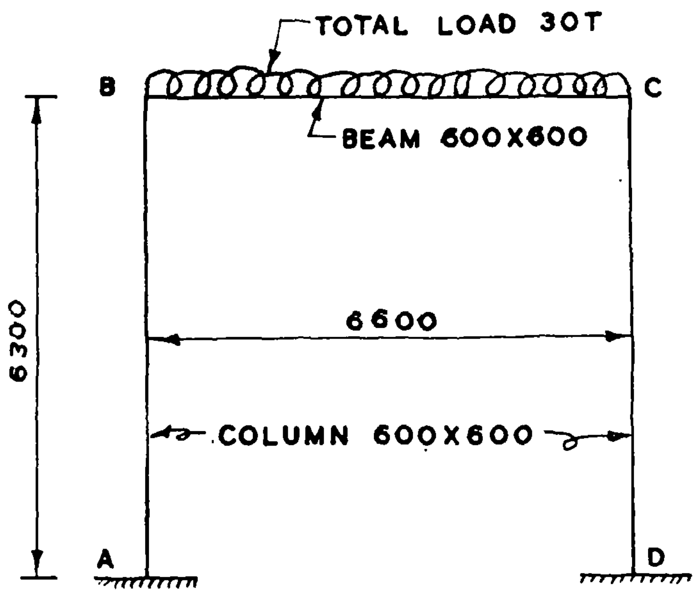


FIG - IV - 2

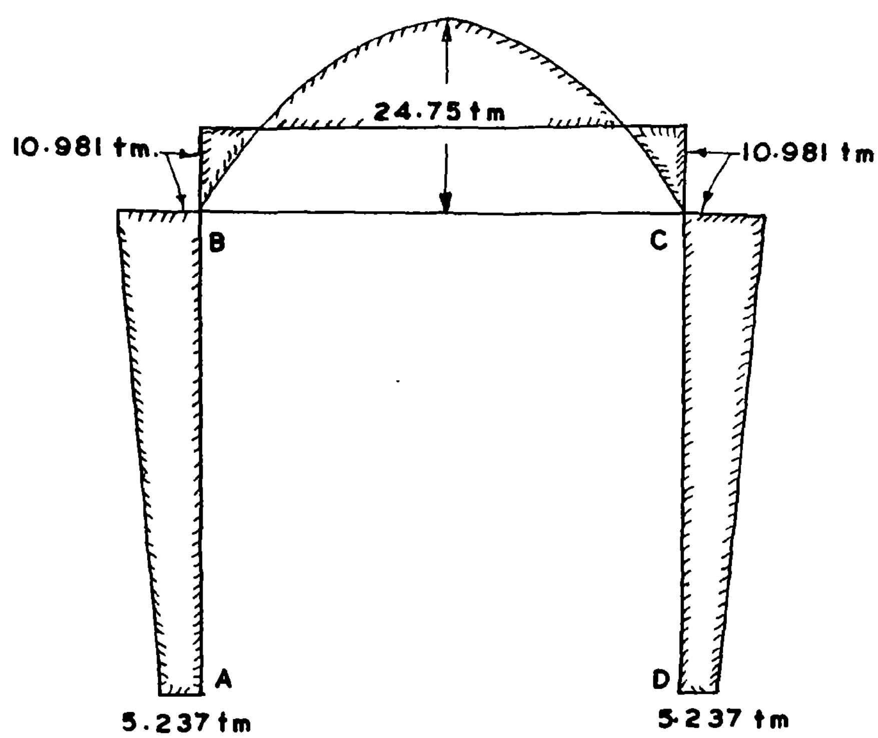


FIG - IV - 3

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