ANALYSIS OF NATHPA JHAKRI UNDERGROUND POWER HOUSE CAVITY

A DISSERTATION

submitted in partial fulfilment of the requirements
for the award of the degree
of
MASTER OF ENGINEERING
in
WATER RESOURCES DEVELOPMENT

By

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C E R T I F I C A T E

Certified that this dissertation entitled, ' ANALYSIS OF NATHPA JHAKRI UNDERGROUND POWER HOUSE CAVITY', which is being submitted by Er. Pavan Kumar Kohli, in partial fulfilment of requirements for the award of degree of Master of Engineering in Water Resources Development(Civil) of University of Roorkee is a record of candidate's own work, carried out by him under our supervision and guidance. The matter embodied in this disseration has not been submitted for the award of any other degree or diploma.

This is to further certify that he has worked with effect from 16th July, 1986 to 30th November, 1986.

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S Y N O P S I S

The design of underground cavern requires the knowledge of stress field, developed as a result of redistribution of insitu stresses, consequent upon the excavation of rock mass. The zones of compression and tension are delineated and the points of local stress concentrations are identified once the stress field has been defined. The parameters influencing the redistribution of stresses around an opening in rock mass include its shape, the magnitude and direction of insitu stresses, strength and mechanical properties of rock mass and the stage of excavation. The stability of the underground structure depends upon the local stress concentrations as well as the stress gradient around the opening. The stresses should be within allowable limits and steep stress gradients are to be avoided. Thus, extensive stress analysis, incorporating the parameters described above, is necessary to ensure the stability of cavern and to select best possible shape, layout and sequence of excavation.

The problem can be solved making use of the various numerical techniques. The 'Finite Element Method' provides an versatile tool for such an analysis and can incorporate nonlinearity, anisotropy, inhomogeneity and structural defects in rock mass.

In this work, the analysis of Nathpa Jhakri Underground Power House,based on the assumptions of linear,isotropic and homogeneous rock mass, has been carried out using finite element method.

The time required for data preparation in finite element analysis is quite large as compared to the total time for the

analysis and interpretation of results. Therefore, emphasis has been laid on the automatic data preparation. A computer programme, capable of generating complete input data for all construction stages for single cavity, has been prepared. The programme has been extended to generate the data for the analysis of two caverns. It also facilitates the user to draw the geometrical outline of the mesh, prescribe loadings and fix boundary conditions. Thus, a substantial saving in time has been achieved. It will be a very useful tool for the analysis of other structures of this type.

The results of the analysis indicate that the radius of roof arch should be kept as 17 metres. Further, it has been found that the width of the machine hall can be increased to 22.5 metres to house the upstream control valves. This will result in considerable saving in construction and auxiliary equipment cost. The factors influencing the stress field such as $K(\sigma_h/\sigma_v)$ and E have been varied and relationship between the stability factors such as local stress values and K has been obtained to foresee the behaviour and stability of cavern after the results of extensive field investigations, presently in progress, are obtained. If, the actual value of parameters influencing the analysis, indicate a substantial variation from the values used in this work, a separate analysis will be required.

A chapter on review of literature has been included, describing in brief, the underground power house structure and the relation of rock mechanics principles with the analysis of structures in rock mass.

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CHAPTER-1

INTRODUCTION

1.1 HISTORY OF UNDERGROUND DEVELOPMENTS

The planning, analysis, design and construction of structures in rock, particularly in Himalayan Geology, is a subject of vast interest for engineers of this region. As one looks back into the history of mankind, the existence of underground structures dates back to 4000-3000 B.C. when the Sumerians and Egyptians had developed the art of rock construction, the main structures being tunnels, vaults and other works. The use of underground structures was normally restricted to the field of mining including shafts and tunnels.

The earliest underground power station is 'Edward Dean Adams' which was constructed in the year 1890 in U.S.A. (Niagara falls). The structure consisted of water wheels at the bottom of the shaft 24' wide and 130' deep with the generators at the surface. The inspiration from Niagara fall station led to the development of first true underground power station at Snoqualmie falls in U.S.A. in the year 1898. The structure housed four generating units with a total capacity of 6000 KW. The 30'x40'x200' machine hall chamber was constructed in basaltic rock, about 268' below the rock line.

1.2 ROCK AS A STRUCTURAL MATERIAL

Rock insitu, as a structural material, forms part of every major engineering activity. The rock, alongwith other building materials such as steel and concrete, forms a composite structure. A concrete gravity dam built on the rocky foundation is a good example of composite structure.

The quantitative data regarding the properties and behaviour of rock mass is essential to take up the design and construction assignment. In the recent past, many sophisticated methods have been developed to assess the mechanical and engineering properties of rock mass. The laboratory tests have revealed that behaviour of rock can range from that of a brittle material to that of a ductile material depending upon the combinations of loadings i.e. pressure and temperature variations. The range of temperatures and pressures in most engineering and mining works is such that, for harder rocks, their basic behaviour is that of a brittle material or in transition from brittle to ductile. The ductile behaviour is predominant in softer rocks.

In practice, the rock in nature can range from a massive unjointed mass to a highly jointed mass with geological and structural defects. Thus, the study of rock mass properties insitu is of great significance. The schistosity results in anisotropic behaviour which is even noticed in hard igneous rocks.

The rock mass analysis was restricted to simple two dimensional, analogue and experimental stress analysis such as photoelastic methods till the advent of digital computers. The evolution of fast digital computers and the subsequent development of techniques for applying the finite element procedure to non-elastic and jointed rock masses has given a powerful tool to the rock engineers.

1.4 DESIGN AND CONSTRUCTION

It would be like a dream to think of a perfectly homogeneous, elastic and isotropic rock mass. The rock in-situ is always under the influence of stresses. The measurements, actually done in many cases ,have shown that the vertical and horizontal stresses vary considerably from theoretical values based on the elastic and gravity analysis. It has been observed that horizontal stresses are considerably higher than those indicated by the theory. The cause can be attributed to the historic tectonic developments in the region.

It must be emphasised that the absolute values of stresses in rock are not so important as their relative magnitudes and orientation with respect to the excavation works. The factor is of prime importance to the design and construction engineers, as there is redistribution (readjustment) of initial or primary stresses resulting in development of stress concentrations. It is the position and magnitude of these stress concentrations which govern the choice of shape and geometry of the excavation. The stress concentrations, in

1.3 ANALYSIS OF STRUCTURES IN ROCK

The insitu rock mass is always subjected to triaxial stress field and the resulting strains. It is, therefore, necessary to treat all the structures built in rock mass e.g. underground caverns and tunnels and also the structures built over the mass such as dams etc. as continuous, indeterminate structures.

However, owing to the nature of complexities involved in three dimensional analysis, the problem is generally simplified as biaxial with either plane stress or plane strain. The excavation of rock mass disturbs the equilibrium of the insitu stresses resulting in secondary stress pattern. Thus, the problem can be defined as twofold, firstly to determine the exact amount and nature of insitu stresses present before the construction work is taken up and; secondly to study the deformation pattern of rock mass owing to the excavation. The other aspect of the analysis is to check the stability of the excavation under the new stress field.

In order to make an approximate analytical appraisal of the stability of the rock structure, it is essential to have the quantitative data regarding the insitu stress field and deformation and strength characteristics of rock mass with respect to both load and time. The rock mass behaviour, though comprising of many non-elastic characteristics can be described with reasonable degree of accuracy by using the elastic theory.

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The construction procedure of the underground caverns needs due consideration at the design stage. The shape should be such that the excavation can be carried out without difficulty; this applies in particular to the turbine pits and draft tube cavern of the underground power structures. In the design of cavities in weak rocks, the ultimate conditions, which may develop during or after the construction period have to be considered. The theory given by Terzaghi (1946) for the estimation of rock loads is still an useful tool for the engineers engaged in planning and design of structures in rock.

The most important aspect of the design of underground structures is to correlate the results of the laboratory tests or small scale test openings with the conditions that can be anticipated in full size work. The behaviour of the rock, which was anticipated on the basis of the exploratory drift results may be quite different in the actual work. The other important aspect in the design of underground structures is monitoring the behaviour during construction. This can give an idea of behaviour of rock mass in near future and thus, potentially hazardous conditions can be avoided. The instrumentation to include all the critical points is an important and necessary assignment. The output of the instrumentation is important in providing a check on the design assumptions and design forecast of the behaviour. Thus, the output data should be available in the form which can be readily used by the engineer-in-charge.

1.5 STRESSES AROUND AN OPENING

The excavation of the cavity in the rock mass results in setting free the horizontal and vertical forces around the opening. The overburden rock pressure by-passes the opening and the rock around the opening has to support it. It may result in displacement/deformation of rock mass around the cavity.

An underground opening results in lateral squeeze of vertical force lines resulting in large stress concentrations at the abutments. It has been noticed that between the tunnel vault and the natural arching of the rock, tension zones form at the crown and invert.

The stability of rock mass excavation, after the renewal of force equilibrium resulting from excavation, is a function of rock mass properties like shear and tensile strength mobilization. The ratio $K(\sigma_h/\sigma_v)$ is one of the most important parameters of stress analysis. The value of K generally varies from 0.3 to 1.5. The stress pattern around the opening is dependent largely upon the 'K' value. Another important parameter influencing the stresses around the opening, is its shape. Sharp corners are points of high stress concentrations and should, therefore, be avoided. Negative stress pattern develops along high flat vertical walls and on flat bottom of the excavations. Similarly, a low rock vault results in stress patterns that could be detrimental to the stability of the cavern. The rise to span ratio of the roof arch should be selected such that minimum or even no tensile stress pattern develops. The orientation of several parallel caverns also influences the stress pattern.

1.6 TRIAXIAL AND BIAXIAL ANALYSIS

The present day fast digital computers and advancement in the application of the numerical techniques, have made it possible to analyse the complex three dimensional problems. However, simplifying assumptions are generally made to reduce the analysis to biaxial conditions i.e. plane stress or plane strain cases. For a long and uniform cavern in rock mass, following assumptions are made.

i) Plane strain analysis can be used i.e. two dimensional analysis of a slice perpendicular to the axis of the cavern.

ii) Radial stress σ_r is smallest principal stress and tangential stress σ_θ is the largest principal stress, σ_z, i.e. the stress ||lel to the longitudinal axis of the tunnel is intermediate principal stress.

iii) The Mohr's stress criterion holds good.

iv) The rock is assumed to be isotropic and perfectly elastic.

The first step in the analysis is to assume and select the geometrical sections which can be analysed for two-dimensional stress field. Photoelastic method and F.E.M. Analysis are the methods generally adopted. The elastic analysis delineates the zones of excessive tension or compression. The areas of large stress gradients and points of low and zero stresses are specified. The planes along which the shear failure is foreseen can also be identified.

It must be mentioned here that the points of zero stress separate the tension and compression zones. These are also points of surface deformation inflection. The stress patterns around the opening can be obtained, incorporating the geological and structural features of the rock mass around the cavern. The best possible shape and layout of the caverns can be achieved through such an analysis. The arrangement selected should be such that the excessive stress concentrations are eliminated avoiding high stress gradients between adjoining excavations and around portals.

Three dimensional studies should be made for the cases where the situation warrants it. However, for many cases, the results of two dimensional studies are adequate.

1.7 SCOPE OF THE STUDY

The present work deals with the analysis of Nathpa Jhakri underground Power House. The analysis has been done assuming the rock mass to be linear, isotropic and homogeneous medium. The two dimensional, linear, elastic, plane strain, finite element analysis has been conducted to study the following -

i) Stress distribution around the cavity during different stages of excavation.

ii) The effect of changing rise to span ratio of the roof arch.

iii) Stresses in the concrete arch in roof.

iv) The effect of increasing the width of the machine hall to accommodate u/s control valves.

v) The effect of transformer hall excavation on the stress distribution.

vi) The effect of different values of the ratio of horizontal to vertical principal stresses.

In order to analyze a large number ofcranesageneralised computer programme, for generation of finite element mesh and corresponding input data required for the analysis, has been developed. The programme is capable of generating input data of single cavity for all stages of excavation. The programme can also include a predefined second cavity with the main cavity. In the present case, the transformer hall cavity on d/s of machine hall cavity has been included. The programme generates complete input data for the analysis of two caverns taken together.

The programme facilitates the user to draw the geometrical outline of the finite element mesh directly on the terminal of HP 1000 system, perform modifications, prescribe loading conditions and fix the boundary conditions. The finite element mesh for different cases, alongwith the necessary input data, were generated through this programme. Thus, a substantial saving in time has been achieved.

1.8 NATHPA JHAKRI PROJECT

The project, located in District Kinnaur of Himachal Pradesh has been conceived as a run of the river type development on river Sutlej. The project envisages a 60.5 m high diversion dam across the river Sutlej at Nathpa, two intakes and an underground desilting complex with four parallel chambers each 350 m x 25 m x 38 m , a 27.7 km long and 10.15 m dia head race tunnel, a depressed aqueduct crossing at Manglad Khad nearly 1 km long, two auxiliary surge shafts 10.15 m ⌀ and 118 m high, at either end of this crossing, a main surge tank - 25m dia and 130 m high, three circular steel lined penstocks 650 m long and an underground power house 170 m x 20m x 40m, housing six generating units of 250 MW each, utilizing a head of 425 metres with a 10.15 m ø shaped tail race tunnel 280 m long. The project will also utilize the waters of an intervening stream - the Sholding Khad- through a trench weir and a drop shaft 125 m deep for power generation.

A brief description of the project components is as under -

a)	Diversion Dam	Height 60.5M(above foundation)
		Type - Straight Gravity(concrete)
b)	Pressure tunnel	Type - Circular, concrete lined
		Dia - 10.15 M
		Length - 27.70 km
		Design discharge - 405 cumecs
c)	Surge tank	Type - Restricted Orifice
		Height - 130 M
		Diameter - 25 M
d)	Penstock tunnels	Type - Circular steel lined
		Lining: - high tensile steel (12mm to 45 mm)
		Diameter - 6.00 M
		Length - 650 M
		Branches - 4.0 M dia, 60 M long.

e)	Power house	Type - Underground Size - 170m x 20m x 40 M Type of turbine - Vertical Axis, Francis turbine Gross head - 488 M Design head - 425 M Units - 6 x 250 MW
f)	Tail Race Tunnel	D Shaped, 10.15 M dia metre Length - 280 M. Velocity - 5M/sec.
g)	Power Potential	Installed capacity - 1500 MW Annual energy generation-7447GWH in 50 percent mean year. Annual energy generation - 6700 GWH in 90 percent dependable year

The underground power house of internal dimensions 170m x20m x 40m would be located about 150 m below the natural surface level. The cavern will have an circular arched roof with concrete lining. The valve house cavern will be 115 m x 8 m x 17 m and is located u/s of machine hall cavern. The transformers and underground switchyard and tail race surge chamber caverns are located downstream of the power house cavern. The sizes being 170 m x 17m x 24m and 120 m x 12m x 32 m respectively. The rock will be strengthened by providing rock bolts at suitable spacing. Two numbers, 300 MT capacity, E.O.T. cranes, supported on rock at either end, will be provided in machine hall.

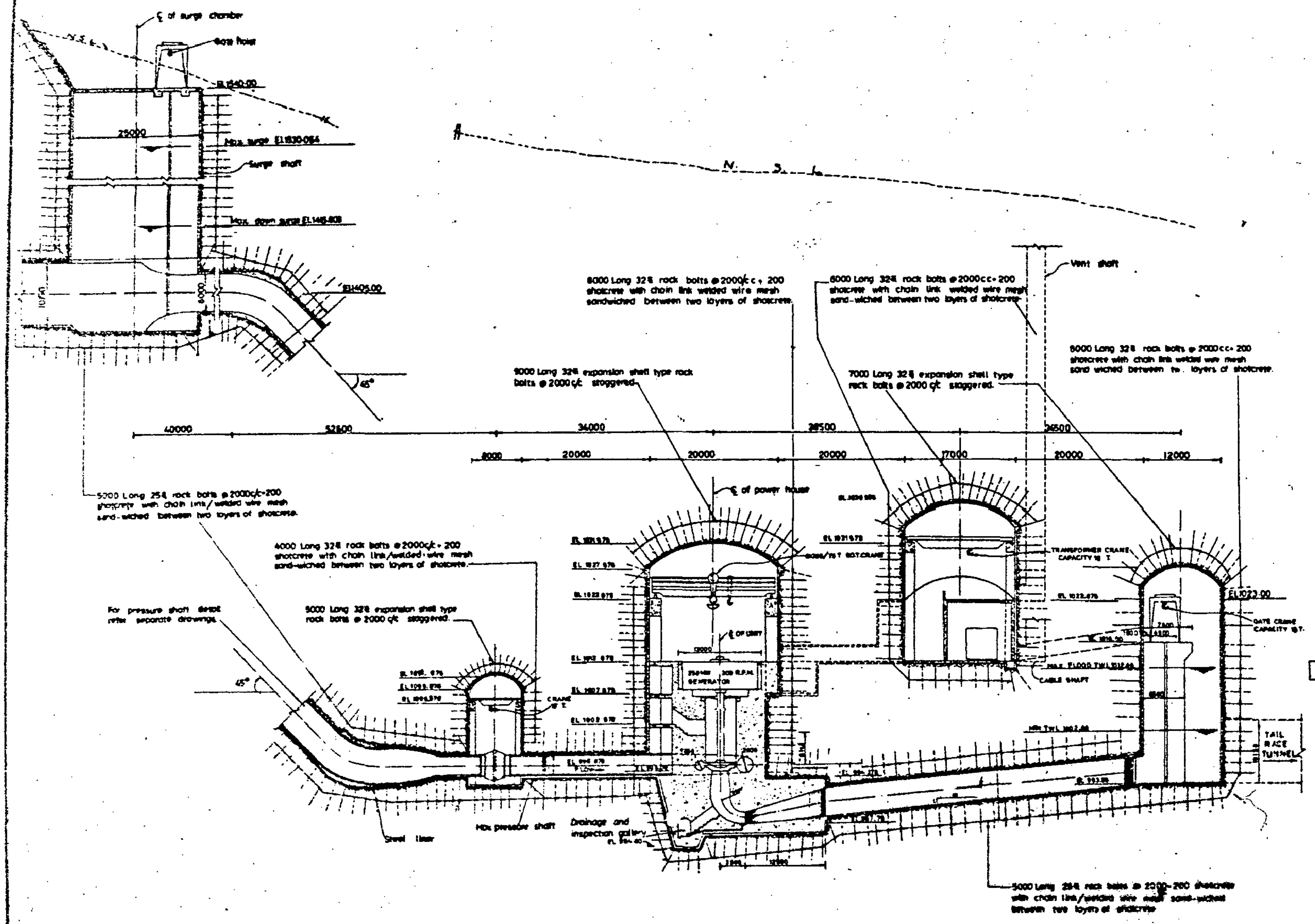
The capacity of cranes for valve house, transformer hall and tail race surge chamber will be 125T, 25T and 10T respectively.

The generator floor and service bay floor are proposed to be at same level. The auxiliary rooms and other service facilities will be located at one end of the power house.

The excavation of the machine hall shall be facilitated through the provision of two utility tunnels to approach the bottom of machine hall cavity. To approach the top of machine hall and also the top of transformer gallery and tail race chamber, an adit is proposed to be constructed with its portal at El $\pm$ 1028 m. This adit is to be used for the excavation of arch portion and to facilitate other construction activities.

The approach tunnel to power house and the transformer gallery will be D shaped 410 m long having a grade of 1 in 35. The transverse section of power house complex is shown in Fig.1.1.. .

The underground power house is located in foliated gneisses with about 130 m of rock cover and nearly 300 m inside from the river bank. There is no geologically better site available in the vicinity for the power house complex. The geological features along the water conductor system are highlighted in Fig.1.2.



NOTES

All dimensions are in millimetres and elevations in metres.

This drawing is based on C.W.C. drawings showing the details of cavities in plan and section.

This drawing showing the arrangement and disposition of different cavities for support details refer separate drawings.

SPECIFICATION DRAWING NOT TO BE USED FOR CONSTRUCTION

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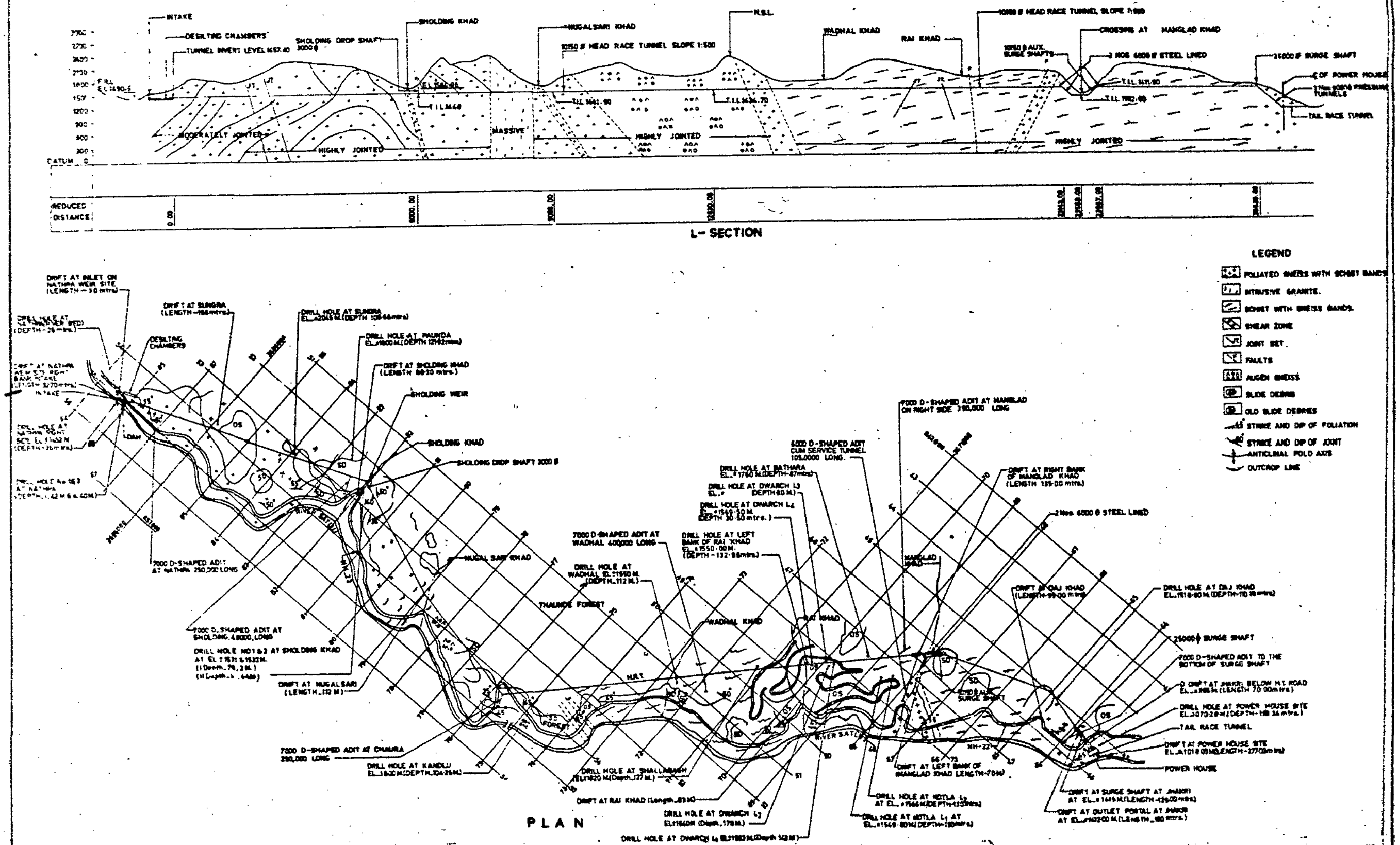
NATHPA JHAKRI POWER HOUSE
(Cross Section Thrugh ℄ of Unit)

FIG.1.1



NATHPA JHAKRI HYDEL PROJECT
(Geological Section Throgh ℄ of water conductor System)

FIG.1.2

CHAPTER - 2

REVIEW OF LITERATURE

2.1 UNDERGROUND POWER HOUSE STRUCTURE

2.1.1 Selection and Layout

Economy in construction cost, nature conservation and land protection aspects, reduced cost of penstock liner, reduced requirements for land aquisition, security considerations and feasibility with respect to site conditions are the important factors which may lead to the choice of underground power house in preference to the conventional surface power house. However, the factors, such as increase in cost of power house when sound geological formations are not available, additional cost of approach adits ; cable ducts, galleries, ventilation and air conditioning systems and additional downstream collection chamber, are to be given due consideration before ariving at a final decision.

In India, on cost basis, underground power houses have been found to be costlier and hence, have generally been adopted only at the locations where topographical features are such that adequate space for the layout of conventional surface type power house is not available. The first Indian underground power station was constructed across Konar River, known as Maithan Power Station (DVC), utilizing a head of 34.5 metres to generate 50.0 MW of power. Koyna Power Station (4 x 60 MW), Idukki (840 MW) and Chibro (240 MW) are other important structures of this type in the country. Bhaba Power House (120 MW) and Varahi Power House (460 MW) are at present under final stages of construction.

The reduced length of the water conductor system, in case of underground power stations, results in reduction of head losses thereby decreasing the operational cost to a considerable extent. The maintenance costs, on the other hand also become lower. The studies carried out in case of Iddikki Power Station (Kerala) have shown a considerable saving of cost by adopting underground power station.

The underground power stations are best suited in case of deep canyons and highly seismic areas. The cost estimates of underground power structures are generally likely to increase on account of less accurate geological predictions. The existence of sound rock is a basic necessity to select the underground power structure. Relative location of the penstock branches and the power house alignment depend upon the dip and strike of the geological formations and structures. The stress analysis around the cavity becomes little difficult specially in case of plastic rock. The expenditure incurred to provide suitable supporting system in such cases is considerable.

2.1.2 Types

On the basis of layout, the underground power stations are classified as :

Type	Features
i. Upstream station or Head Development	Location is close to intake, are suitable for low heads and high discharges.
ii. Downstream station or Tail Race Development	Short tailrace, are most suited for rugged terrain and utilize high heads.
iii. Intermediate Station Development	Long head and tail race tunnels

2.1.3 Components and Relative Layout :

An underground power station may comprise of some or all of the following components :

(i) Valve Chamber, (ii) Machine hall, (iii) Erection bay, (iv) Transformer hall, (v) Control room, (vi) Cable gallery, (vii) Access gallery or shaft, (viii) Ventilation gallery or shaft, (ix) Downstream surge chamber/Draft tube gate chamber.

The valves may be located either in a separate gallery upstream of the machine hall or in the machine hall itself (when the large span cavern is possible). Similarly, the transformers hall may be located underground or on the surface. However, all the underground layouts will comprise of an extensive system of caverns, galleries and shafts for proper utility, control, access and maintenance. Stress concentrations, occuring in the walls and rock pillars separating various components, may exceed permissible limits. The stress gradient, developed consequent upon the excavation of cavern, may result in rock falls. Thus, the location, shape, alignment and size of the cavities have to be carefully planned. Finite Element Analysis is an important and useful tool to carryout such study. The layout selected should be such that the best possible results are obtained.

2.1.4 Types of Layouts

The various possible layouts for the underground station are :

(i) Parallel Hall Arrangement : In this type, the valve house machine hall and transformer hall are essentially IIlel to each other (Koyna Power House, India). The valve chamber and machine hall may be combined together in case the good quality rock is available. Limited dimensions of machine hall cavern and safety in case of fire hazards or pipe bursts can be quoted as chief advantages of this type. Large excavation volumes and requirements of separate cranes etc. are the chief disadvantages.

(ii) Linear Arrangement : This type has generating sets, erection bay and transformers in one large cavity. A common access is thus possible and single gantry crane is required for maintenance and erection. This arrangement also eliminates separate busbar galleries Chibbro Power House in Yamuna Valley is an example of this type.

(iii) Composite Arrangement : It is a combination of the two types described earlier. The valves (upstream), generators and transformers (downstream) are all located in single large cavity. In Idduki Project, the valve chamber has been separated from machine and transformer halls.

(iv) Perpendicular and Skew Arrangements : In these types the transformers are arranged with certain variation at right angles or skew to the machine hall.

(v) Cluster or Cylindrical Layout comprises of domed roof type structure. The generating units are arranged around the circumference of cylindrical cavity.

The advantages of cluster type layout, over the conventional type, include mainly the low ratio of excavation (m^3/kW), the smaller overall surface to be reinforced with rock bolts and lower stresses in the dome and the cylindrical walls of the cavern. The slower progress in excavation and careful planning of construction operations can be considered as some of the disadvantages.

Thus, layout of the underground power house is an important decision making assignment for the design engineer. It should be finalized in such a way that overall safety and economy are achieved without hampering the functional requirements. The width of rock pillar between two adjacent parallel caverns, the optimum rise to span ratio of the roof arch, the orientation of the cavern with respect to the strike of the bed rock etc. are some of the important areas which need careful attention. The linear orientation should preferably be normal to the strike of rock foliations as this is considered to be, structurally, the most sound and favourable position. As regards the shape of caverns, while functionally vertical walls give maximum working space, structurally oval or elliptical cavities are better.

2.2 ROCK MASS, BEHAVIOUR AND STRENGTH PROPERTIES

2.2.1 Rock Mass Properties

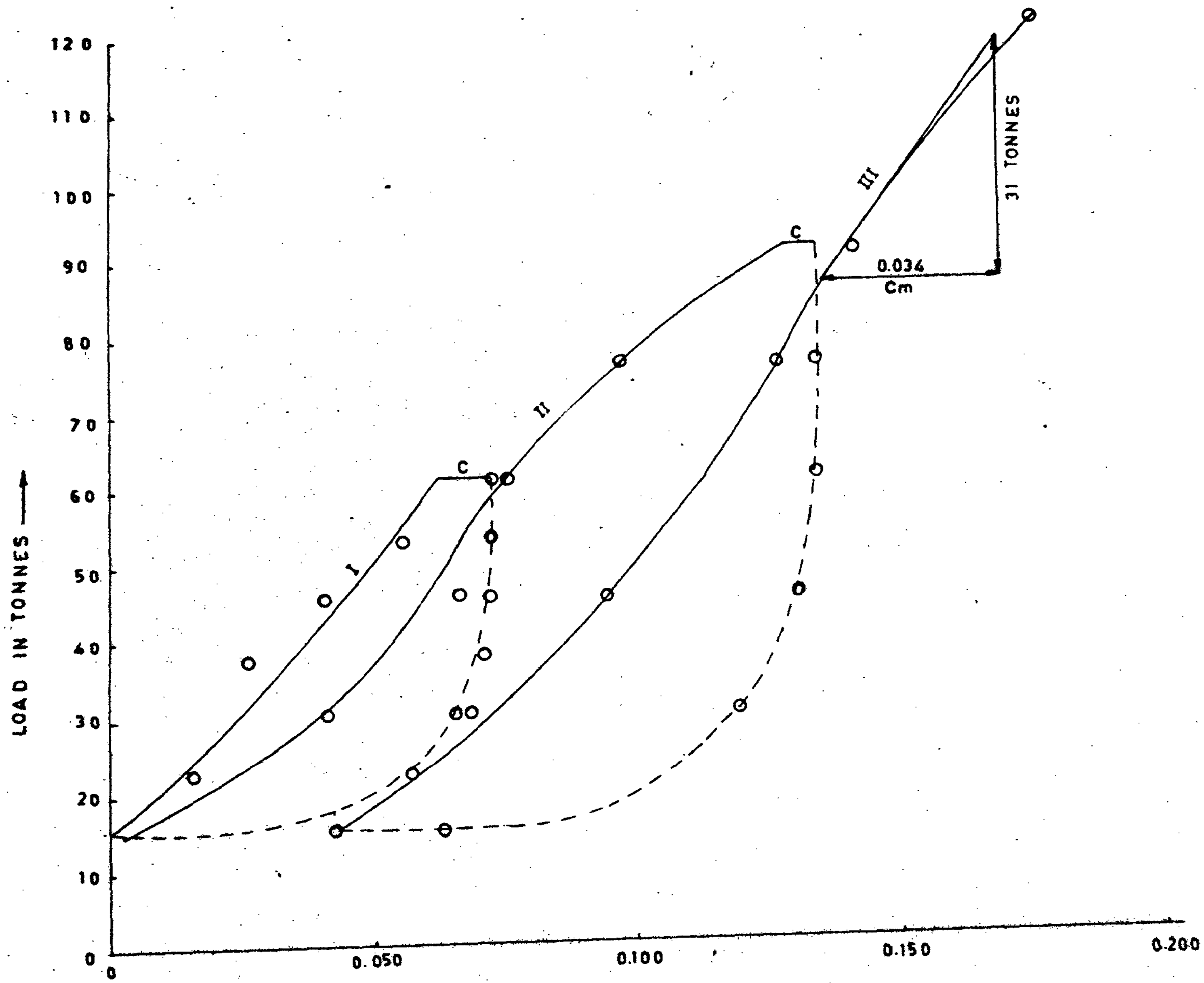
' In Civil engineering terms, rock may be defined as hard and solid formations of the earths crust. In broad terms the rock may be defined as any naturally occuring aggregate of minerals or mass of mineral matter, whether or not coherent, constituting an essential part of the earth's crust. The rock mass is the insitu rock, made up of rock substance and structural discontinuities'[7].

The rock mass is essentially anisotropic in character. The apparent breaking strength of anisotropic rock is strongly dependent upon the orientation and direction of loading with respect to pre-existing strong or weak directions. The static modulus of elasticity of the rock material 'E' and the associated Poisson's ratio 'ν' are the main strength properties. The examination of load-strain diagram for the direction parallel to the direction of load application reveals that the strain seldom varies linearly with the load. Instead, a curvilinear plot is obtained. The moudlus of elasticity is obtained by tracing the secant to the curve between the point of origin of stress and a point corresponding to test load. Sometimes the value at the origin of stress strain curve is used to define the modulus. The numerical value of the modulus is a function of maximum applied stress, rate of loading and sequence of successive loadings and unloadings etc. The load deformation curve in case of Jhakri Power House drift is shown in Figure 2.1.

It has been established through experiments that slow loading gives higher value of E as compared to rapid loading and the variation is about 30 percent in case of sandstone.

Seismic modulus of elasticity 'E_s' and modulus of rigidity 'G_s' are other rock parameters used in structural analysis. The parameters are obtained from the experimental determination of velocity of P and S waves (Seismic shock or ultrasonic waves). There is no practical method of measuring G (static) for the rock samples. However, E_s is obtained from the results of single pulse method using quartz crystals in the laboratory.

Comparison of static and sonic measurements of E show that the values obtained by the static techniques are in general lower than those obtained by sonic measurements. This can be



DEFORMATION (CM)

SCALE 1:100

STRESS DEFORMATION CVRVE FOR ROCK
(Vertical plate bearing test of Jhakri Drift)

FIG.2.1

attributed to the presence of fractures, cracks etc. which increase the static yielding by their deformation, the sonic measurements being least affected. The values of E_s/E_o for various types of rocks as given by Clark[1] are given in the table below :

Sl. No.	Rock Type	E_s/E_o	Sl. No.	Rock Type	E_s/E_o
1.	Chelsectonic Limestone	0.85	6.	Biotite Schist	1.48
2.	Oelitic Limestone	1.18	7.	Limestone	1.70-1.86
3.	Quartzose Shale	1.33	8.	Quartzose Phyllite	2.45
4.	Monzonite Porphyry	1.36	9.	Granite(Slightly) altered	2.75
5.	Quartz Diorite	1.42	10.	Graphite Phyllite	2.78

It has also been observed that during the uniaxial compression tests that deformation perpendicular to the direction of stress increases rather more than longitudinal strain, for the stresses above 50 percent of the fracture strength, resulting in decrease in the value of Poisson's number $m = 1/\nu$. The usual values of Poisson's ratio vary between 0.2 to 0.3.

The tensile strength of rock mass is another important property used in structural analysis. Bending test, Brazilian test, Cylindrical tests etc. are various laboratory tests used for its determination. The rock can either fail by brittle fracture or by visco plastic deformations, the latter case being more frequent. According to fumagalli, the brittle type of failure occurs whenever the classical ideal principal stress equals the limiting value of uniaxial tensile strength.

2.2.2 Classification of Rock Mass

The rock masses are classified in a different way than the rock materials. Rock quality designation is one of the significant parameters for rock classification. The properties, rather than the thickness of the filling material in the fissures and faults, are of importance to a civil engineer. The shear strength of the filling material may be higher than the material itself. This may happen when there are geological slips, sufficient to cause considerable relative displacement, at the lips of the geological faults.

2.2.3 Residual Stresses

The experience on existing underground structures shows that the rock mass, through which the galleries are excavated, is stressed. These stresses, which exist in the virgin rock before excavation, are called as virgin stresses or residual stresses. The first conclusion in this regard was made by Heim, a Swiss Geologist. It was assumed that there exists a vertical stress component 'σ_v' which is related to the weight of the rock overburden and is proportional to it. In addition, a horizontal component 'σ_h' also exists, most probably of the same magnitude as the vertical one. Heim related these residual stresses to gigantic forces which, many million years ago, caused mountain ridges of 'secondary' and 'tertiary' geological periods to be lifted.

Terzaghi related the residual stresses on the basis of theory of elasticity. As the horizontal expansion of rock mass, based on Poisson's ratio ν, is hindered by the neighbouring rock mass (at great depth), the lateral displacement in horizontal direction along a vertical plane is nil and the stresses are produced opposing lateral expansion.

The hydrostatic distribution of stress, as given by Pascal, relates the local pressure $p = \gamma . y$ where γ is the specific weight of the mass and y is the depth under the surface. This distribution, mathematically, is linked to the absence of any shear stress, as in case of static rock mass, there exists a tendency for the shear stresses to be progressively relieved giving a hydrostatic stress distribution.

2.2.4 Effective Modulus of Elasticity and Effective Poisson's Ratio

The relation between vertical stress σ_v and horizontal stress σ_h at a depth in rock masses is given by $K = \sigma_H / \sigma_V$.

Terzaghi related the factor 'K' to the Poisson's ratio considering the three co-ordinate system.

$$\epsilon_x = \frac{1}{E} (\sigma_x - \nu(\sigma_y + \sigma_z))$$

$$\epsilon_y = \frac{1}{E} (\sigma_y - \nu(\sigma_z + \sigma_x))$$

$\epsilon_z = \frac{1}{E} (\sigma_z - \nu(\sigma_x + \sigma_y)$

where E is the modulus of elasticity and ν is the Poisson's ratio of the rock mass. If the stresses in horizontal plane are assumed to be equal and the displacements in the horizontal direction are equal to zero, then

$$\epsilon_z = 0 \qquad \text{and} \quad 0 = \sigma_y - \nu(\sigma_z + \sigma_x)$$

$$\sigma_y = \sigma_z = \frac{\nu}{1-\nu}\sigma_x \quad \text{and} \quad K = \frac{\nu}{1-\nu}$$

For $\nu = \frac{1}{5}$ to 1/3, we get $\sigma_y = \sigma_z = \frac{\sigma_x}{4}$ to $\frac{\sigma_x}{2}$ or K varies from 0.25 to 0.50 with K = 0.3 as the most probable value.

For a two dimensional stress case $\sigma_z = 0$.

$\therefore \ \epsilon_x = \frac{1}{E} (\sigma_x - \nu(\sigma_y))$ and $y = 0$. $\therefore \frac{1}{E} (\sigma_y - \nu\ \sigma_x) = 0$.

Thus $\sigma_y = \nu\ \sigma_x$ and $K = \nu$.

$$\therefore \quad \epsilon_x = \frac{1}{E} (\sigma_x - \sigma_x \nu^2) \qquad \text{and} \quad E = \frac{\sigma_x}{\epsilon_x} (1 - \nu^2)$$

However, the ratio $K = \frac{\nu}{1-\nu}$ with the usual value of $\nu = 0.3$ to 0.20 or $\nu = 0.25$, accepted for the rock material, is contradictory to the actual measurements which conform to Heim's hypothesis i.e. $\sigma_z = \sigma_y = \sigma_z$ and K = 1.

Further, due to the presence of fissures and fractures in the rock mass, E_{eff}. is not equal to the modulus E of the rock material. Similarly, the effective Poisson's ratio is different from that of rock material. The relation between effective and observed values was developed by Charles Jaeger in 1966 and is as under :

$$\frac{E_{eff}}{E} = \frac{\nu}{\nu_{eff}} = \nu \frac{(\sigma_1 + \sigma_3)}{\sigma_3}$$

Thus, in cases where the basic equation of Heim is correct and $\sigma_1 = \sigma_v = \sigma_h = \sigma_3$.

$E_{eff} = 2\nu E$, with $\nu = 0.25$ we get $E_{eff} = E/2$

Similarly, $\nu_{eff} = 0.25 \times 2 = 0.50$, as required by Heim's hypothesis for $\sigma_v = \sigma_h$ and $\epsilon_1 = \epsilon_3 = 0$.

2.2.5 Failure of Rock Mass and Stress Distribution

The most common types of failures for the rock mass are shear fracture, viscoplastic deformation and brittle fracture. In the case studies, for many dam ruptures, it has been found that the failure occurred on account of excessive pressure (compression). The compression stresses shear stress and moments are transferred from the overground structures to the foundation rock and in case of underground caverns, redistribution of stresses takes place.

Brittle fracture occurs whenever the classical ideal principal stress equals the limiting value of uniaxial tensile strength. In rock mass the brittle type of fracture normally occurs at the crystal layers where the isotropic triaxial compression stress is usually less intense.

The cohesion bonds are not assisted by numerous hyperstatic connections within the rock mass, owing to their stiffness, and fail abruptly as the elastic deformation limits are reached.

'The failure of arch dam abutments falls under the category of visco-plastic deformation. As the load increases, the deformation process affects the resisting structure by successive frontiers located ever deeper within the rock and farther from the force application plane. On the deeper movement of frontiers, the cohesion bonds in the receding area gradually decrease while resisting stresses due to internal friction of the material are called upon to mutually co-operate through a gradual redistribution of stresses. At constant loading the deformation speed decreases and finally becomes nil as long as state of static equilibrium is possible. When collapse occurs, it is usually a global phenomenon'.[2]

The stress distribution inside a strained-stressed rock mass can by estimated with the following theories:

i) Classical homogeneous, isotropic, elastic half space equations.

ii) Fissured rock treatment

iii) Classic rock treatment

iv) Modern stress-strain analysis.

For the first method, the initial calculation of stresses around a cavity, located deep in rock mass, is a typical example of the treatment of rock mass as homogeneous, isotropic, elastic space and solution by the equations of strength of material. The other quotable example is that of a half space loaded by the forces at the surface. The equations of Boussineq and Cerruti solve most of the problems. The Prandtl-Terzaghi theory, an extension of the former, is also used.

The estimation of stress distribution in fissured rock can be done with Muller and Pochar theory, based on the direction, continuity and spacing of the main families of fissures. The tensile and shear strengths of the rock, at any point in the rock mass in any direction, depend upon these fissures.

The elastic rock theory does not assume the rock mass to be capable of shear and tensile strength. It does not transmit the stresses across a fissure, even to the smallest extent. The clastic mass consists of blocks of rock or other materials piled on top of each other, with no possibility of shear or tensile stress transmission in between. The only possible transmission is in case of compressive stresses.

The modern stress-strain analysis is based on the use of computer methods and experimental methods. These include the finite element method and the photoelastic method. The methods developed by Muller, Zienkiewicz,Trollope and others are worth mentioning.

2.3 GEOLOGICAL CHARACTERISTICS OF ROCK MASS

The properties of rock mass can be categorised as macroscale and microscale. The former being related to the components and the latter to the substance. The inferior properties of a rock mass affect underground construction operations in rock mass and also the performance of the structures built over or underground.

2.3.1 Mechanical and Structural Defects

The mechanical or structural defects essentially influence the strength of rock mass adversely. These defects consist of more or less closely spaced fractures and other planes of weakness in rock e.g.fractures, cracks, haircracks, fissures, bedding planes, laminations, schistosity, parting, stratifications, joints, fault planes, crushed zones, folds, voids, cavities, seams, interbeds of weak and plastically unstable rocks, aquifers, clays, shales, ancient slip planes and other weaknesses.

These potential planes and zones of weakness in the rock are artificially created weaknesses in rock sequences brought about by blasting operations; possibility of influx of water in and seepage through the fissures, cracks, joints and fault zones in rock and possibly by some geological or tectonic factors. These affect the stability of underground openings and also the stability and excavation of slopes made in rock. Further, these also provide passageway for the flow of water underground.

The fissured rock strata are unsuitable for the construction of wide underground openings. Fractures occur in rock mass generally as a result of blasting operations. Much of the excessive leakage of water into an underground cavern takes place through the fault zones. The pattern and distribution of joints affects the extent to which the sides, walls and roofs of excavations must be supported. Joints are respondent to the applied static and dynamic loads and are one of the major causes of the excessive overbreak. The joints also cause potential rock slide hazards and also create seepage trouble in underground works. Sand and other kinds of gauges tend to flow into an underground opening as the excavation is worked through a fault zone. Joints subdivide the rock mass into individual blocks to form a three dimensional network. The jointed rock behaves as uncemented, cohesionless aggregate of cuboid-blocks, somewhat comparable to closely fitted blocks in dry masonry wall. The term imbrication means the lying of blocks, lapped over each other in rock strata, in a regular order so as to form a imbricate-pattern. The more imbricated the blocks, the higher is the shear strength of the rock mass.

2.3.2 Effect of Jointing

The effect of jointing in rock on its strength was defined in terms of coefficient of weakness by Popor.[7] It is the ratio of cohesive force in the weak rock to the cohesive force in the same rock with no visible traces of such weakness. The following table gives a description of the various rock conditions and corresponding coefficient of weakness.

Table 2.1: Weakness Coefficient in Rock as a Function of Jointing Characteristics

Description	Coefficient of weakness	
	Limiting value	Average value
Dense network of fractures in all directions of layered rocks to individual-uncemented blocks	0.0-0.01	0.0005
Dense network of open fractures in all directions	0.001-0.02	0.005
Dense Jointing	0.01-0.04	0.02
Jointing above average	0.04-0.08	0.06
Average jointing(open and closed fractures every 20-30 cms)	0.08-0.12	0.10
Rocks with below average Jointing	0.12-0.9	0.2
Network of deep joints every 30-50 cms; insignificant number of open fractures	0.4-0.6	0.5
Little jointed rocks, closed fractures	0.4-0.6	0.5
Microfractures almost absent	0.6-0.8	0.7
Monolithic rock with no sign of jointing	0.8-1.0	0.9

The application of loads, static or dynamic, in the faulted rock mass may lead to sliding of the mass along the fault plane. In seismic regions, structures built underground or over it, when built across faults may experience distress and/or total destruction if subsequent movement of fault would occur.

2.3.3 Structural Geology and Underground Structures

In underground structures in synclinal zones, the problem of water accumulation needs careful consideration. The surcharge pressure, imparted to the structure by the folded rock mass, has to be accounted for in the analysis where the situation warrants it. In case of anticlines, the surcharge pressure is concentrated at the sides whereas the middle portion is highly stressed in case of synclines.

The location, extent and distribution of sinkholes or cavities in limestone, gypsum and salty rock masses needs careful consideration of structural engineer. Large voids and great porosity of rock mean weaknesses and discontinuities requiring chemical injection, grouting or similar solutions.

The nature of rock mass and its complex structure necessitate specialized approach to the analysis, design and construction of structures in or on the rock. The strength properties of the rock can be improved, depending on the technical and economic feasibility, to impart stability to the structures built underground. However, analytical comprehension of the weaknesses such as stratification, fissures, joints etc. is a complex problem. Thus, an analytical treatment of stresses in underground rock medium alongwith a technologically advanced stress measurement insitu and testing of rock strengths are urgent necessity for the solution of complex problems related to underground planning, analysis, design and construction. In dealing with the rock structures, the following forces need to be considered.

i) Self weight of the structure
ii) Rock loads
iii) Geological forces brought about by tectonic processes and seismic action
iv) Hydrostatic uplift
v) Hydrostatic shear, induced by the load of seeping/impounded water.
vi) Seismic forces and
vii) Reactions brought about by the structures.

The basic concept in rock mechanics studies and analysis of underground structures lies in the consideration of rock and the structure as a functionally and statically coherent, integral entity. Thus, the indepth study and knowledge of geological and structural weaknesses and physical and mechanical properties of the rock mass is essential.

2.4 SCOPE OF ANALYSIS

2.4.1 General

The analysis of underground structures is a complex problem owing to the complex geology of the area in which these are located. The main objectives of such a study are :

(i) To select the best possible site

(ii) To establish the best orientation of the cavern based on the insitu stress patterns and geological formations.

(iii) To establish the best shape, size and spacing of cavities with the restricted limits imposed by the generating equipment to be housed in the cavern.

(iv) To estimate the extent and magnitude of deformations and the redistribution of stresses and to check the stability of the opening.

(v) To obtain the best system of supports, for the highly stressed zones, such as rock bolts, shotcreting, concrete lining and ribs etc.

(vi) To find the best possible sequence of excavation in order to minimize the deformations and avoid stress concentration zones.

2.4.2 Data Required

The data required to carryout such an analysis will include detailed geological information, depth of consolidated material or sound rock, rock cover, information about ground water, results of water pressure and grout intake tests, insitu modulus of deformation of rock, insitu primary stresses, mechanical and strength properties of rock mass and filling material in case of jointed or faulted rock mass, results of pull out tests on rock bolts and tests of installation of ground anchors, information about construction equipment, detailed layout of power house and other appropriate data.

2.5 INSITU STRESS AND UNDERGROUND CAVERNS

2.5.1 General

The steps in rational analysis of the underground excavation have been highlighted in para 2.4.

The insitu state of stress is related to almost all the steps listed therein. It is, therefore, necessary to determine the insitu stresses before taking up the assignment of analysis and design. The insitu stress measurements can be done in a simple and inexpensive manner using deep hole hydrofacturing and triaxial strain methods.

2.5.2 Location of Cavern

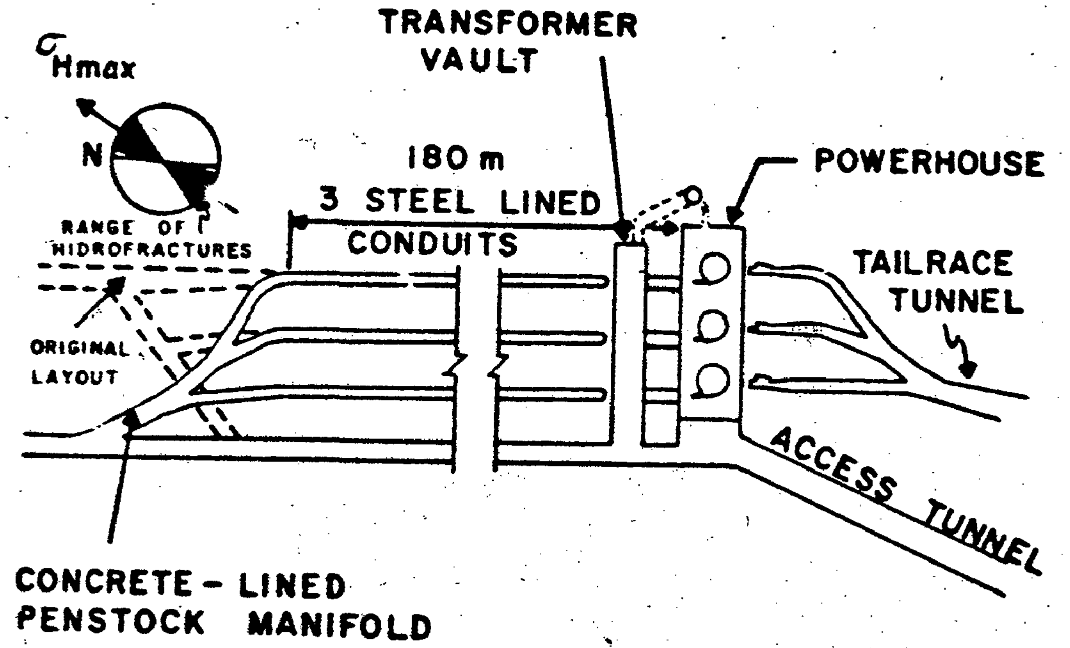
In case of hydroelectric development, the site for power house is relative to the location of the reservoir alignment of water conductor system, availability of land and the economic and feasibility studies. In site selection process, the nature of rock is very important with respect to the uniformity, strength and ease of excavation. Rock structures i.e. faults, joints etc. are also decisive factors, especially in case of near surface location of the cavern. The insitu state of stress is another crucial parameter. The state of stress can be used to arrive at the optimal depth of excavation within a given location. In case of near surface excavation, the vertical stress magnitudes are low, the horizontal stress being unpredictable needs to be measured.

The location of near surface excavations needs to be studied vis-a-vis the excavation geometry and functional requirements. The principal stresses increase with the increase in depth of opening (location). The rate of increase is usually not uniform among the stress components and may also vary from location to location. Thus, a deep seated excavation at several hundred metres below the surface may offer lower density joints but higher stresses will be encountered. While fewer decontinuities imply improved stability, the increase in stress magnitude may result in high stress concentration around the opening and possible localized failures and rock bursts.

2.5.3 Orientation of Cavern

The term orientation implies the direction of opening long axis. In shallow excavation the governing factors are distribution of natural fractures and bedding planes, the principal stress directions and magnitudes being equally important. However, in case of deeper excavation, less defects are often found and the state of stress becomes the sole deciding factor. The objective of orientation is to align the cavern in such a way that would minimize instability. In intact rock, instability implies failure due to excavation-induced stresses higher than rock strength, in densely jointed rock it implies slipping of blocks into the opening. The 'block' parameters are dependent upon the joint distribution whereas the normal and shear forces, developed along the planes of discontinuity, depend on the state of stress. 'Broch, an Norwegion engineer, recommends that near the surface the long axis should bisect the larger angle between two important joint sets, while in deep seated openings the length should be at 15° - 30° from the direction of major horizontal principal stress avoiding parallelism with a major joint set. The suggestion given by others recommend that the horizontal axis should be actually aligned with the major horizontal stress. This has been verified by Licoand Shi (1983) while carrying studies in case of Jinchuan Mine, Gicmsu Prevince, China.' 4

The another quotable example is of the Holms Hydro-Project power plant (Fig.2.2). The power house complex including machine hall, transformer hall and other accessories were first planned without regard to the insitu stresses in the granitic terrain. A series of measurements, by hydrofracturing stress measuring technique (useful in deep exploration holes)



HEIMS PROJECT POWER PLANT COMPLEX
ORIGINAL AND MODIFIED PENSTOCK MANIFOLD

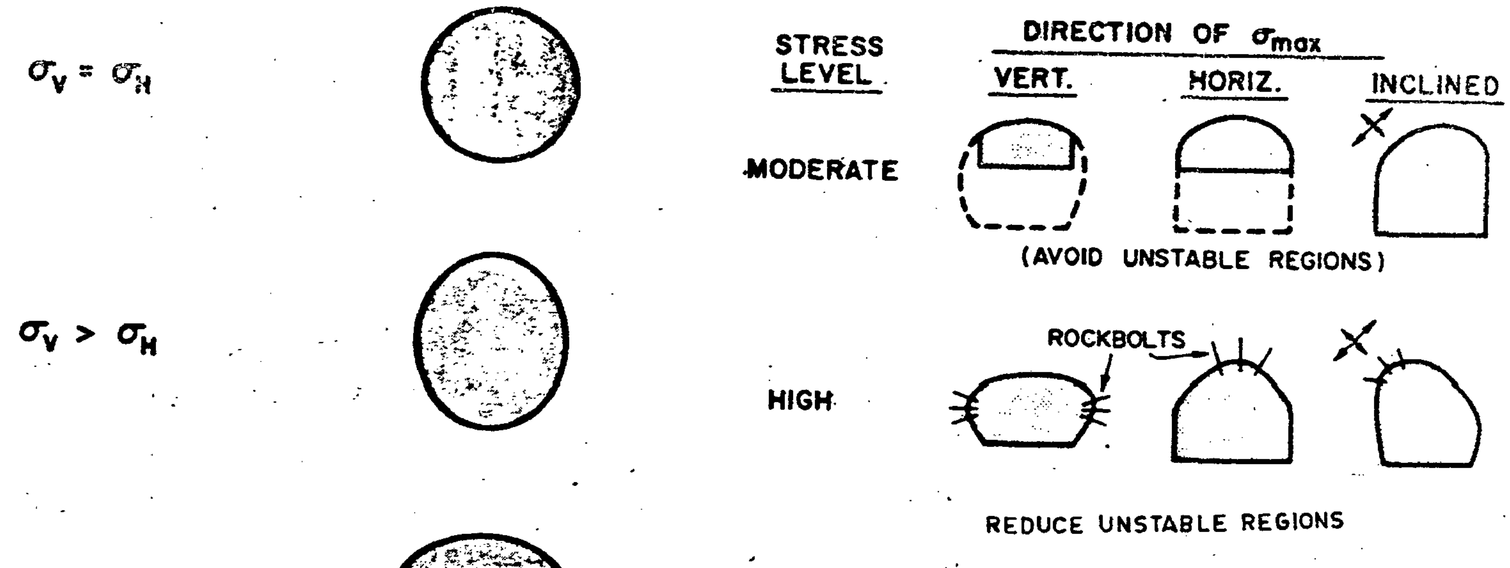
FIG 2.2

during final design stages, indicated the magnitude and direction of principal stresses. It was noted that since the water pressure of the penstock manifold was going to be equal or larger than σ_{Hmin} and since the strike of major vertical joint set was subparallel to that of σ_{Hmax}, there existed a possibility of some of the joints being opened by the flowing water resulting in major leaks. The design was modified by rotating the penstock manifold by some 90^{o} so as to minimize water losses and avert a failure.

2.5.4 Shape

The shape of underground excavation is a function of the utility, method of excavation and insitu state of stress. The shape selected should be such that it minimizes the stress concentrations and accumulation of stored strain energy, so as to optimize stability.

A properly conceived shape can save larger investments in reinforcement and supports and also lower the risk of failure around the opening. It has been established that the properly proportioned elliptical section minimizes the stress concentration. If the ratio between the long axis and the short axis of the ellipse is kept equal to the ratio between the two principal stresses, then the stress concentration around the cavern is uniform. The high stress concentrations are avoided and the stability is increased.



BEST THEORETICAL SHAPES

NORWEGIAN PRACTICE

RELATION BETWEEN INSITU STRESSES
AND SHAPE OF UNDERGROUNDOPENINGS

FIG. 23

The Norwegian approach to design of cavern shapes in granite is also based on the knowledge of insitu stresses. Under moderate insitu stress levels the principle is to design the shape that will distribute the stress evenly to avoid local stability problems. Under high stress levels, with large principal stress differentials, the approach is to shape the cavern so as to restrict the zones where stability problems are expected. Fig. 2.3 shows the best possible cavern shapes in relation to insitu stresses.

2.5.5 Dimensioning

Based on the stable shape, the stability of the cavern should theoretically be independent of size. However, practical experience shows that the physical condition of the cavern tends to deteriorate with the increase in size. The reason is that more structural defects are encountered with increased size. The behaviour of rock blocks in general depends upon the stresses around and along the contact surfaces which, inturn, are dependent upon the insitu stresses.

2.5.6 Excavation Sequence and Reinforcement

The best excavation sequence and reinforcement requirements, if any, are worked on the basis of numerical modelling using Finite Element Method or the Boundary Integral Method. In modelling the underground cavern, the boundary conditions include the insitu stress. Thus, the knowledge of insitu stress is very important in any numerical modelling.

2.5.7 Stress Measurements

Till recently, the conventional overcoring methods such as borehole deformation, door stopper and triaxial strain cells etc. were in use for insitu stress measurements. The main limitation of these methods were the extent of depth to which they could be successfully applied. Pilot tunnels were necessary to access the area in which investigations were required. There was usually considerable delay in obtaining the stress values. Sometimes, pilot tunnel excavation was not found feasible owing to farther distance of area from mountain top or slopes. The hydrofracturing method of stress measurements is an advancement in this field and requires no overcoring. The method is limited in depth of measurement only by the length of borehole. The technique consists of sealing-off a section of a borehole at the required depth by means of two rubber packers and pressurising, hydraulically, the packed off segment.

When the 'breakdown pressure' is reached, the rock surrounding the boreholes fails in tension and develops a fracture. This fracture can be extended away from the hole by continuous pumping. When the pumps are shut-off with the hydraulic circuit kept closed, a 'shut in' pressure is recorded. This is the pressure necessary to keep the fracture open. The breakdown and shutin pressures can be related to the prevailing insitu stresses. A commercial oriented impression packer is finally used to determine the exact direction and inclination of the hydrofracture. Thus, both the magnitude and direction of principal stresses can be evaluated.

The newly developed wireline hydrofracturing technique eliminates the need for a drill rig during testing, shortens testing time and considerably lowers the cost. If necessary overcoring tests can also be conducted later on, when the pilot or access tunnels are excavated as a part of construction sequence, to provide a check on the hydrofracturing results and increase the confidence in stress boundary conditions used in design.

Another latest technique isthe improved version of triaxial strain cell technique. It involves three strain gage rossettes which are cemented to the wall of a small pilot hole at the bottom of a NX (76 mm) exploratory size hole. From the strain recordings in the nine presented gages of rossettes, as the pilot hole is overcored by continuing the NX hole drilling, the triaxial strain tensor in the rock can be determined. The improved version consists of drilling concentric pilot holes, positioning the strain cell probe, cementing it under water, and overcoring it at depths reaching 500 m. The advantage of the triaxial cell is that it can yield the complete stress tensor.

2.6 ANALYSIS AND STABILITY OF STRUCTURE IN ROCK MASSES

2.6.1 General

The importance of insitu stresses in analysis of underground structures has already been discussed in previous paras. The stress pattern around the opening is altered after the excavation of structure. The amount and the depth of this alteration are influenced by blasting and the local stress produced by the shape of opening. The test results of Talobre (1957) indicated that the depth over which the stresses are altered extends to about overhalf the width of excavation, based on the assumption that the length of the excavation is several times greater than the width of the excavation.

For the reasonable analysis, the rock is treated as a homogeneous material and making use of the exact elastic properties the calculation of stresses in rock mass, weakened by openings, can be made. These assumptions are however, seldom met in practice.

The following points, however, need particular attention of the designer.

(i) The usual depths in underground rock engineering are too small for Heims pressure distribution at great depth.

(ii) Thermal and other factors may transform the rock into a plastic state i.e. gneiss and slates etc.

(iii) At usual depths, there is a differences between the magnitude of vertical and horizontal stresses resulting in shear stresses.

(iv) The observations show that the vertical stresses are not the largest.

(v) Stratification of rocks result in development of inclined patterns.

(vi) Insitu stresses in rock depend upon tectonic activity of the earth's crest.

Thus, in order to specify a complete gravitational field, it must be assumed that

(i) The rock is linear elastic, isotropic and homogeneous.

(ii) The lateral rockconstraintor confinement is complete.

(iii) There are no stresses of tectonic origin such as those accompanying folds, shrinkage or other distortions of earth's crust.

In some instances, these tectonic stresses, together with effects of inelasticity, inhomogeneity, anisotropy etc. are known to affect the gravitational stress field considerably. The largest principal stress may not always be vertical as a consequence of tectonic stresses. In the Shasy Mountain Power Scheme in Australia, the horizontal stresses are about 20 percent higher than the vertical stresses because of tectonic stresses.

2.6.2 The Stress Fields

The term refers to the primary stresses and according to Obert and Duvall, three kinds of stress fields in rock may exist.

(i) A uniaxial stress field, in which the rock is subjected to compressive or tensile stresses in one direction only. This is the case of shallow depth or near the vertical face structures.

(ii) A triaxial stress field, in which the rock is subjected to compressive or tensile stresses in two mutually, perpendicular directions. This state of stress condition would be encountered at great depth over a vide range of depth, depending upon type of rock, and at a depth of approximately 1000 m or more.

(iii) The hydrostatic stress field in which the rock medium is subjected to equal stresses in three mutually perpendicular directions. Such a condition may be encountered at great depth and in semiviscous or plastic rocks.

For a rock mass with certain characteristics of yield stress, unit weight and Poisson's ratio, there is a limiting depth above which the lateral stresses may be computed on the basis of theory of elasticity, but below which, the horizontal principal stresses σ_2 and σ_3 may be derived from a plastic yield criterion.

$$\sigma_2 = \sigma_3 = \sigma_1 - \sigma_y = \gamma . h - \sigma_y$$

where $\sigma_1 = \gamma h$ and σ_y = yield stress

There is no real justification for assuming $\sigma_h = K.\sigma_v$ as any one geological event could have altered the horizontal stress considerably from the theoretical value. Faulting, folding and other geological processes bring about certain changes in the magnitude and direction of principal stresses. The geological processes in rocks and their weathering would also alter the initial stress field. Practically, a reliable stress field can be determined only by insitu measurements.

2.6.3 Plane Stress and Plane Strain Cases

In a plane stress condition, all stresses induced in the rock mass around a opening act in the cross-sectional planes of the long structure. Thus perpendicular to the cross-sectional slice areas, no stresses are transferred. The primary stresses, however, are in an ideal, undisturbed medium of rock. After making a cavity in the rock, vertical and horizontal forces around the opening are set free, which, prior to the excavation were carried by the rock. Thus, in the post excavation stage, the pressure of the overburden rock must bypass the opening, being transferred through the adjacent rock. The rock around the opening now begins to deform consequent upon loosing its support.

The underground opening brings about a lateral squeeze of vertical force lines, resulting in large stress concentration at the abutments or springings of vault. Also, between the opening vault and the natural arching of rock, tension zones, are formed at the crown and invert of the vault.

The changed stress conditions are termed as secondary stresses. In the transition stage i.e. from primary to secondary, elastic and plastic deformations of the rock occur. The main reasons for inducing the secondary stress conditions include loosening of rock mass around cavities, weight of overburden rock mass and tectonic forces and volumetric expansion of the rock mass by thermal effects or by swelling brought about by the action of physical or physiochemical processes.

These secondary stress conditions depending upon the kind and properties of rock, necessitate the use of different construction methods. All the conditions mentioned in previous paras may also occur simultaneously. The rock mass may be sound, pseudo solid, soft and weathered rock. Sound rock transfers externally applied loads by beam action. The friction developed during the mass displacements transfers the load in loose rock or soil.

The stability of underground opening lies in prevention of intrusion of rock into the opening. The load acting on the supports, provided for prevention, is termed as rock pressure.

When the induced stresses remain below static limit, the analysis can be based on theory of **elasticity**. The assumption that the inhomogeneity and anisotropy are insignificant at large pressures of the rock does not hold good. This is so. because the post excavation period is comprised of reduction or relaxation of the stresses in the rock mass.

The secondary stress conditions in the rock mass by means of theory of elasticity are based on the assumption of ideally elastic, homogeneous and isotropic medium, validity of Hooke's law and the larger thickness of the overburden as compared to the cavern cross-section dimensions such that σ_v and σ_h can be considered constant within the influence zone of the opening.

The above mentioned assumptions are valid to certain extent in hard rock with no planes of separation. In case of layered and schistose rocks, these do not hold good.

Since, the stresses acting perpendicularly to the cross sectional plane are usually not of much interest in the secondary stresses analysis of circular tunnels, therefore, for purpose of simplicity, tunnel problems in rock are analysed under the assumption of plane strain.

Calculations of stresses in rock mass, weakened by openings and cavities require the use of theory of elasticity and theory of plasticity. The radial, tangential and shear stresses are given by the theory of elasticity as :

$$\sigma_r = \frac{\sigma_v}{2}\{(1+K)(1-a^2)+(1-\lambda_o)(1+3a^4-4a^2)\cos 2\omega\}$$

$$\sigma_t = \frac{\sigma_v}{2}\{(1+K)(1+a^2)-(1-\lambda_o)(1+3a^4)\cos 2\omega\}$$

$$\gamma t = -\frac{\sigma_v}{2}(1-K)(1-3a^4+2a^2).\sin 2\omega$$

where

$\sigma_r = \gamma.h$ = vertical overburden stress

γ = Unit weight of rock

h = thickness of overburden

r_i = inside radius of opening

r = variable radius

ω = amplitude = angle between vertical axis and radius r.

$K = \frac{1}{m-1} = \frac{\mu}{1-\mu} = \frac{\sigma_h}{\sigma_v}$ = rock lateral pressure coefficient at rest.

$a = r_1/r$ for brevity.

The problems of long and uniform structures in rock mass whose geometry and loading do not vary significantly in longitudinal direction are referred to as plane strain cases. In contrast to this, when the structure is characterised by large dimensions in X-Y plane and very small dimension in Z direction, the case is analysed as plane strain case.

CHAPTER-3

METHODS OF ANALYSIS

The analysis of structures in rock mass can be done with the help of either ' Physical Models' or the 'Mathematical Models'. The physical models such as Photo-Elastic Analysis require special laboratory facilities and expertise and are generally expensive. The mathematical models are based on either the principles of continnum mechanics and the theories of elasticity and plasticity or on the mechanics of discontinuous media, wherein, the equilibrium forces are maintained between a series of rigid blocks. The former approach is generally adopted unless physical situation specially warrants the use of latter.

The basis of the study of theory of elasticity is the equations of infinitesimal equilibrium, compatability and the constitutive laws for the material. In the past one decade, the application of numerical techniques to soil and rock mechanics problems have become more popular. Finite Element Method and Finite Difference Method are the most popular amongst these.

The arbitrary stress and boundary conditions in case of underground openings are usually taken as -

i) Surface of the excavation to be stress free.

ii) At a large distance from the openings either the displacements induced by the openings are zero or the perturbation in the original stress field is zero.

The problem can be solved by using any of the following methods -

i) Finite element method.

ii) Dynamic relaxation method

iii) Boundary element method

iv) Displacement discontinuity method

The limitations associated with each method should be carefully observed while applying it for the solution of a particular problem. The stress field around the opening can be obtained by -

i) Calculating the perturbation from the p stress field due to the opening and adding it to the original stress field.

ii) Applying the total stresses at a distance from the opening and calculating the total stress field in one operation.

The observations have shown that the displacement values given by the first method are similar to the actual measured values. In this work the scope of the study has been limited to the use of Finite Element Method to find the stress distribution around the opening.

3.1 FINITE ELEMENT METHOD

The method is based on principle of discretization. In simple terms, this can be defined as a procedure in which a complex problem of large extent is divided or discretized into smaller equivalent units or components. The method is based on the discretization of the physical body or continnum that constitutes the system. The continnum is divided into a equivalent system of smaller continua, called as finite elements. The intersections of the nodal lines separating the elements are called nodal points.

The basic characteristic of Finite Element Method is that finite elements are analysed and treated separately one by one. Each element is assigned its physical or constitutive properties and its property or stiffness equations are formulated. Subsequently, the elements are assembled to obtain the equations for the total structure. The assembly procedure is totally mechanical and involves putting together the element equations by observing certain conditions such as compatability requirements.

3.1.1 Isoparametric Elements

The use of isoparametric elements in finite element formulations offers a number of advantages. These include efficient integrations and differentiations and handling of curved and arbitrary geometrical shapes. It has been observed that accuracy is achieved by the use of fewer complex elements than using a large number of simple elements.

3.1.2 Formulation of the Problem

The problem can either be formulated by choosing displacements as primary unknowns, with the stress being determined from the calculated displacement field. Alternatively, the stresses can be chosen as primary unknowns. The former is known as displacement method and the latter as equilibrium method. In some models, both the displacements and stresses are employed as the primary unknowns and the method is known as mixed or hybrid method. The problems in geotechnical engineering are generally solved by using displacement method. The number and bandwidth of the final stiffness equations in displacement method are smaller than those produced by other methods. Moreover, it is relatively easier to establish approximation function to satisfy compatability requirements. However, the displacement formulation can be more sensitive to variation in problem parameters i.e. geometry and material properties. The steps in Finite Element Method analysis include (i) Discretization of structure (ii) Selection of approximation functions (iii) derivation of element equations (iv) Assembling of elemental properties to form global equations and (v) computation of primary and secondary quantities.

The finite element method has some restrictions for use in geomechanic applications, arising from the necessity to set the displacements in the area remote from the opening equal to zero. The large portion of Finite Element Mesh is used to model the above condition. The development of infinite elements, which include in their formulation a decay term and coupling the finite and boundary element techniques, have overcome this difficulty. The formulation of problem for analysis of stresses around the caverns has been discussed in Chapter- 4.

3.2 SPECIAL CASES IN FEM ANALYSIS

In practice, the assumptions of linear, isotropic and homogeneous rock mass are seldom met with. The deformation of the rock mass, consequent upon the excavation of cavity, is dependent upon the following factors -

i) Rock defects, ii) Petrographic structure, (iii) Geological structure, iv) Degree of weathering and alteration of rock mass, v) strength and deformation properties of rock mass, vi) Anisostropic rock behaviour, (vii) Magnitude and direction of insitu stresses.

These factors affect the stress deformation behaviour of rock mass. The problem of analysis,thus,becomes complex in nature due to non-linear rock mass behaviour. The finite element method has proved successful in approximating the effects of many of these factors. In the recent past, many non-linear rock mass problems have been analysed successfully with finite element method.

3.2.1 Non Uniform Material Properties

As discussed earlier in Chapter-2, the structural defects like joints, fractures fissures etc. render the behaviour of the rock mass as non-linear. A rock mass, where the extent of such defects is small i.e. with closely spaced joints, can be approximated by a solid continuum, with modified strength properties. If the extent of defects is large, each individual block must be treated as separate unit within the surrounding rock mass. The joints render the rock mass highly succeptible to tension. The load transfer method developed by Goodman, Taylor and Brekkie is very useful for the analysis of structures in jointed rock mass. In this method, the stress deformation characteristics of the joints are approximated by normal and shear stiffnesses. The shear strength of the joint is expressed by

$$\tau_f = C_t . \sigma_n$$

where,

τ_f = shear strength of the joint.

C_t = cohesion along the joint

σ_n = normal stress across the joint

ϕ_e = effective friction angle of joint surface

If the normal stress across the joint is tensile, it is assumed that the joint is incapable of resisting any shear stress i.e. it has no strength.

The rock mass having joints and fissures is incapable of sustaining any tension. The scheme developed by Zienkiewicz is useful to handle such a no tension situation. The main steps in such a analysis are -

i) Initialization of problem, defining boundary conditions and assignment of initial stresses.

ii) Analysis of the problem as linear elastic problem. The induced changes in stresses are added to the initial stresses to compute principal stresses.

iii) If there are tensile zones in the surrounding mass, the nodal forces are applied in such a manner that the tensile zones are eliminated.

iv) Repitition of the elastic analysis for calculated equivalent nodal forces.

v) If the tensile zones are eliminated at the end of step(iv)the analysis is complete ,otherwise steps (iii) and (iv) have to be repeated until there is no appreciable difference in magnitude and distribution of stresses with further iterations.

3.2.2 Construction Sequence in FEM Analysis

The conventional linear analysis is carried out for underground structures assuming that the entire construction takes place in a single operation. In actual practice, the excavation is carried out in stages. The linear elastic approach assumes the insitu stresses to be acting on the completed structure. However, the stresses in the final configuration are dependent upon the intermediate configurations and loadings.

The analytical simulation procedure, developed by Goodman and Brown, divides the rock mass into Finite Element Mesh with sides and boundaries located at appropriate distances. Before the construction starts, the mass is subjected to insitu stresses. A cycle of finite element analysis is performed for the insitu loads in which the resulting stresses

(σ_o) are computed. After the first stage excavation, equivalent forces(Q_1) at the nodes on x,y and z planes are computed. Forces equivalent in magnitude but opposite in sign are then applied at these nodes. The finite element analysis is carried out with(Q_1) and the resulting stress vector $(\Delta\sigma_o)$ is added to (σ_o) to give the principal stress vector (σ_1).

$$(\sigma_i) = (\sigma_o) + \Sigma\Delta\sigma_i$$

where σ_i is the stress at ith stage of excavation.

3.2.3 Non-Linear Material Behaviour

(a) Elastic behaviour

In this, the material parameters depend upon the state of stress. In the incremental approach, the load is applied in increments requiring a separate solution at each stage of load increment. The technique approximates the behaviour as piecewise linear. During the application of each load increment, the material is considered to be linear and elastic but different material properties are used for each incremental value.

(b) Elasto-plastic Behaviour

In this approach, the elastic formulations for the behaviour prior to proportionality limit are used. The piecewise linear approximations are used upto yield point. However, the plastic and inelastic behaviour beyond the yield point requires a different approach. It is necessary to have a yield criterion to ascertain the state of stress at which yielding starts. The behaviour beyond yield point is described by two theories, namely deformation theory and incremental or flow theory. In the deformation theory, the plastic strains are uniquely defined by the state of stress whereas in the incremental theory, the plastic strains depend upon a combination of factors such as increments of stresses and strain and the state of stress. The strain vector (ϵ) is decomposed into elastic (ϵ e) and plastic (ϵ p) vectors

$$(\epsilon) = (\epsilon_e) + (\epsilon_p)$$

3.2.4 Anisotropic Rock Mass

An anistropic rock mass is one, whose properties vary with the direction such as schists, slates, gneisses, phyllites, sandstones, shales and limestones. The response of such rock masses to underground excavations is strongly affected by (i) Orientation of the opening with respect to direction of anisotrophy (ii) type of anisotrophy(iii) Orientation of the opening and anisotropy with respect to the direction of rock in situ stresses (iv) magnitude of insitu stresses.

For the analysis of structures in anisotropic rock mass the assumptions of linear elastic, homogeneous continuous and anisotropic rock mass are made. In the plane strain formulation, the anisotropic rock mass and cavity therein are assumed to be of infinite length and to be subjected to the action of body and surface forces directed normal to the Z direction.

Rock mass insitu may be subjected to different horizontal and vertical stresses. This initial stress anisotropy produces different strengths in different directions. The modulus of elasticity in different directions may be different. The direction of principal stresses, on loading, varies and causes changes in the anisotropy with respect to stresses and mechanical properties of rock mass.

The anisotropy can be accounted for in Finite Element analysis as suggested by Dunlop, using the relation

$$E_{Bi} = E_h - (E_h - E_v) \sin^2 B_i$$

where B_i = angle between the horizontal and instantaneous directions of principal compression.

E_{B_i} = Modulus value to be used in analysis.

CHAPTER-4

STRESS ANALYSIS OF NATHPA JHAKRI UNDERGROUND POWER HOUSE

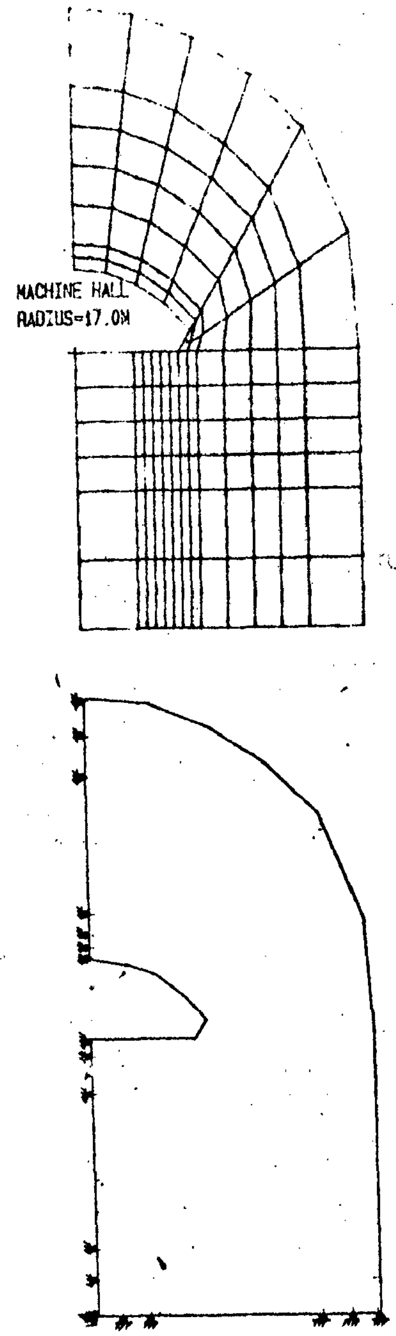
4.1 FORMULATION OF THE PROBLEM

The schematic layout of power house complex and the geological sections have shown in Chapter-1. A computer programme has been prepared to generate the 'Finite Element Mesh' and the related input data for the finite element analysis. The programme can handle all cases of sequential excavation for the machine hall cavity. The case of transformer hall cavity fully excavated and the machine hall excavation upto generator floor/turbine floor can also be solved through this programme. Thus, all cases of machine hall have been dealt through the programme resulting in substantial time saving. The programme generates the mesh of eight nodal isoparametric elements and the corresponding input data for finite element analysis. The excavation for all cases has been simulated mainly in one step. The various mesh formations are shown in figures 4.1 to 4.4.

4.2 ASSUMPTIONS AND DATA

i) The rock mass has been assumed as linear, isotropic and homogeneous medium.

ii) The insitu stresses have been assumed vertical and horizontal respectively. The test results of flat jack test have shown the magnitudes of the stress field as -



BOUNDARY CONDITIONS
(All nodes not shown)

NATHPA JHAKRI POWER HOUSE CAVERN

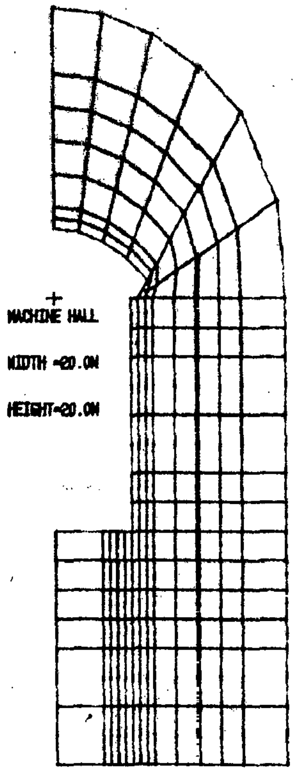
1ST STAGE EXCAVATION

FINITE ELEMENT MESH

NUMBER OF ELEMENTS=120

NUMBER OF NODES =411

FIG. 41



NATHPA JHAKRI POWER HOUSE CAVERN

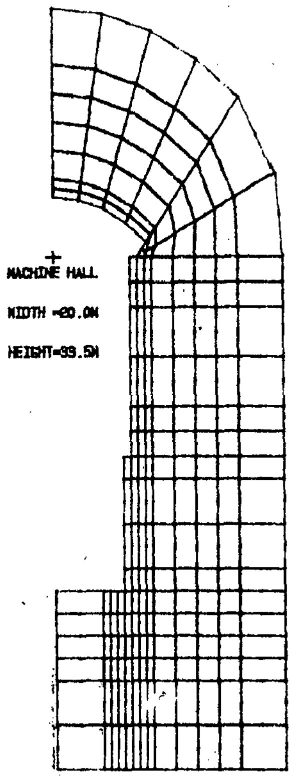
2ND STAGE EXCAVATION

FINITE ELEMENT MESH

NUMBER OF ELEMENTS=168

NUMBER OF NODES =567

FIG. 4.2



NATHPA JHAKRI POWER HOUSE CAVERN

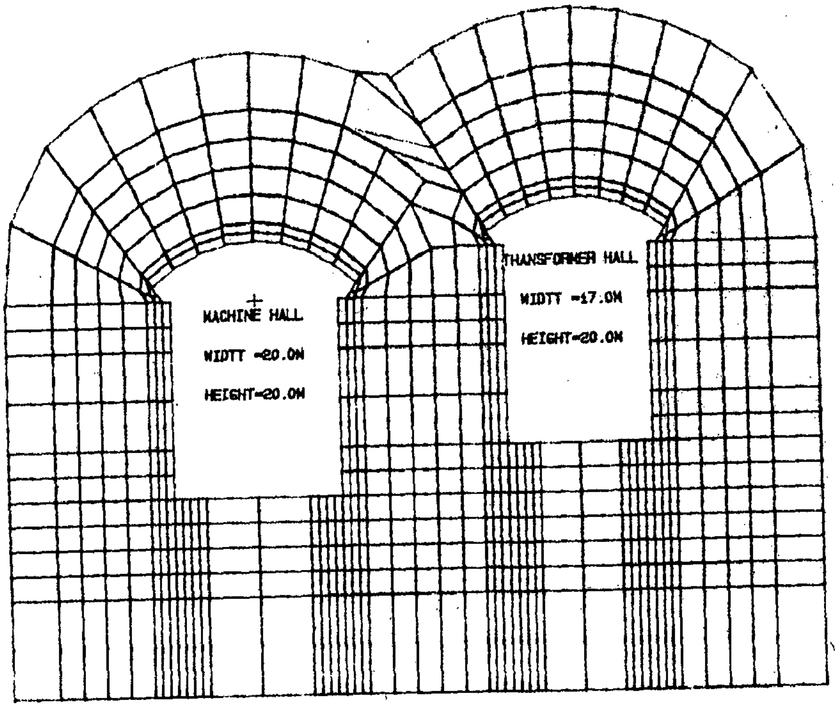
3RD STAGE EXCAVATION

FINITE ELEMENT MESH

NUMBER OF ELEMENTS=204

NUMBER OF NODES =663

FIG 4.3



NATHPA JHAKRI POWER HOUSE CAVERN

FINITE ELEMENT MESH

NUMBER OF ELEMENTS=598

NUMBER OF NODES =1938

BOUNDARY CONDITIONS

FIG.4.4

Vertical = 10.42 kg/cm^2

Horizontal in longitudinal direction = 7.71 kg/cm^2

Horizontal in transverse direction = 8.06 kg/cm^2

The value of modulus of elasticity of rock mass, as given in the test reports (335600 kg/cm^2), has been used in the analysis.

iii) The ratio of horizontal to vertical insitu stresses 'K' has been varied from 0.0 to 0.8.

iv) The poisson's ratio for the rock mass has been taken as 0.2.

v) The unit weight of rock, as per test results, is 2.85 T/M^3.

4.3 BOUNDARY CONDITIONS

i) For the machine hall case studies, the problem has been treated as symmetrical about the centre line of the machine hall (vertical). The horizontal displacements have been restrained along this line of symmetry.

ii) The extreme horizontal surface of the finite element mesh has been restrained from vertical movement. The horizontal movement of all nodes, but one, is permitted. (Fig.4.4)

iii) The insitu stress field has been converted to equivalent nodal forces in horizontal and vertical directions along the boundary. The load calculations have been included in the data generation programme.

4.4 TESTING OF COMPUTER PROGRAMME

The programme for finite element analysis, being used for three dimensional stress analysis of earthen dam and available with Shri R.P. Singh, Reader, WRDTC, has been suitably modified to deal the two dimensional plane strain cases.

A problem of cantilever has been checked with the modified programme (ref. Fig. 4.5).

The cases given in table 4.1(a), have been studied and the comparison of results obtained has been made with the results given by the equations based on theory of elasticity.

Table 4.1(a)

CHECK TEST RESULTS

Sl.No.	Case description	Check parameter with E = 1 t/m^2 Actual	Test result
1.	Gravity load in vertical direction	6.0T/ M^2 Base stress	5.8T/M^2 Gaussian point stress
2.	Uniformly distributed load in horizontal direction	243 M (displacement at free end)	260 M
3.	No load in any direction	0.00 stress and displacement	0.00
4.	Hydrostatic pressure on vertical face	103.23 M (displacement at free end)	107.23 M

The above comparison indicates that the results given by the programme are correct upto a fair degree of accuracy. and the programme can be used for the analysis of proposed problem.

4.5 CASE STUDIES

Following cases were studied to see the effect of radius variation, increased span, concrete arch, sequential excavation, modulus of elasticity and K.

A. Machine hall only.

i) a) Construction stage 1 (see fig.4.1)

b) construction stage 1 with increased span.

ii) Construction stage 1 with concrete arch in position.

iii) 2nd stage excavation

a) Radius of arch varying from 15 M to 19 M at an increment of 1M.

b) K value varying from 0 to 0.8.

c) With value of modulus of elasticity as 2.0×10^5 kg/cm^2.

d) with increased span and K = 0.8.

iv) 3rd stage Excavation

a) K value varying from 0.4 to 0.8.

B. With increased span.

B) Two cavities with K = 0.8.

a) Pillar width varying from 0.5x(combined span) to 1.0x(combined span).

4.6 DISCUSSION OF RESULTS

4.6.1 Roof Arch Excavation (Ist Stage)

Three cases, as discussed in para 4.5 were studied. The principal stress distribution (σ_1/σ_V) around the cavern, is shown in figure 4.6. The variation of displacements and principal stresses are shown in Table 4.1.

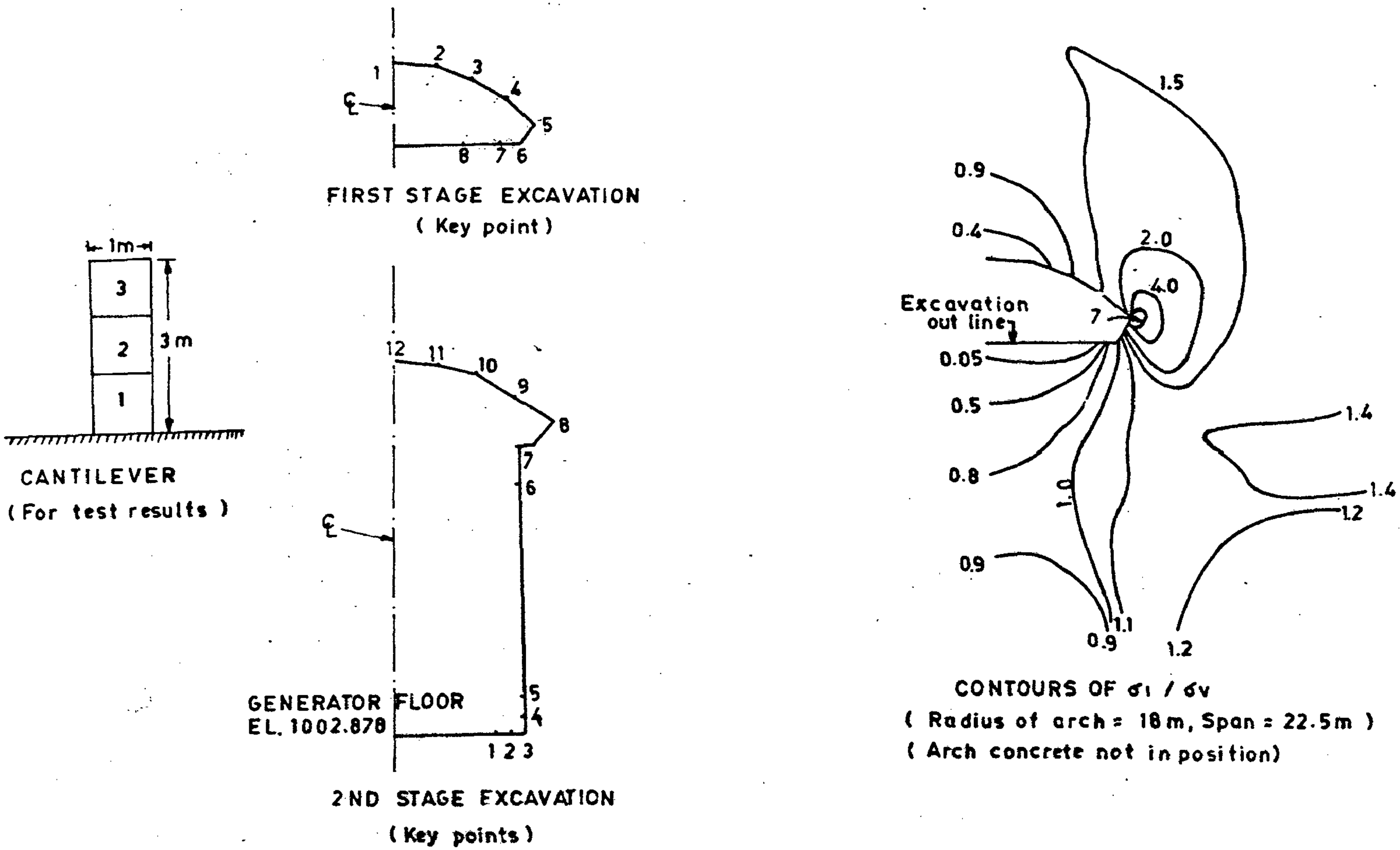
An examination of table 4.1 shows that with the increased span of 22.5 metres, the stresses do not vary significantly along the boundary of excavation. Further, the plots of figure 4.5 show that the stress gradient along the corner point, at the bottom of arch, is reduced. Thus, the span can be increased safely without endangering the stability of the opening.

The concrete arch case has been simulated by considering boundary elements to be constituted of concrete(Fig.4.6).The E value for concrete has been taken as 2.0×10^5 kg/cm^2. The maximum stress in concrete arch develops in the haunch portion, gradually reducing to small value at crown. The values are -

Max. stress at haunch = 75.6 kg/cm^2(compression)

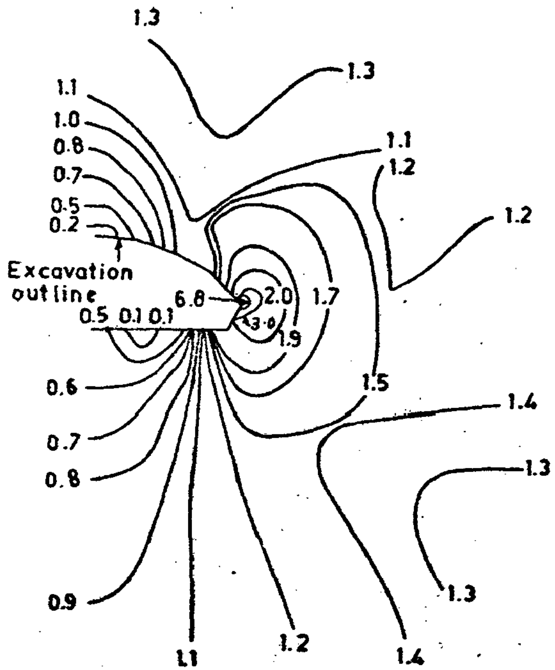
Max. stress at crown = 20.4 kg/cm^2(compression)

There is little tensile (minor principal) stress along the boundary. The stress pattern outside the concrete arch does not vary significantly from that developed without concrete arch.

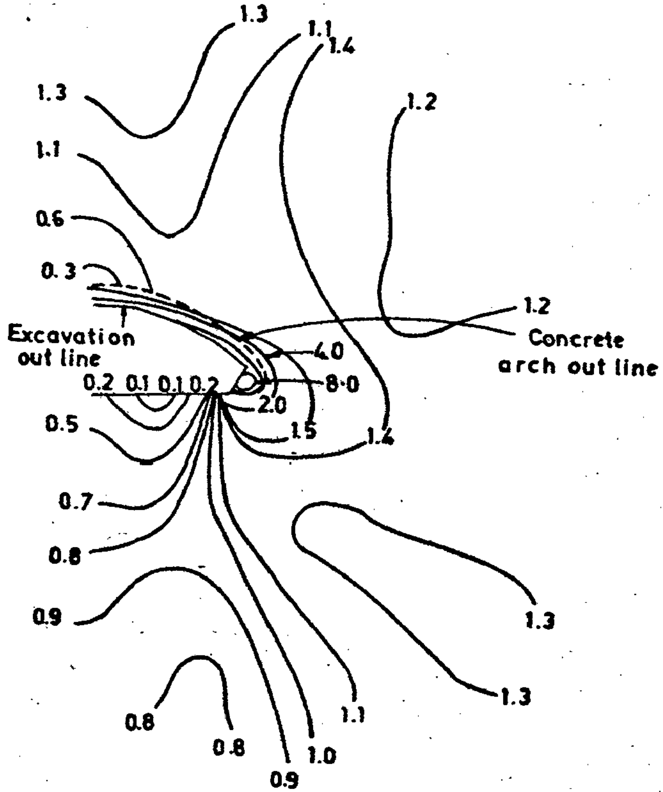


BOUNDARY KEYPOINTS AND STRESS CONTOURS

FIG 4.5



Arch concrete not in position
Maximum displacement = 20.9 mm.



Arch concrete in position
Maximum displacement = 19.6 m m.

FIRST STAGE EXCAVATION
CONTOURS OF σ_1 / σ_v
(σ_v = 100 T/m^2, K = 0.8 , Radius of arch = 17 m)

FIG. 4.6

Table 4.1

IST STAGE EXCAVATION

Key point \ Case	R = 17.0 M Span = 20.0 M				R = 17.0M Span = 20.0M (concrete arch placed)				R = 18.0 M Span = 22.5 M			
	Principal stresses (T/M^2)		Displacements (cms)		Principal stresses (T/M^2)		Displacements (cms)		Principal stresses (T/M^2)		Displacements (cms)	
	σ_1	σ_3	U	V	σ_1	σ_3	U	V	σ_1	σ_3	U	V
1.	-14.6	0.8	0.00	1.87	-204.6	-4.2	0.00	1.69	-13.2	-0.8	0.00	2.09
2.	-24.6	+1.5	0.29	2.00	-230.8	3.4	0.26	1.88	-26.0	1.5	0.34	2.19
3.	-67.3	-8.4	0.49	1.76	-101.8	-13.6	0.45	1.67	-75.9	-9.3	0.57	1.89
4.	-138.6	-4.3	0.58	1.45	-413.3	16.6	0.54	1.40	-154.8	-7.8	0.65	1.51
5.	-698.2	-121.8	0.58	1.16	-756.0	-50.7	0.55	1.13	-725.8	-127.2	0.61	1.17
6.	-303.8	+34.0	0.02	0.50	-600.8	-65.3	0.00	0.55	-283.7	+42.3	0.01	0.51
7.	-63.4	2.6	0.01	0.37	-69.8	5.8	0.04	0.41	-60.3	2.7	0.02	0.39
8.	-0.8	+0.6	0.01	0.14	-0.2	9.0	0.03	0.16	-0.8	1.7	0.02	0.14

4.6.2 Machine Hall Excavation upto El.1002.878 M(2nd stage)

In this case, the following studies were made -

i) Effect of radius variation,

ii) Effect of 'E'.

iii) Effect of 'K'.

4.6.2.1 Effect of Radius variation.

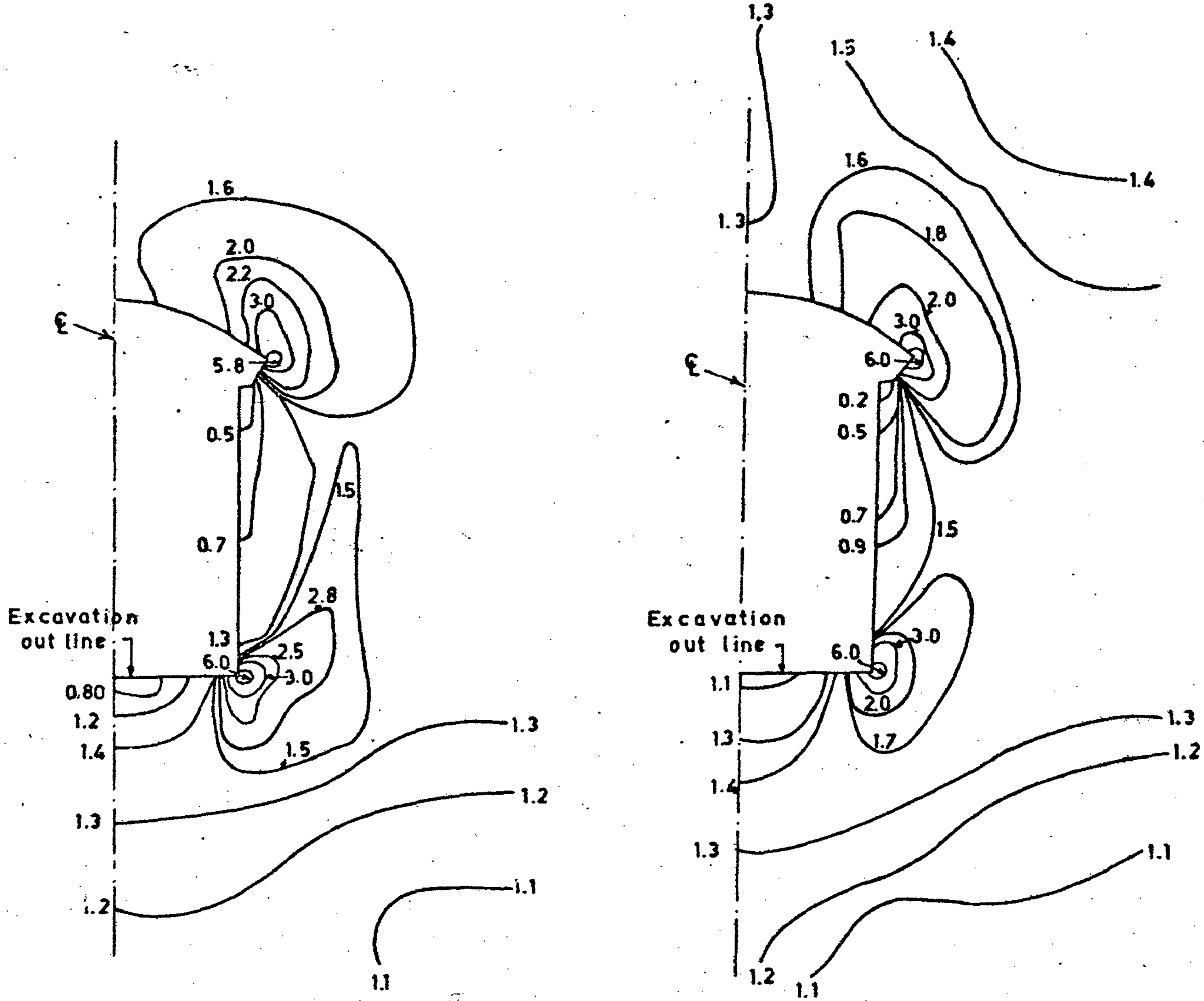
The test results indicate the value of 'K' to be nearly equal to 0.8. Thus, the analysis has been done keeping the value of K as 0.8. The radius of roof arch has been varied from 15 m to 19 m. The results of the analysis are given in Table 4.2.

It has been observed that for the value of radius equal to 17 metres, the maximum compressive stresses at key points 2 and 8 (Fig.4.5) are minimum. There is no significant change in the value of displacements. Thus, the radius of roof arch has been kept as 17 metres for subsequent analysis. The distribution of major principal stress, around the cavern, has been shown in figures 4.7 to 4.9(for different values of radius). The contours of σ_1/σ_V become closer with the increase in the value of radius. However, the local boundary stress values at key point '3' decreases with the increase in radius.

Table 4.2

EFFECT OF CHANGE IN RADIUS
(K = 0.8)

Radius R / Key points	R = 15M			R = 16 M			R = 17 M			R = 18M			R = 19M		
	σ_1 (T/M^2)	Displacement (cms) U	V	σ_1 (T/M^2)	Displacement (cms) U	V	σ_1 (T/M^2)	Displacement (cms) U	V	σ_1 (T/M^2)	Displacement (cms) U	V	σ_1 (T/M^2)	Displacement (cms) U	V
1.	-362	0.20	0.28	-355	0.20	0.28	-355	0.20	0.27	-353	0.20	0.28	-351	0.29	0.45
2.	-470	0.36	0.50	-466	0.22	0.30	-463	0.35	0.50	-460	0.21	0.31	-458	0.21	0.3[illegible]
3.	-603	0.43	0.59	-605	0.42	0.59	-602	0.42	0.59	-593	0.41	0.58	-591	0.41	0.58
4.	-475	0.63	0.95	-472	0.62	0.74	-·69	0.61	0.74	-467	0.61	0.74	-466	0.60	0.73
5.	-361	0.74	0.81	-359	0.73	0.81	-·57	0.72	0.80	-355	0.71	0.80	-354	0.71	0.80
6.	-8	0.98	1.13	-12.7	0.90	1.13	-·6.4	0.89	1.14	-151	0.87	1.14	-22.4	0.86	1.14
7.	-2	0.74	1.23	-2	0.71	1.24	-3	0.69	1.25	-3	0.68	1.26	-3	0.66	1.26
8.	-609	0.70	1.29	-621	0.67	1.36	-6·7	0.65	1.30	-653	0.63	1.31	-655	0.61	1.32
9.	-239	0.47	1.87	-232	0.43	1.37	-2.5	0.43	1.85	-217	0.37	1.90	-212	0.35	1.9[illegible]
10.	-193	0.32	2.05	-184	0.29	2.06	-1·3	0.27	2.08	-167	0.25	2.09	-160	0.24	2.10
11.	-164	0.16	2.16	-152	0.15	2.13	-140	0.14	2.19	-137	0.15	2.15	-131	0.12	2.20
12.	-155	0.00	2.20	-145	0.00	2.21	-140	0.00	2.22	-129	0.00	2.22	-124	0.00	2.24

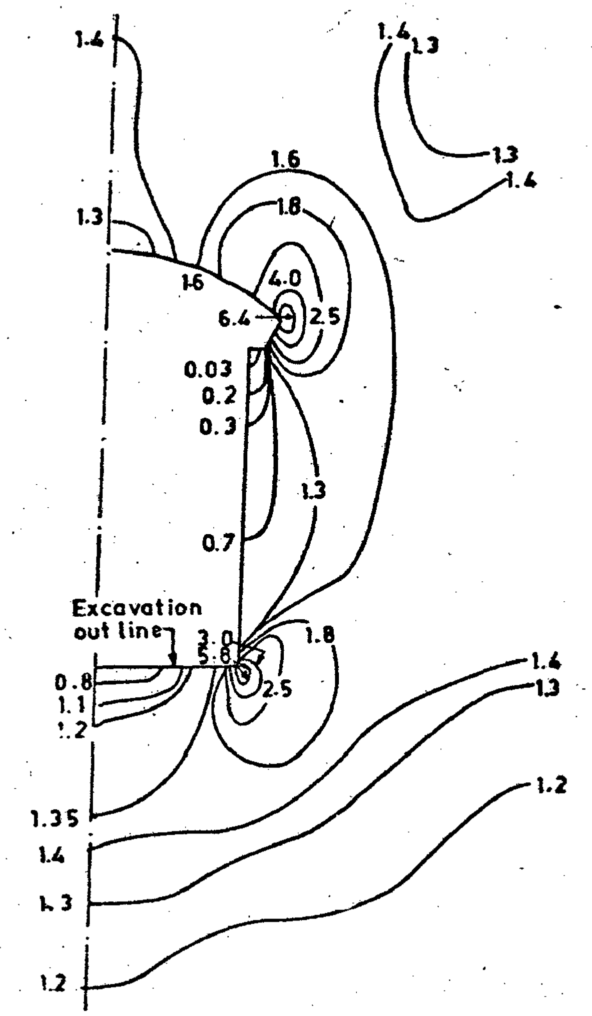


Radius of arch = 15 m.
Maximum displacement = 22.0 mm.

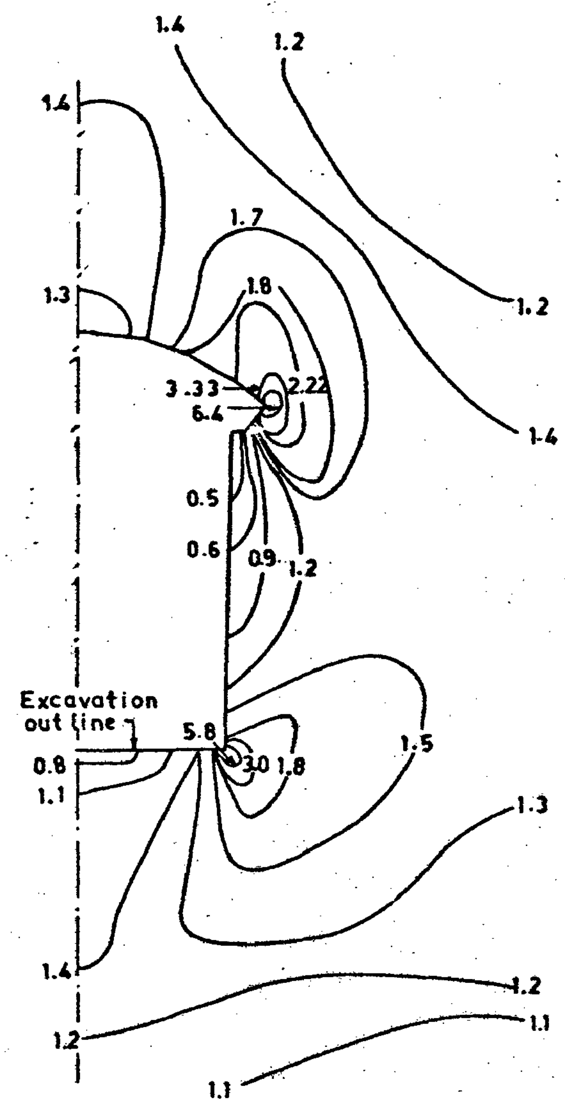
Radius of arch = 16 m.
Maximum displacement = 22.1 mm.

2ND STAGE EXCAVATION
CONTOURS OF σ_1 / σ_v
(σ_v = 100 T / m^2, K = 0.8)

FIG. 4.7



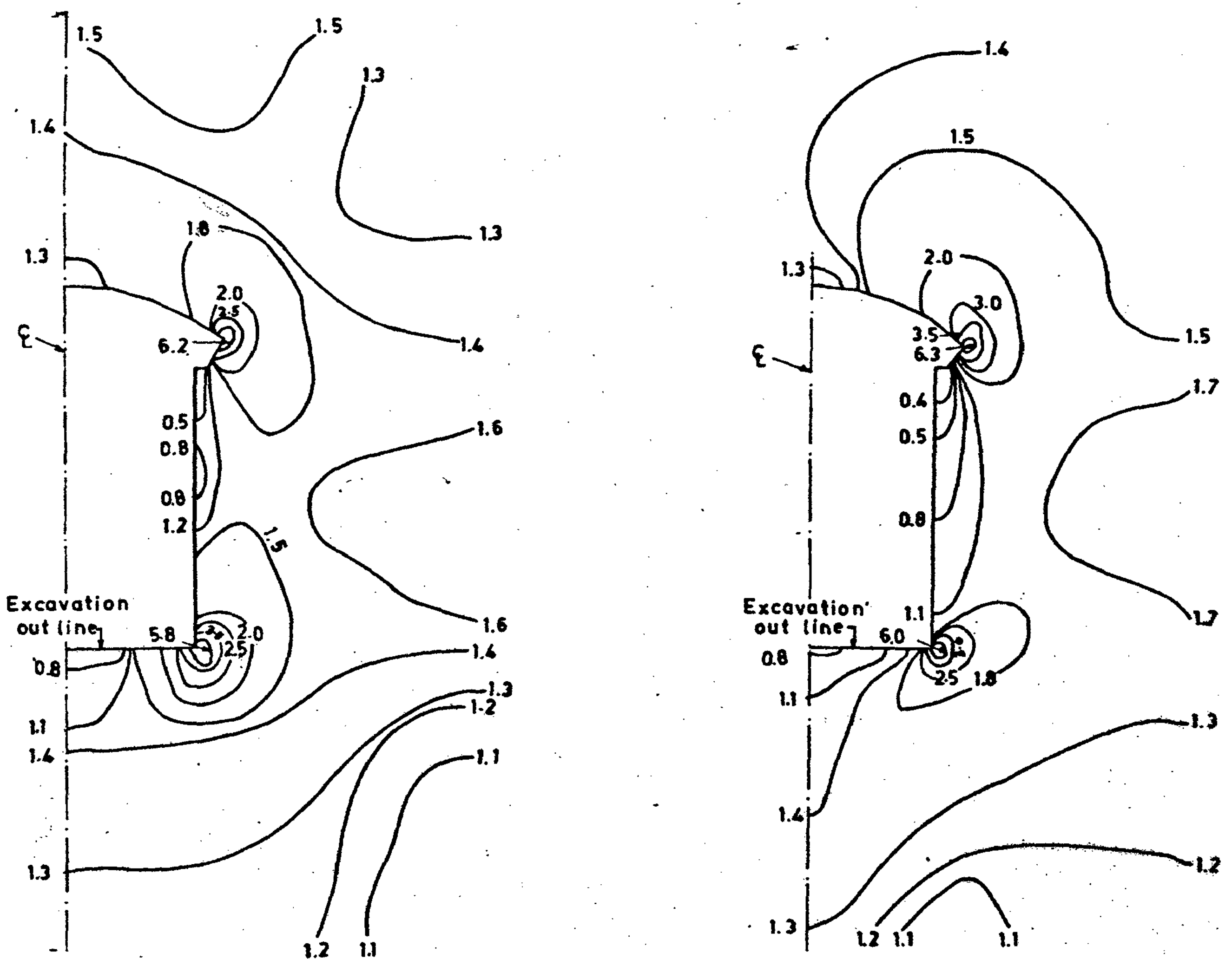
Radius of arch = 18 m
Maximum displacement = 22.2mm



Radius of arch = 19 m
Maximum displacement = 22.4 mm.

2ND STAGE EXCAVATION
CONTOURS OF σ_1 / σ_v
(σ_v = 100T/m^2, K = 0.8 m)

FIG 4.8



Span = 20m , Radius of arch = 17m
(Maximum displacement = 22.2 mm)

Span = 2 2.5 m , Radius of arch = 18 m
(Maximum displacement = 24.5 mm)

2ND STAGE EXCAVATION
CONTOURS OF σ_1 / σ_v
($K = 0.8$, $\sigma_v = 100 T/m^2$)

FIG. 4.9

4.6.2.2 Effect of 'K'

The value of K has been varied from 0.0 to 0.8. It has been observed that the horizontal displacement, along the vertical sides, increases with the increase in 'K'. On the other hand, the vertical displacement along the roof arch decreases with the increase in 'K'. The results of the analysis are given in Table 4.3.

Further, the nature of major principal stress 'σ_1' at crown and at flat bottom, changes from compressive to tensile with the decrease in 'K'. The maximum stress at haunch increases with the decrease in 'K'.

For K = 0 σ_1 = 73.05 kg/cm^2(compression)

and for K =0.8 σ_1 = 63.75 kg/cm^2(compression)

At key point 3, the values of compressive stress are -

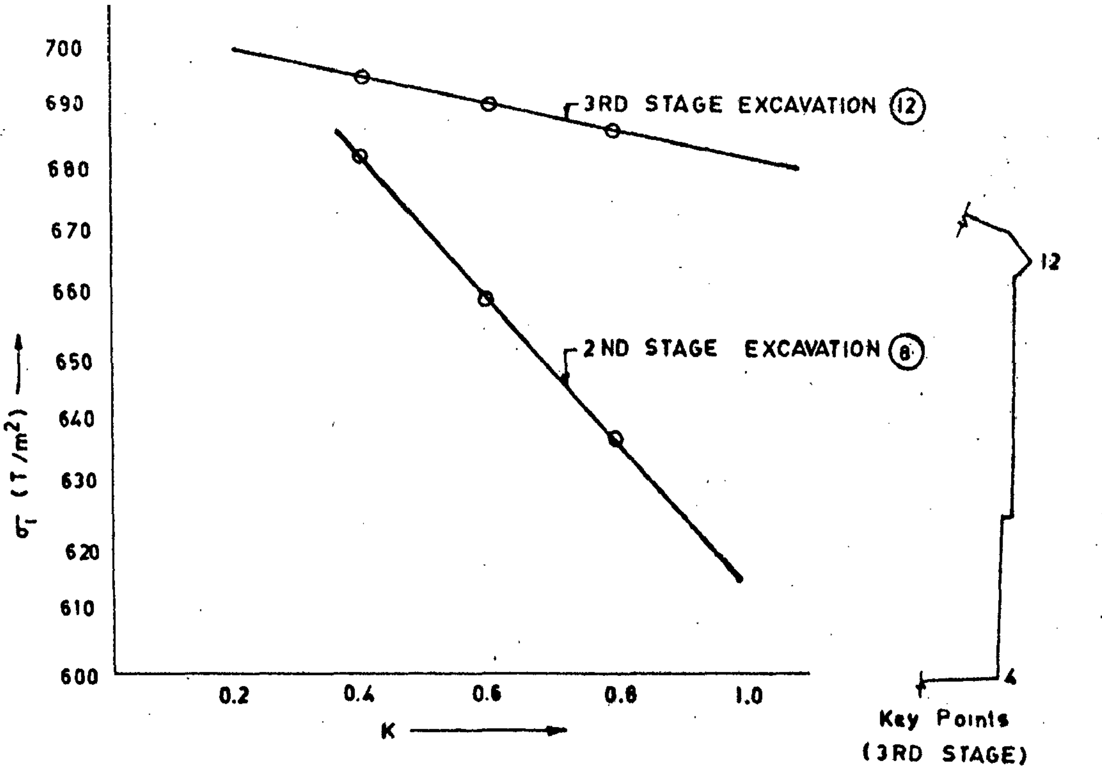
For K = 0.8 σ_1 = 59.82 kg/cm^2

and for K = 0.0 σ_1 = 31.97 kg/cm^2

Figure 4.10 shows the relation between σ_1 and K for different key points. It has also been observed that the stress gradient around key point 8 increases with a corresponding decrease in K. For the key point 2, the behaviour is exactly opposite. The contours of σ_1/σ_v, for different K values, have been shown in figures 4.11 and 4.12. The deflected boundary profile of the cavern, for different values of K, has been shown in Fig.4.13.

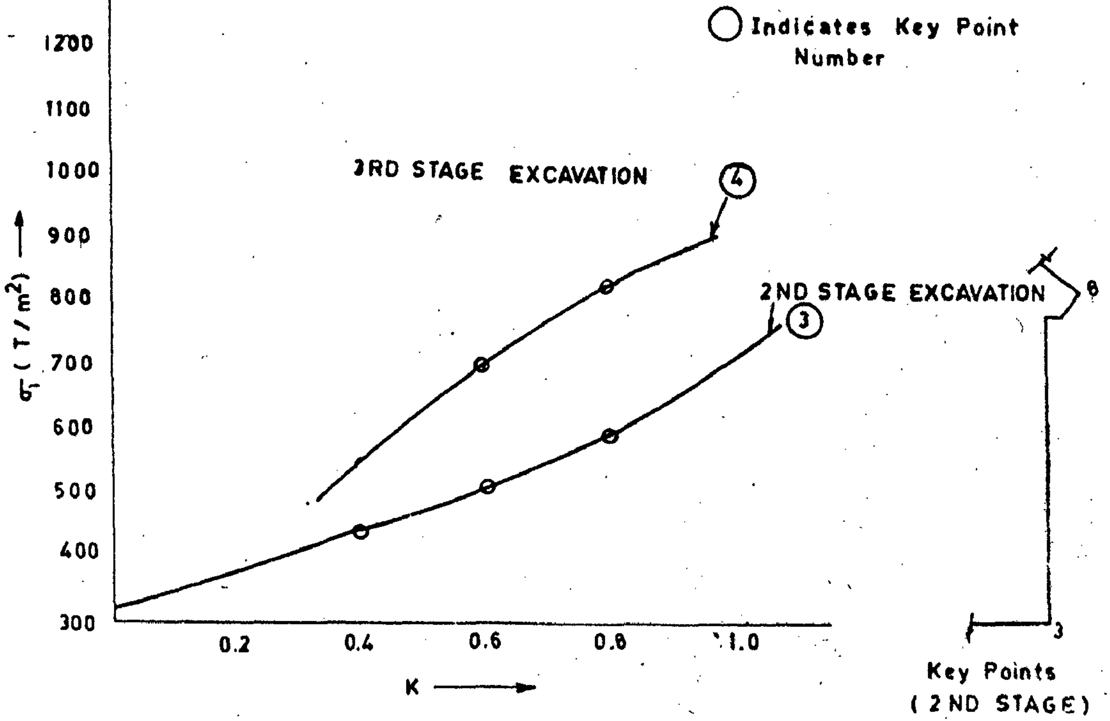
4.6.2.3 Effect of Modulus of Elasticity 'E'

To study the effect of change in the value of E, the analysis was done with 'E' equal to 2.0 x10^5 kg/cm^2. The results of the analysis indicate little variation in the stress



A—AT ABUTMENT

○ Indicates Key Point Number

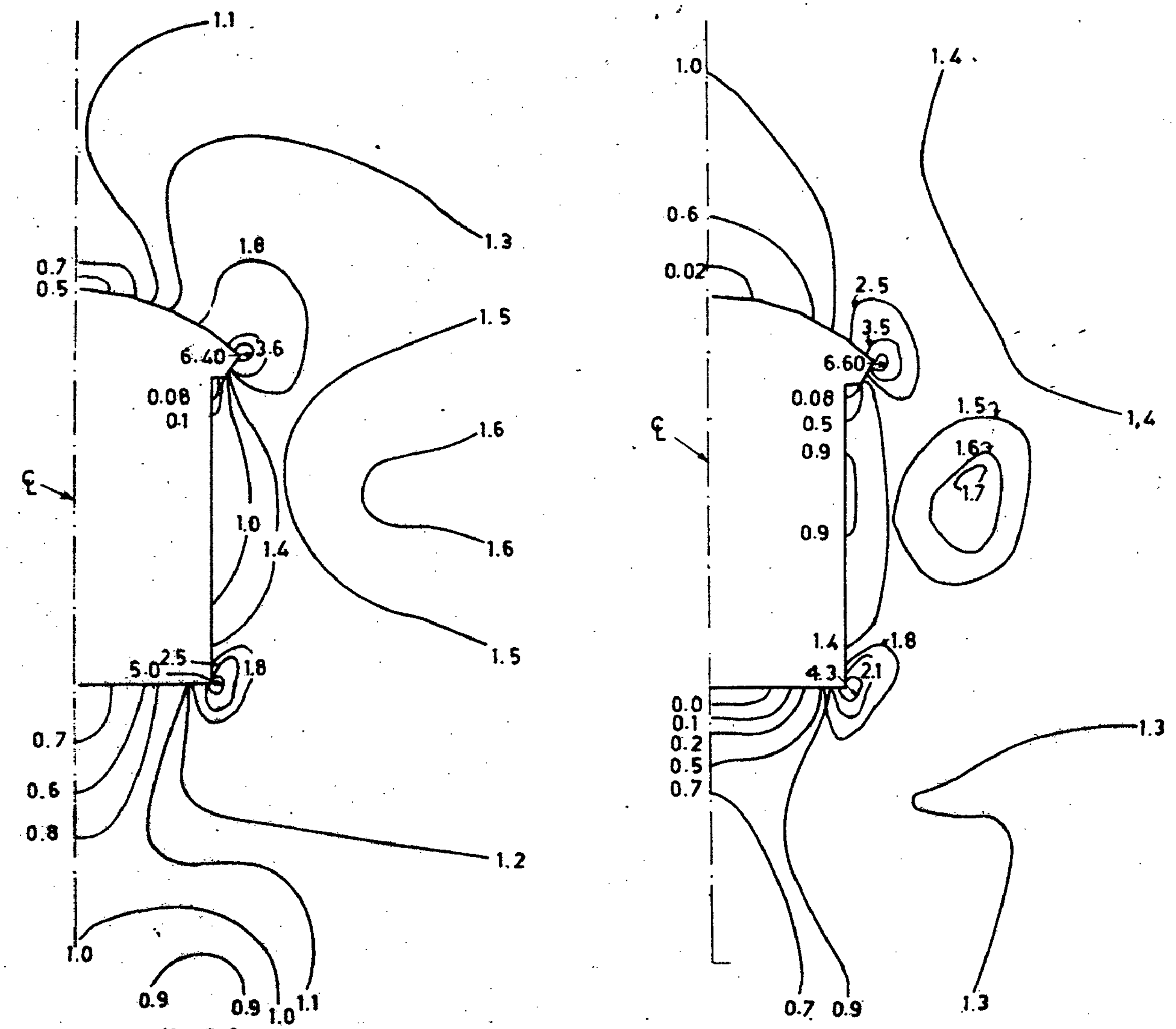


B—AT BOTTOM SHARP CORNERS

VARIATION OF σ_1 (MAX.) WITH - K

($K = \sigma_h / \sigma_v$)

FIG. 4.10



K = 0.6

(Maximum displacements(mms))

(Horizontal = 5.3, Vertical = 24.7)

K = 0.4

(Maximum displacements (mms))

(Horizontal = 2, Vertical = 27.2)

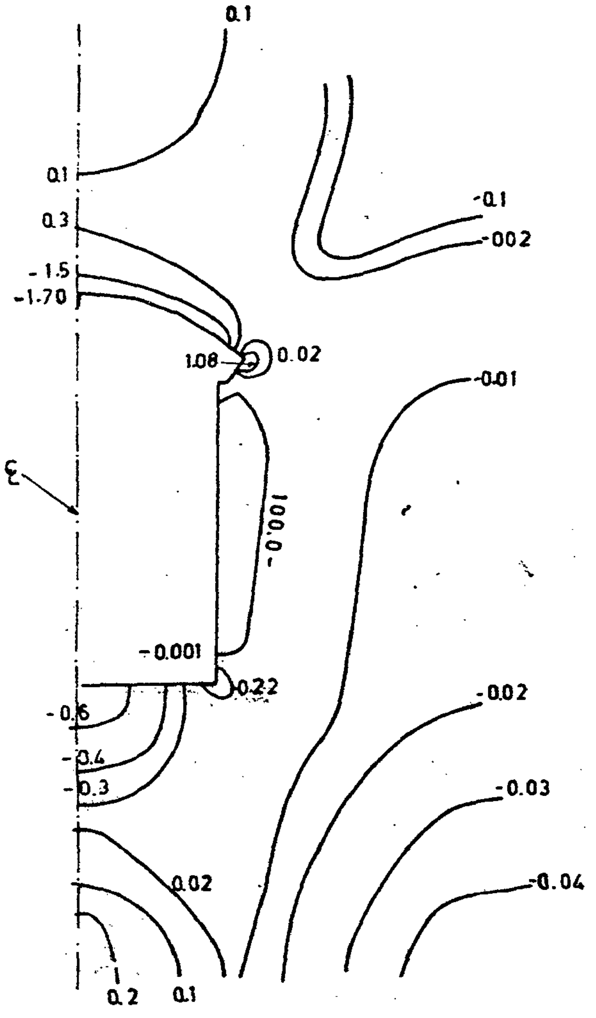
2ND STAGE EXCAVATION

CONTOURS OF $\sigma_1/\sigma V$

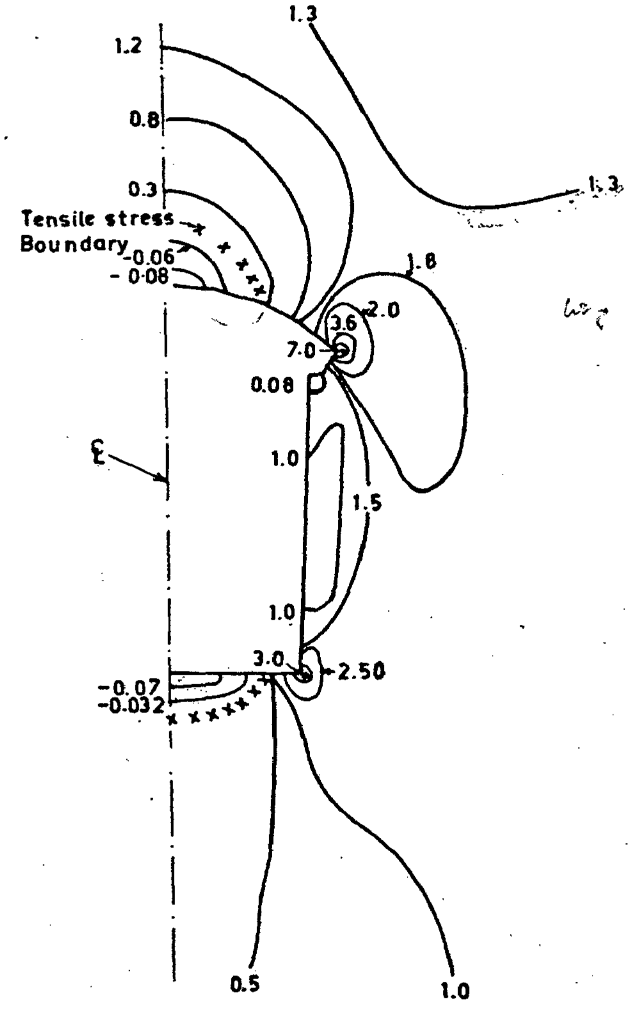
(Radius of roof arch: 17 m)

(σV = 100 T/m^2)

FIG. 4.11



Contours of σ_2 / σ_v



Contours of σ_1 / σ_v

2ND STAGE EXCAVTION
Radius of arch = 17 m.
σ_v = 100 T/m^2, K = 0.00
Maximum displacements (mms)
Horizontal=7.0, Vertical= 32.2

FIG. 4.12

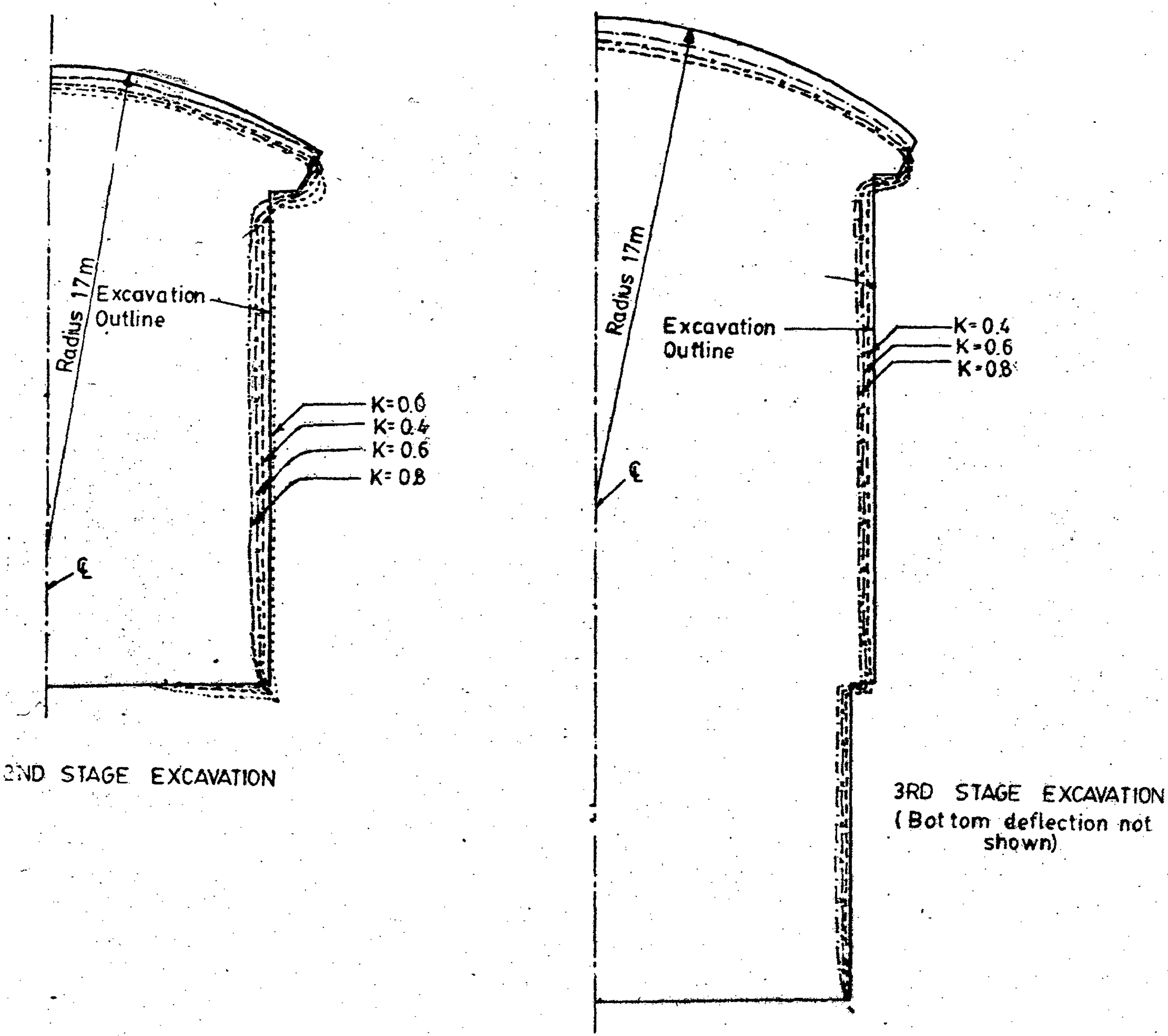


FIG 4.13 DEFLECTED BOUNDARY PROFILES

Table 4.3

EFFECT OF K

(R = 17.00M, 2nd Stage Excavation)

K → Key point	0.0 Principal Stress (T/M²)		0.0 Displacement (cms)		0.4 Principal Stress (T/M²)		0.4 Displacement (cms)		0.6 Principal Stress (T/M²)		0.6 Displacement (cms)		0.8 Principal stress (T/M²)		0.8 Displacement (cms)	
	σ_1	σ_3	u	v	σ_1	σ_3	u	v	σ_1	σ_3	u	v	σ_1	σ_3	u	v
1.	-3.7	52.9	-0.23	0.31	-120 9	2.2	0.00	0.32	-168.5	-1.9	0.12	0.33	-183.5	-3.0	0.20	0.27
2.	-453	44.1	-0.26	0.38	-150.5	6.4	0.02	0.40	-177.9	3.0	0.16	0.41	-253.7	0.8	0.35	0.50
3.	-319.7	-23.0	-0.27	0.51	-450.4	-73.4	0.08	0.68	-522.0	-92.3	0.25	0.57	-596.2	-108.7	0.42	0.59
4.	-255.0	-8.3	-0.30	0.62	-272.0	-7.0	0.16	0.62	-281.1	-5.9	0.38	0.71	-290.5	-4.4	0.61	0.74
5.	-206.9	2.3	-0.33	0.71	-166.0	+14.1	0.20	0.75	-146.3	20.8	0.46	0.78	-127.4	28.2	0.72	0.80
6.	-88.5	2.5	-0.52	1.53	-52.4	2.0	0.18	1.33	-344	1.7	0.53	1.23	-16.4	1.5	0.89	1.14
7.	-1.3	7.4	-0.62	1.55	-0.8	9.4	0.05	1.34	-0.5	10.4	0.39	1.24	-0.3	11.5	0.61	1.25
8.	-730.5	-108.6	-0.69	1.98	-681.7	-106.2	0.06	1.74	-659.1	-103.2	0.25	1.62	-637.5	-99.0	0.65	1.30
9.	-42.9	-2.6	-0.57	2.63	-134.0	-2.1	0.09	2.26	-179.7	-1.6	0.16	2.07	-225.5	-1.1	0.43	1.85
10.	-10.3	93.4	-0.38	2.96	-42.8	-6.7	+0.06	2.52	-107.7	-8.2	0.11	2.30	-178.8	-8.3	0.27	2.08
11.	3.4	174.8	-0.20	3.16	1.8	15.0	+0.05	2.61	-65.0	1.0	0.05	2.43	-144.9	0.2	0.14	2.19
12.	1.0	196.2	-0.00	3.22	-0.4	29.5	0.00	2.72	-53.9	-1.1	0.00	2.47	-140.0	0.4	0.00	2.22

distribution. However, the displacements decrease significantly The stress contours (σ_1/σ_v) are shown in Fig.4.16.

For E = 35600 kg/cm^2 Maximum Displacement = 22.2 MM

and for E = $2 \times 10^5 kg/cm^2$ Maximum displacement = 03.4 MM

4.6.2.4 Effect of Increasing width of cavity

The u/s valve can be accommodated in the machine hall cavity itself if its width is increased by 2.5 metres (as proposed in case of Tehri Underground Power House). Thus, a case with width of cavity equal to 22.5 metres was analyzed. The results of the analysis have been tabulated below -

Table 4.4
EFFECT OF INCREASED WIDTH
(2nd stage Excavation)

Key points	Span = 20.0M R = 17M				Span = 22.5M R = 18M			
	σ_1	σ_3	u	v	σ_1	σ_3	u	v
1	-218.3	-17.8	0.30	0.42	-220.7	-16.9	0.32	0.43
2	-258.7	0.8	0.35	0.50	-255.1	1.1	0.38	0.51
3	-596.2	-108.3	0.42	0.59	-614.5	-111.3	0.44	0.60
4	-290.5	-4.4	0.61	0.74	-303.9	-4.9	0.64	0.76
5	-127.4	28.2	0.70	0.80	-137.2	284	0.74	0.83
6	-16.4	1.5	0.89	1.14	-18.5	1.7	0.90	1.20
7	-0.3	11.5	0.74	1.14	-0.3	12.2	0.74	1.20
8	-637.8	-99.0	0.57	1.50	-647.7	-99.4	0.58	1.58
9	-225.5	-1.1	0.43	1.85	-228.0	-3.7	0.42	2.04
10	-173.8	-8.3	0.27	2.08	-173.5	-8.8	0.20	2.27
11	-144.8	0.2	0.14	2.19	-138.1	0.3	0.14	2.41
12	-137.2	-1.8	0.00	2.22	-129.6	-1.7	0.00	2.45

σ_1 and σ_3 are in t/m^2
u and v are in cms.

The distribution of major principal stress around opening is shown in Fig.4.9.

The stress contours, for the increased cavity width case, become wider around the corner points and elsewhere have been brought closer. The corner boundary stresses, however, increase slightly as shown in Table 4.4. The observation of the stress pattern and the displacements indicate that the cavity width can be increased without endangering its stability.

4.6.3 3rd Stage Excavation

The effect of increasing cavity width to house the control valves in the machine hall and the effect of change in K value has been studied for the case when the excavation is complete upto the turbine pit level. The results have been discussed in the following paragraphs.

The case has been analyzed for width of cavity equal to 22.5 metres and K equal to 0.8. The results have been given in the Table 4.5.

It is clear from the values given in the Table 4.5 that the magnitude of corner boundary stresses decreases slightly with the increase in the width of the cavity. The stress contours have been shown in fig.4.17. The magnitude of stresses around the cavity is slightly on the higher side with the increased span. However, dangerous stress patterns do not occur anywhere around the opening and the increased cavity width will not affect the stability of the cavity at this stage of excavation.

4.6.3.2 Effect of K

The results of analysis have been compiled in Table 4.6. A critical examination of Table 4.6 shows the trend of variation of displacements and principal stresses is similar to that observed in case of 2nd stage excavation. The **vertical** displacements of crown decrease with increase in K value. For example

For K = 0.4 Max. displacement = 29.5 MMS.

for K = 0.8 Max. displacement = 21.0 MMS

The horizontal displacements along the vertical wall increase with increase in K value. The relationship between displacements and K has been shown in Figure 4.15.

The compressive stress at haunch of roof arch also increases slightly with increase in K value, whereas the corners stress at key point 4 decreases significantly with the decrease in K value. For example

for K = 0.4 Max. compressive stress = 55.38 kg/cm^2

and for K = 0.8 Max. compressive stress = 86.26 kg/cm^2.

The compressive stresses along the arch decrease towards the crown and also decrease with the decrease in K values (Fig.4.14 and 4.10). The distribution of stress in other

Table 4.5

EFFECT OF INCREASED CAVITY WIDTH

(3rd Stage Excavation)

Key points	Span =20.0 M R=17.0M				Span = 22.5M R = 18 M			
	σ_1	σ_3	U	V	σ_1	σ_3	U	V
1.	-197.8	-0.4	0.30	0.33	-192.5	-0.8	0.34	0.32
2.	-262.8	-1.9	0.36	0.40	-261.8	-1.1	0.40	0.39
3.	-382.4	7.1	0.45	0.50	-385.4	7.8	0.49	0.50
4.	-896.1	-175.5	0.64	0.71	-884.8	-159.6	0.68	0.71
5.	-168.0	-3.8	1.13	0.98	-177.6	-3.8	1.18	0.99
6.	-116.4	9.9	1.41	1.07	-125.1	10.1	1.46	1.10
7.	-16.7	1.0	1.94	1.18	-22.5	11	1.68	1.16
8.	-1.1	4.6	2.04	1.19	-1.5	4.9	2.08	1.23
9.	-14.4	0.6	2.12	1.18	-60.3	-4.8	2.16	1.23
10	0.8	11.2	1.68	1.17	0.9	8.9	1.74	1.26
11	0.1	13.2	1.41	1.16	-0.1	13.5	1.47	1.37
12.	-685.0	-100.3	+1.25	1.60	-673.2	·97.1	1.22	1.70
13.	-286.3	-9.3	0.53	2.01	-280.7	-9.5	0.57	2.19
14.	-263.2	-3.0	0.00	2.10	-252.1	-2.8	0.00	2.30

σ_1 and σ_3 are in T/M^2

U and V are in cms.

Table 4.6

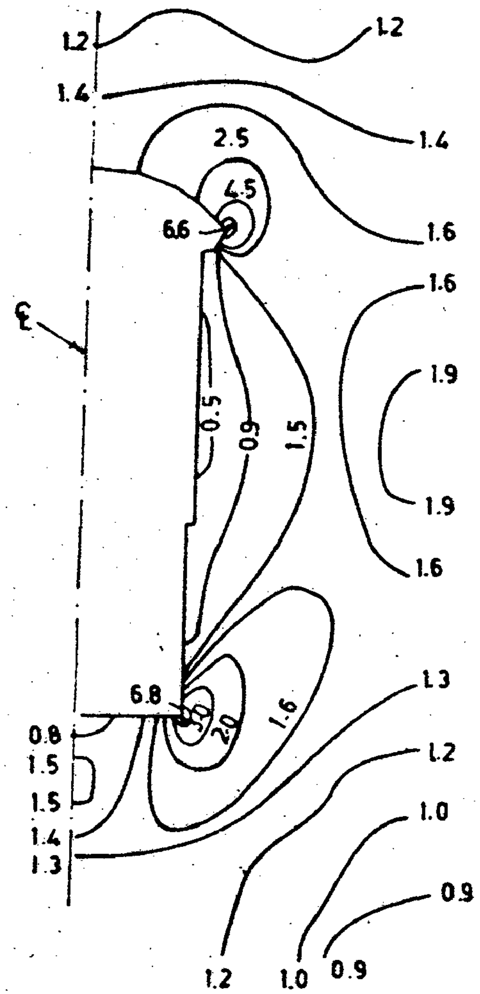
EFFECT OF K

3rd STAGE EXCAVATION

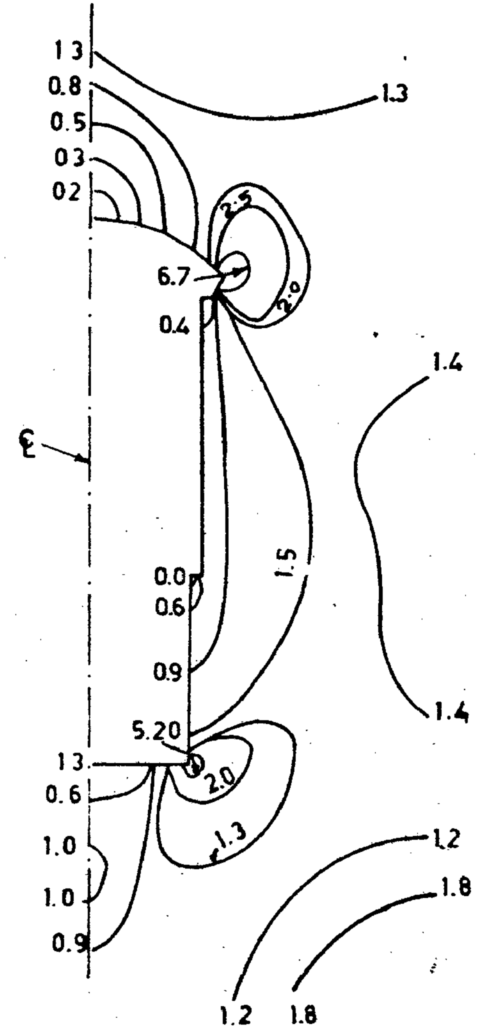
Key point	K = 0.4				K = 0.6				K = 0.8			
	σ_1 (T/M^2)	σ_3 (T/M^2)	U(cms)	V(cms)	σ_1 (T/M^2)	σ_3 (T/M^2)	U(cms)	V(cms)	σ_1 (T/M^2)	σ_3 (T/M^2)	U(cms)	V(cms)
1.	-128.4	-4.0	0.07	0.26	-139.2	0.3	0.18	0.30	-212.8	-0.1	0.30	0.32
2.	-164.0	-4.0	0.09	0.32	-192.9	-0.2	0.22	0.36	-318.6	-22.5	0.36	0.40
3.	-223.8	-20.5	0.12	0.40	-389.8	-24.8	0.28	0.45	-385.4	-67.1	0.45	0.50
4.	-553.8	-95.9	0.21	0.55	-707.4	-126.6	0.42	0.63	-862.6	-155.6	0.64	0.71
5.	-164.8	17.8	0.43	0.76	-162.0	27.4	0.78	0.87	-159.6	37.3	0.94	0.90
6.	-136.9	6.0	0.54	0.86	-126.6	7.9	0.98	0.97	-116.6	9.9	1.41	1.07
7.	-61.6	1.7	0.73	1.06	-39.1	1.3	1.33	1.12	-16.7	1.0	1.94	1.18
8.	-5.0	6.4	0.73	1.08	-2.9	5.4	1.39	1.13	-1.1	4.6	2.04	1.19
9.	-111.0	1.6	0.76	1.22	-62.7	1.1	1.44	1.20	-46.0	-3.5	2.12	1.18
10.	-40.1	1.8	0.51	1.84	-14.6	1.5	1.10	1.40	0.8	11.2	1.68	1.17
11.	-0.6	10.0	0.33	1.69	-0.3	11.6	0.87	1.40	0.1	13.2	1.41	1.16
12.	-694.4	-108.4	0.18	2.07	-688.2	-105.8	0.66	1.83	-685.0	-100.3	1.15	1.60
13.	-88.2	-8.3	0.05	2.77	-187.1	-9.0	0.29	2.39	-283.2	-94.3	0.53	2.01
14.	-36.5	1.4	0.00	2.95	-152.5	0.3	0.00	2.53	-288.4	-0.8	0.00	2.10

areas around the opening does not vary significantly within the range of variation of K values. The relationship between K and σ_1 has been shown in Figure 4.10 and the corresponding variation in displacement values has been shown in figure 4.15. The deflected shape of the cavern has been shown in Fig.4.13.

4.6.3.3 The analysis with transformer hall could not be completed due to non-availability of disk space on Deck System 2050. However, the complete input data for the analysis has been generated through the programme.



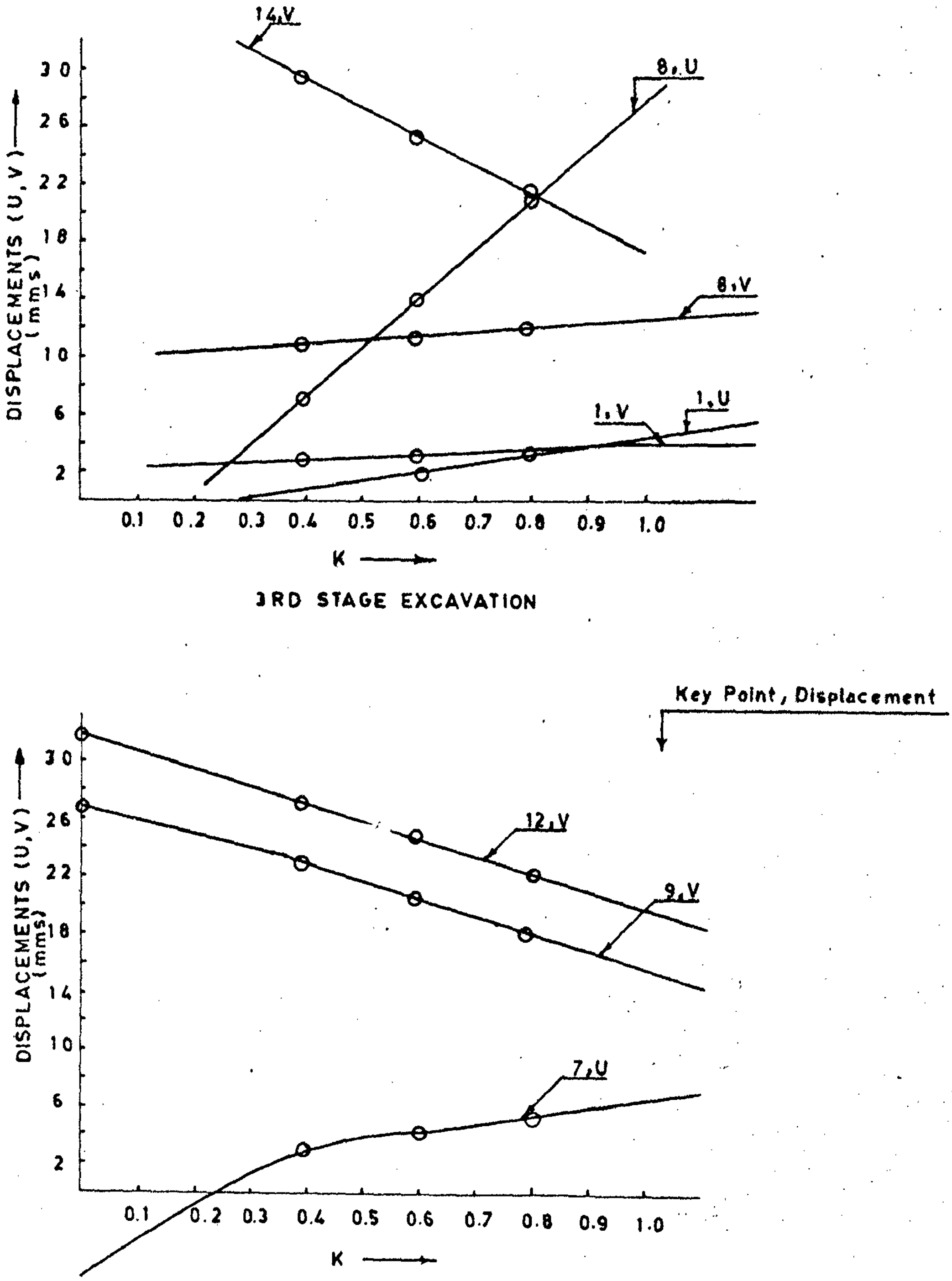
K= 0.6
Maximum Displacement(MMS)
Horizontal=14.4 Vertical=25.3



K= 0.4
Maximum Displacacent (mms)
Horizontal=7.6 Vertical= 29.5

3RD STAGE EXCAVATION
CONTOURS OF $\sigma 1/\sigma V$
(Radius of Arch=17M)

FIG. 4.14



3RD STAGE EXCAVATION

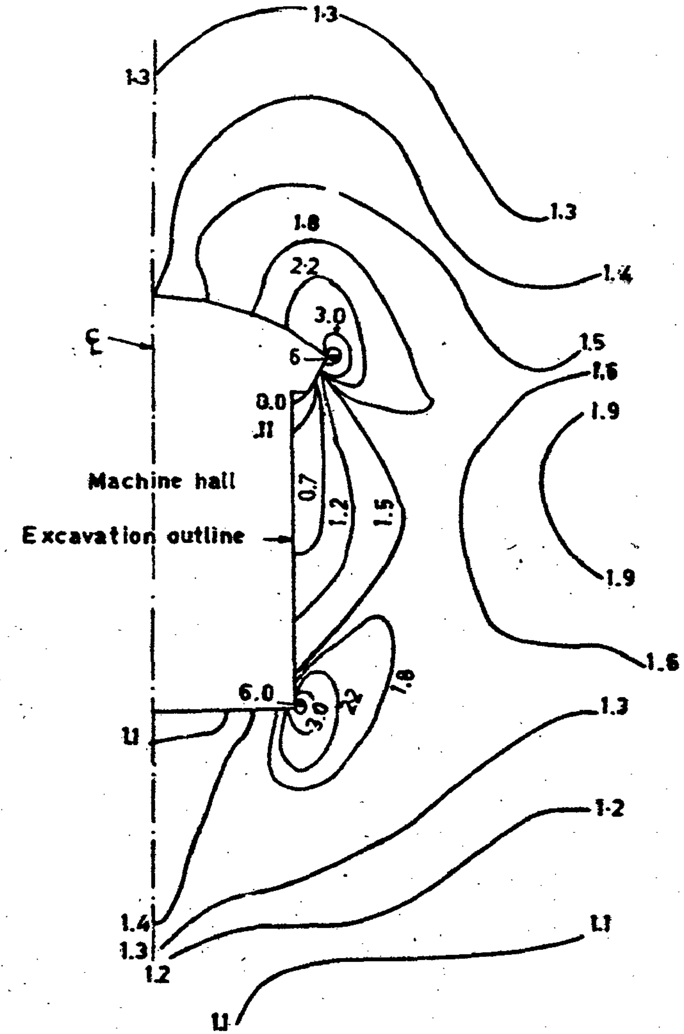
2 ND STAGE EXCAVATION

EFFECT OF K ON DISPLACEMENTS

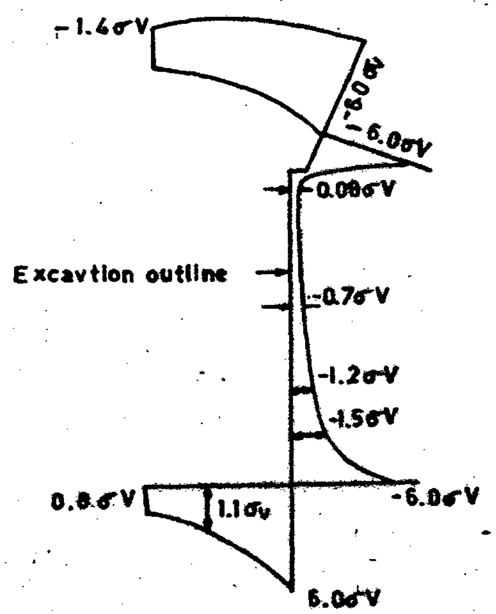
U = HORIZONTAL DISPLACEMENT

V = VARTICAL DISPLACEMENT

FIG. 4.15



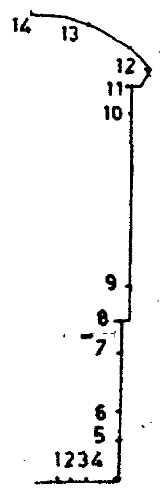
Contours of $\sigma 1/\sigma V$
(Maximum displacement =3.4 MM)



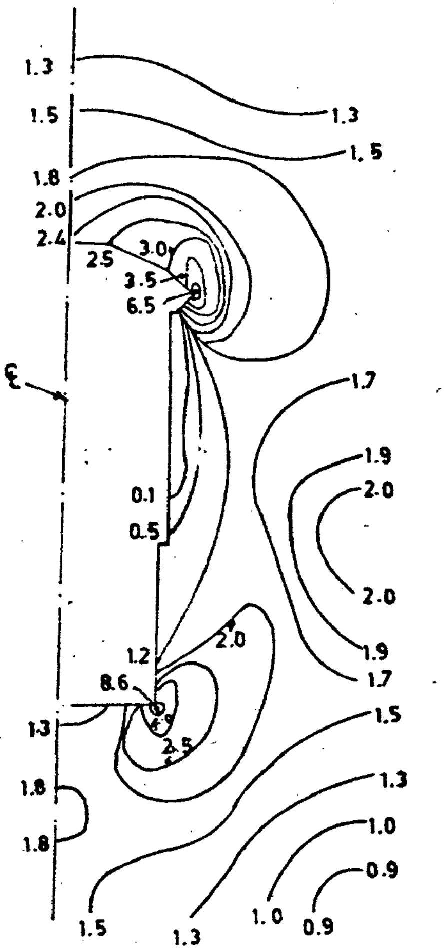
Surface stress profile
((-) indicates comprssion)

2 ND STAGE EXCAVATION
($K = 0.8$, $R = 17m$, $E = 2.0 \times 10^{6} T/m^2$)
($\sigma_v = 100T/m^2$)

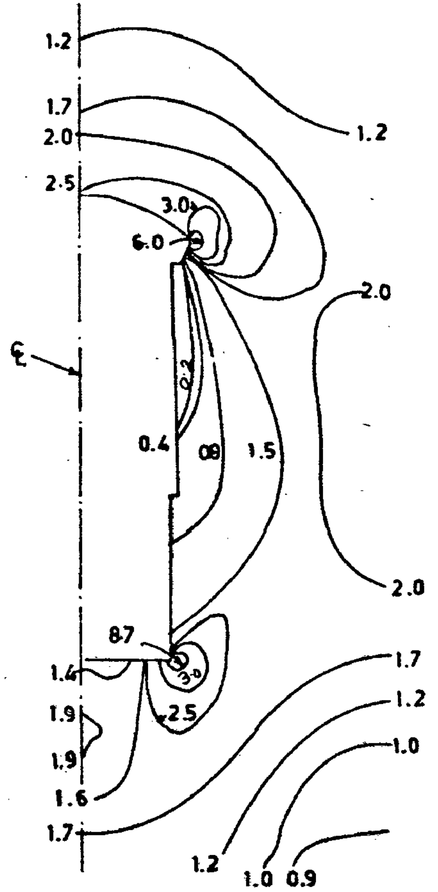
FIG 4.16



Key Points



Radius of Arch = 18M Span = 22.5M
Maximum Displacement = 21.0MM.



Radius of Arch = 17M Span = 20M
Maximum Displacement = 23.0MM

3RD STAGE EXCAVATION
(Contours of = σ_1/σ_v)
(K = 0.8 , σV = 100T/M^2)

FIG. 4.17

CHAPTER-5

CONCLUSIONS, RECOMMENDATIONS AND SCOPE OF FURTHER STUDY

5.1 CONCLUSIONS

The results of the analysis have been discussed in Chapter-4 . Following conclusions have been made on the basis of these discussions.

i) There is little variation in the principal stress distribution, around the opening, with the value of radius of roof arch ranging from 15m to 19m. The minimum boundary stress values have been noticed with radius equal to 17 metres.

ii) The pattern of stress distribution, around the opening, does not vary to a large extent with the provision of concrete arch in the roof. The opening is stable, even without concrete arch.

iii) The tensile stresses (small) develop at the crown and at the flat bottom of the cavern when the K value approaches zero.

iv) The values of compressive stress in the concrete arch decrease towards the crown, being maximum at the abutments.

v) The increase in the span of the opening by 2.5 metres does not result in excessive stress concentrations. The cavern is found to be stable with span equal to 22.5 metre.

vi) The horizontal displacement, along the vertical sides, increases with the increase in the K value. The vertical displacement at crown, on the other hand, decreases with the increase in K value.

vii) The increase in the modulus of elasticity of rock mass does not affect the redistribution of stresses. However, displacement values decrease significantly.

viii) The maximum absolute values of stress are noticed when the machine hall cavity is excavated upto the turbine pit level (El. 994.778).

ix) The stress concentration is localized at the arch abutment and bottom sharp corner of the cavern.

5.2 RECOMMENDATIONS

Based on the conclusions, following recommendations have been made -

i) The concrete arch for the roof is not required for the K value equal to 0.8.

ii) The width of the machine hall can be increased to accommodate the u/s control valves. This will result in considerable saving in costs of construction and auxiliary equipment like E.O.T. crane.

iii) The extensive investigations are necessary to define the insitu stress field more accurately. The stress measurement techniques described in chapter-2 may be used for investigations.

iv) If the actual K value is found to be less than 0.5 the concrete arch support in the roof will be required. The stresses in the arch do not warrant heavy reinforcement. Hence, nominal reinforcement may be provided.

v) For K value equal to or more than 1, the separate analysis will be required.

vi) The thickness of the concrete arch should be reduced towards the crown.

vii) The slight tensile stresses, developing at the haunch, can be accounted for by the provision of the shotcreting and rock bolting etc.

viii) The sharp corner at haunch of roof arch results in higher local stress concentration. The stress concentration can be reduced by avoiding the sharp corners. Similarly, the bottom sharp corner should be avoided during each stage of excavation.

ix) The extensive field investigations are required to study the joint distribution pattern around the opening. In case the joints are widely distributed, the finite element analysis, as described in Chapter- 3, should be carried out.

x) The behaviour of the caverns, consequent upon the excavation, should be monitored constantly. Such an exercise is necessary to correlate the results of the

Finite Element Analysis with the actual behaviour. Extensive instrumentation is, therefore, recommended to include all critical zones around the opening.

xi) The radius of the roof arch should be kept as 17 metres.

xii) In case, the rock mass properties show large variation from the assumptions of linear, isotropic and homogeneous strata, the analysis may be repeated. The requirements of rock supports can also be worked out on the basis of guidelines given in reference [8]

5.3 SCOPE OF FURTHER STUDY

The analysis done in the present work is restricted to the machine hall and transformer hall caverns. The analysis of two caverns, as discussed in para 4.6.3.3 can be completed to study the effect of width of rock pillar between the openings and K. The computer programme to generate the input data for finite element analysis can be suitably modified to include the surge chamber opening, located d/s of the transformer hall.

The stress pattern around the opening, keeping in view the effect of caverns on each other, should be analyzed. The best sequence of excavation should be finalised on the basis of extensive stress analysis of different excavation sequences.

In case, the investigations indicate jointed rock mass or any other structural defect in the rock mass surrounding the cavern, the analysis should be done to study their effect on the stability of the caverns.

Further, analysis with different values of width of influence zone, can indicate its limiting value. Extensive investigations, particularly in the limiting zone, are necessary to make a rational analysis.

R E F E R E N C E S

1. Charles Jacgers, ' Rock Mechanics and Engineering'.
2. Fumagalli. E, ' Statistical and Geomechanical Models'.
3. Hinton E. and Owen D.R.J., ' Finite Element Programming'.
4. International Society on Rock Mechanics, Symposium, Cambridge, British Geotechnical Society, London.
5. International Society on Rock Mechanics, Symposium, Melbourne, 1983, Vol.II.
6. International Society on Rock Mechanics, Proceedings, Montreux (Suisse), 1979.
7. Jumixis, A.R., ' Rock Mechanics'.
8. 'Manual on Planning and Design of Hydraulic Tunnels', Central Board of Irrigation and Power, Publication No.178.
9. 'Numerical Methods in Geomechanics', Proceedings of 2nd International Conference, Newyork, 1976, Vol. I, Vol.II and Vol.III.
10. Overt L. and Duvall, ' Rock Mechanics and Design of Structures in Rock'.
11. Proctor and White, ' Rock Tunnelling with Steel Supports'.
12. Shiva Prasad, ' Structural Analysis of Varahi Underground Power House', M.E. Dissertation submitted at WRDTC, University of Roorkee, 1982.
13. S. Desai and F. Abel, ' Introduction to the Finite Element Methods'.
14. 'Symposium on Underground Rock Chambers', American Society for Civil Engineers, 1971.
15. Zienkiewicz, ' The Finite Element Method in Structural and Continnum Mechanics'.

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C                               ANNEXURE-I
C        PROGRAM FOR GENARATION OF FEM DATA FOR POWER HOUSE CAVERN
         INTEGER GN,BN,BNX,BNY
         REAL PLOAD,PX,PY
         DIMENSION X(300),Y(300),NN(900),PLOAD(300,2),NF(300),BNX(100),
     1BNY(100),NX(9),BN(100),TITLE(17)
         OPEN(UNIT=20,DEVICE='DSK',FILE='DL2.DAT')
         OPEN(UNIT=22,DEVICE='DSK',FILE='D4.DAT')
         NT=37
         COMMON/AA/XO,YO,GX(900,8),GY(900,8),XN(3000),YN(3000),GN(610,9)
         COMMON/BB/PX,PY,NC
         NEE=0
         XO=0.
         YO=0.
         NOI=0
         READ(20,*)X1,Y1,X2,Y2,X3,Y3,NEXA
         READ(20,*) WCS,RA,TA,N,PW,NC,PX,PY
  380    Q=WCS/RA
         THETA=ASIN(Q)
         REXA=RA+TA
         R1=WCS/(SIN(THETA)/COS(THETA))
         ORD=REXA-R1
         XCEN=XO
         YCEN=-R1+YO
         ROUT=REXA+PW
         AN=N
         ANGELE=THETA/AN
         STEP=PW/7.
         STEP1=STEP/3.
         MM=208
         MM1=208+8*N
         LN=0
  100    AN=0
         DO 10 I=MM,MM1,8
         ANG=THETA-ANGELE*AN
```

```
           AN=AN+1
           X(I)=ROUT*SIN(ANG)+XO
           Y(I)=ROUT*COS(ANG)-R1+YO
     10    CONTINUE
           LN=LN+1
           IF(LN.GT.7) GO TO 200
           MM=MM-1
           MM1=MM1-1
           GO TO(11,12,12,12,12.14,14)LN
     11    ROUT=ROUT-2.*STEP
           GO TO 100
     12    ROUT=ROUT-STEP
           GO TO 100
     14    ROUT=ROUT-STEP1
           GO TO 100
     200   THET=3.1415926/2.-THETA
           THET1=THET/2.
           SET=0.
           DO 110 I=195,199
           SET=SET+STEP
           X(I)=SET*COS(THET1)+WCS+XO
           Y(I)=SET*SIN(THET1)+YO
     110   CONTINUE
           SET=SET+2.*STEP
           X(200)=SET*COS(THET1)+WCS+XO
           Y(200)=SET*SIN(THET1)+YO
           SET=STEP/2.
           X(194)=SET*COS(THET1)+WCS+XO
           Y(194)=SET*SIN(THET1)+YO
           HTAB=Y1-Y2
           HTABST=HTAB/4.
           ABST1=HTABST/2.
           Y(184)=YO
           Y(130)=Y(184)-HTAB
           DO 20 K=9,45,9
           MN=130+K
           IF(K.LE.18)GO TO 15
```

```
      GO TO 16
15    Y(MN)=Y(MN-9)+ABST1
      GO TO 20
16    IF(K.LE.36) GO TO 17
      GO TO 18
17    Y(MN)=Y(MN-9)+HTABST
      GO TO 20
18    Y(MN)=Y(MN-9)+ABST1
20    CONTINUE
      X(184)=WCS-STEP1+XO
      KN=1
      M1=184
      M2=192
      DO 30 K=M1,M2
      MN=184+KN
      IF(KN.LE.3) GO TO 31
      GO TO 32
31    X(MN)=X(MN-1)+STEP1
      GO TO 30
32    IF(KN.LE.7) GO TO 33
      GO TO 34
33    X(MN)=X(MN-1)+STEP
      GO TO 30
34    X(MN)=X(MN-1)+2.*STEP
30    KN=KN+1
      CONTINUE
      MN1=130
      MN2=138
      DO 50 K1=MN1,MN2
      MN3=K1+54
      DO 50 K2=K1,MN3,9
      X(K2)=X(MN3)
50    CONTINUE
      MN4=184
      DO 55 K1=MN1,MN4,9
      MN5=K1+9-1
      DO 55 K2=K1,MN5
```

```
               Y(K2)=Y(K1)
     55        CONTINUE
               X(129)=WCS-2.*STEP1+X0
               ML1=89
               ML2=89+9
               HTBC=Y2-Y3
               IF(HTBC.LE.0.) GO TO 300
               GO TO 54
     300       HTB1=ABST1*2.
               HTB2=ABST1
               Y(89)=Y(130)
               GO TO 89
     54        DO 60 K1=ML1,ML2
               ML3=K1+4.*10
               DO 60 K2=K1,ML3,10
               X(K2)=X(ML3)
     60      CONTINUE
                HTB1=HTBC/3.
                HTB2=HTB1/2.
                KK=129
                Y(KK)=Y(KK+1)
                Y(119)=Y(129)-HTB2
                Y(109)=Y(119)-HTB1
                Y(99)=Y(109)-HTB1
                Y(89)=Y(99)-HTB2
                DO 75 K1=ML1,KK,10
                ML5=K1+10-1
                DO 75 K2=K1,ML5
                Y(K2)=Y(K1)
      75        CONTINUE
      89        KK=85
               IF(NDI.EQ.2)HTB1=HTB2
                Y(85)=Y(89)
                Y(29)=Y(89)-4.*HTB2
                Y(15)=Y(29)-HTB1
                Y(1)=Y(29)-HTB1
                Y(1)=Y(15)-HTB1
```

```
           Y(43)=Y(89)-3.*HTB2
           Y(57)=Y(89)-2.*HTB2
           Y(71)=Y(89)-HTB2
          STEP2=WCS-5.*STEP1
           X(86)=STEP2+XO
           X(87)=X(86)+STEP1
           X(88)=X(87)+STEP1
           DO 95 K1=1,KK,14
           KK1=K1+13
           DO 95 K2=K1,KK1
           Y(K2)=Y(K1)
  95       CONTINUE
          X(KK)=XO
          IF(HTBC.LE.0) GO TO 90
          DO 105 K1=1,14
          K2=K1+84
          DO 105 K3=K1,K2,14
          X(K3)=X(K2)
 105      CONTINUE
          GO TO 91
 90       DO 106 K1=1,4
          K2=K1+84
          DO 106 K3=K1,K2,14
          X(K3)=X(K2)
 106      CONTINUE
          DO 107    K1=5,14
          K2=K1+84
          K4=K1+124
          DO 107 K3=K1,K2,14
          X(K3)=X(K4)
 107      CONTINUE
 91       Y(193)=(Y(201)+Y(185))/2.
          X(193)=(X(201)+X(185))/2.
          MI=1
 120      GO TO (111,112,113,114)MI
          IF(MI.GT.4) GO TO 121
 111      IN=1
```

```
      JM=12
      JK=6
      GO TO 119
 112  IF(HTBC.LE.0.) GO TO 301
      IN=89
      JM=8
      JK=4
      GO TO 119
 301  MI=MI+1
      GO TO 120
 113  IN=130
      JM=7
      JK=6
      GO  TO 119
 114  IN=185
      JM=6
      JK=N+2
 119  JJ=JM+3
      JN=JM+2
      DO 150 KI=1,JK
      KK=IN
       KM=IN+JM
      DO   115 I=KK,KM
      NEE=NEE+1
         NN(NEE)     =NEE
      GX(NEE,1)=X(I)
      GX(NEE,3)=X(I+1)
      GX(NEE,5)=X(I+JJ)
      GX(NEE,7)=X(I+JN)
      GX(NEE,2)=(GX(NEE,1)+GX(NEE,3))/2.
      GX(NEE,4)=(GX(NEE,3)+GX(NEE,5))/2.
      GX(NEE,6)=(GX(NEE,5)+GX(NEE,7))/2.
      GX(NEE,8)=(GX(NEE,7)+GX(NEE,1))/2.
      GY(NEE,1)=Y(I)
      GY(NEE,3)=Y(I+1)
      GY(NEE,5)=Y(I+JJ)
      GY(NEE,7)=Y(I+JN)
```

```
          GY(NEE,2)=(GY(NEE,1)+GY(NEE,3))/2.
          GY(NEE,4)=(GY(NEE,3)+GY(NEE,5))/2.
          GY(NEE,6)=(GY(NEE,5)+GY(NEE,7))/2.
          GY(NEE,8)=(GY(NEE,7)+GY(NEE,1))/2.
115       CONTINUE
         IN=IN+JN
150       CONTINUE
          MI=MI+1
          GO TO 120
121       IF(NC.EQ.1) GO TO 225
          IF(NOI.GE.1) GO TO 311
          KL=208+8*N
          DO 350 I=1,KL
          X(I)=-X(I)
350       CONTINUE
          NOI=1
          MI=1
          GO TO 120
311       IF(NOI.EQ.2) GO TO 220
          NOI=2
          YO=YO+5
          XO=XO+PW+18.5
          RA=RA-0.5
          WCS=9.5
          TA=TA-0.2
          NEE1=NEE/2
          NEE2=2*NEE1
          GO TO 380
220       KL=NEE
          KL1=NEE-(7*N-1)
          DO 400 I=KL1,KL
          NEE=NEE+1
          DO 400 J=1,8
          GX(NEE,J)=-GX(I,J)+2.*XO
          GY(NEE,J)=GY(I,J)
400       CONTINUE
          KM=KL-140-(7*N-1)
```

```
      DO 410 K=1,6
      KM1=KM+7
      DO 420 JJ=KM,KM1
      NEE=NEE+1
      DO 420 J=1,8
      GX(NEE,J)=-GX(JJ,J)+2.*XO
      GY(NEE,J)=GY(JJ,J)
420   CONTINUE
      KM=KM+13
410   CONTINUE
      KN=KL-(7*N-1)-46
      DO 430 K=1,4
      KN1=KN+2
      DO 440 JJ=KN,KN1
      NEE=NEE+1
      DO 440 J=1,8
      GX(NEE,J)=-GX(JJ,J)+2.*XO
      GY(NEE,J)=GY(JJ,J)-HTABST
440   CONTINUE
      KN=KN+8
430   CONTINUE
      DO 441 JJ=1,3
      NEE=NEE+1
      DO 441 J=1,8
      GX(NEE,J)=GX(NEE-9,J)
      GY(NEE,J)=GY(NEE-9,J)+HTABST*2
441   CONTINUE
      DO 450 I=13,78,13
      DO 450 J=3,5
      GX(I,J)=GX(I,J)-STEP
450   CONTINUE
      DO 460 I=86,126,8
      DO 460 J=3,5
      GX(I,J)=GX(I,J)-STEP
460   CONTINUE
      DO 470 I=133,154,7
      DO 470J=2,6
```

```
      GX(I,J)=0.
      GY(I,J)=0.
470   CONTINUE
      KM=KL+4
      M=KL-7*N-13
490      M1=M+2
      DO 480 I=M,M1
      NEE=NEE+1
      DO 480 J=1,8
      GX(NEE,J)=-GX(I,J)+2.*X0
      GY(NEE,J)=GY(I,J)
480   CONTINUE
      M=M+7
      IF(M.EQ.(KL-7*N-6)) GO TO 490
      JK1=NEE1-7*N-8
      JK2=JK1+7
      JK3=JK1+1
      JK4=NEE1-7
      GX(JK1,5)=GX((NEE-3),3)
      GY(JK1,5)=GY((NEE-3),3)
      GX(JK2,3)=GX(JK1,5)
      GY(JK2,3)=GY(JK1,5)
      GX(JK1,6)=(GX(JK1,5)+GX(JK1,7))/2.
      GY(JK1,6)=(GY(JK1,5)+GY(JK1,7))/2.
      GX(JK2,5)=GX((NEE-3),5)
      GY(JK2,5)=GY((NEE-3),5)
      GX(JK3,7)=GX(JK1,5)
      GY(JK3,7)=GY(JK1,5)
      GX(JK3,5)=GX(JK1,5)+STEP
      GY(JK3,5)=GY(JK1,5)
      GY(JK3,3)=GY(JK1,3)
      GX(JK3,3)=GX(JK3,5)
      CALL GLOBAL(JK3)
      CALL GLOBAL(JK2)
      KM1=NEE1-7*N
      GX(KM1,1)=GX(KM1-1,7)
      GY(KM1,1)=GY(KM1-1,7)
```

```
      GX(KM1,3)=GX(KM1-1,5)
      GY(KM1,3)=GY(KM1-1,5)
      GX(KM1,5)=GX(KM,1)
      GY(KM1,5)=GY(KM,1)
      GX(KM1,7)=GX(146,3)
      GY(KM1,7)=GY(146,3)
      CALL GLOBAL(KM1)
      KM1=NEE1-14
      GX(KM1,3)=GX(KM+1,1)
      GY(KM1,3)=GY(KM+1, 1)
      GX(KM1,5)=GX(KM+1,3)
      GY(KM1,5)=GY(KM+1,3)
      CALL GLOBAL(KM1)
      KM1=KM1-7
      GX(KM1,3)=GX(KM,1)
      GY(KM1,3)=GY(KM,1)
      GX(KM1,5)=GX(KM,3)
      GY(KM1,5)=GY(KM,3)
      CALL GLOBAL(KM1)
      KM=KL+6
401   NEE=NEE+1
      GX(NEE,1)=GX(JK4,1)
      GY(NEE,1)=GY(JK4,1)
      GX(NEE,5)=GX(KM,3)
      GY(NEE,5)=GY(KM,3)
      GX(NEE,3)=GX(KM,1)
      GY(NEE,3)=GY(KM,1)
      GX(NEE,7)=GX(JK4,3)
      GY(NEE,7)=GY(JK4,3)
      CALL GLOBAL(NEE)
      NEE=NEE+1
      GX(NEE,1)=GX(JK4,3)
      GY(NEE,1)=GY(JK4,3)
      GX(NEE,3)=GX(KM,1)
      GY(NEE,3)=GY(KM,1)
      GX(NEE,5)=GX(KM,3)
      GY(NEE,5)=GY(KM,3)
```

```
          GX(NEE,7)=GX(KM,3)-0.4
          GY(NEE,7)=GY(KM,3)+0.4
          CALL GLOBAL(NEE)
          IN=0
          NEE1=NEE1+1
    201   DO 101 I=NEE1,NEE2
          GGX=GX(I,1)
          GX(I,1)=GX(I,3)
          GX(I,3)=GGX
          GGX=GX(I,8)
          GX(I,8)=GX(I,4)
          GX(I,4)=GGX
          GGX=GX(I,7)
          GX(I,7)=GX(I,5)
          GX(I,5)=GGX
          GGY=GY(I,1)
          GY(I,1)=GY(I,3)
          GY(I,3)=GGY
          GGY=GY(I,8)
          GY(I,8)=GY(I,4)
          GY(I,4)=GGY
          GGY=GY(I,7)
          GY(I,7)=GY(I,5)
          GY(I,5)=GGY
    101   CONTINUE
          NEE1=KL+1
          NEE2=NEE
          IN=IN+1
          IF(IN.EQ.1)GO TO 201

          CALL NUMBER(NEE)
          DO 125 I=1,26
          DO 125 J=1,8
          GX(I,J)=GX(I+26,J)
          GY(I,J)=GY(I+26,J)
    125   CONTINUE
          DO 117 I=27,47
```

```
          DO 117 J=1,8
          GX(I,J)=GX(I+52,J)
          GY(I,J)=GY(1,J)
 117      CONTINUE
          NEE=NEE-5
          DO 118 I=48,NEE
          DO 118 J=1,8
          GX(I,J)=GX(I+5,J)
          GY(I,J)=GY(I+5,J)
 118      CONTINUE
          DO 219 I=1,47
          DO 229 J=1,3
          GY(I,J)=GY(I,J)-HTABST
 229      CONTINUE
          II=I
          CALL GLOBAL(II)
C         TYPE*,(GY(I,J),J=1,3)
 219      CONTINUE
 225      IF(NEXA.EQ.1) CALL NARCH(NEE)
          IF(NC.EQ.2) GO TO 1001
        CALL NNODE(120,NLN,PLOAD,NF)
          CALL NELE(120)
C         CALL PLOTS(0,0,5)
C         CALL PLOTN(NEE)
          PRINT 250
 250    FORMAT(1X,'ELEMENT NO.',21X,'GLOBAL COORD.X',46X,'GLOBAL COORD
        PRINT 251,(NN(I),(GX(I,J),J=1,8),(GY(I,J),J=1,8),I=1,30)
 251    FORMAT(/5X,I3,5X,8F7.3,5X,8F7.3)
         PRINT 252
 252    FORMAT(2(3X,'ELEMENT NO',14X,'COORDINATE X',14X,'COORDINATE Y'
        PRINT 253,(I,X(I),Y(I),I=1,30)
 253    FORMAT(/2(6X,I3,20X,F8.3,18X,F8.3))
 1001     CALL NCTWO(NEE,NLN,PLOAD,NF)
          TYPE999
999       FORMAT(5X,'OUT OF NCTWO')
         READ(20,5)TITLE
 5        FORMAT(1X,17A4)
```

```
      WRITE(22,5)TITLE
        READ(20,*)(NX(I),I=1.9)
      WRITE(22,*)(NX(I),I=1,9)
      READ(20,*)E,XU,GAMA
      WRITE(22,*)E,XU,GAMA
      READ(20,*)(BN(I),I=1,95)
      WRITE(22,3)(BN(I),I=1,95)
 3    FORMAT(15(1X,I4))
      READ(20,*)(BNX(I),I=1,95)
      WRITE(22,3)(BNX(I),I=1.95)
      READ(20,*)(BNY(I),I=1,95)
      WRITE(22,3)(BNY(I),I=1.95)
     WRITE(22,1)(I,(GN(I,J),J=1,9),I=1,598)
  1   FORMAT((I4,9(1X,I5)))
      WRITE(22,2)(L,  XN(L),YN(L),L=1,1938)
 2    FORMAT(4(1X,I4,1X,F6.2.1X,F6.2))
      READ(20,*) ACLX,ACLY
      WRITE(22,*) ACLX,ACLY
C      READ(20,*)(LNOD(I),I=1,NT)
C      WRITE(22,*)(LNOD(I).I=1,NT)
C      READ(20,*)(XLNOD(I).I=1,NT)
C      WRITE(22,*)(XLNOD(I),I=1,NT)
C      READ(20,*)(YLNOD(I).I=1,NT)
C      WRITE(22,*)(YLNOD(I),I=1,NT)
      WRITE(22,3)NLN,(NF(I),I=1,NLN)
      WRITE(22,4)((PLOAD(I.J),I=1,NLN),J=1,2)
 4    FORMAT(8(1X,F8.1))
C .   TYPE*,NLN,(NF(I),I=1.NLN)
      CLOSE(UNIT=20)
      CLOSE(UNIT=22)
C     CALL PLOT(0.,0.,999)
     STOP
      END
C      SUBROUTINE PLOTN(NEE)
      SUBROUTINE PLOTN(NEE)
      COMMON/AA/XO,YO,GX(900.8),GY(900.8),XN(3000),YN(3000)
      DO 157 I=1,NEE
```

```
            DO 157 J=1,8
            GX(I,J)=GX(I,J)/4.5
            GY(I,J)=GY(I,J)/4.5
  157       CONTINUE
C           CALL PLOT(XO,YO,+3)
            DO 10 I=1,NEE
C            CALL PLOT(GX(I,1),GY(I,1),+3)
            DO 20 J=1,8
C            CALL PLOT(GX(I,J),GY(I,J),+2)
  20        CONTINUE
C           CALL PLOT(GX(I,1),GY(I.1),+2)
  10        CONTINUE
            RETURN
            END
C            SUBROUTINE GLOBAL
            SUBROUTINE GLOBAL(NEE)
            COMMON/AA/XO,YO,GX(900.8),GY(900,8),XN(3000),YN(3000)
            GX(NEE.2)=(GX(NEE,1)+GX(NEE,3))/2.
            GX(NEE.4)=(GX(NEE,3)+GX(NEE,5))/2.
            GX(NEE.6)=(GX(NEE,7)+GX(NEE,5))/2.
            GX(NEE.8)=(GX(NEE,7)+GX(NEE,1))/2.
            GY(NEE.2)=(GY(NEE,1)+GY(NEE,3))/2.
            GY(NEE.4)=(GY(NEE,3)+GY(NEE,5))/2.
            GY(NEE.6)=(GY(NEE,5)+GY(NEE,7))/2.
            GY(NEE.8)=(GY(NEE,7)+GY(NEE,1))/2.
            RETURN
            END
            SUBROUTINE NUMBER(NEE)
            DIMENSION GGX(20),GGY(20),GLX(300,8),GLY(300,8)
            COMMON/AA/XO,YO,GX(900.8),GY(900,8)
            NB=00
            LNX=0
            M=127
            L=168
  402       DO 401 I=M,L
            NB=NB+1
            DO 401 J=1,8
```

```
          GLX(NB,J)=GX(I,J)
          GLY(NB,J)=GY(I,J)
    401   CONTINUE
          LNX=LNX+1
          M=M+168
          L=L+168
          IF(M.EQ.295) GO TO 402
          IF(LNX.EQ.3) GO TO 403
          M=447
          GO TO 402
    403   M=505
          L=532
    405   DO 404 I=M,L
          NB=NB+1
          DO 404 J=1,8
          GLX(NB,J)=GX(I,J)
          GLY(NB,J)=GY(I,J)
    404   CONTINUE
          M=M+87
          L=NEE
          IF(M.EQ.592) GO TO 405
          ZX=NEE-NB
          NE=NEE
          I=1
          GO TO 606
    60    N=NEE
          IF(GY(I,1).LT.0.) GO TO 50
    31    IF(GY(N,1).LT.0.) GO TO 12
          NEE=NEE-1
          N=NEE
          GO TO 31
    12    DO 55 J=1,8
          GGY(J)=GY(N,J)
          GY(N,J)=GY(I,J)
          GY(I,J)=GGY(J)
          GGX(J)=GX(N,J)
          GX(N,J)=GX(I,J)
```

```
          GX(I,J)=GGX(J)
 55       CONTINUE
          NEE=NEE-1
 50       I=I+1
          IF(I.LE.(NEE-1)) GO TO 60
 606      K=1
 35       II=K
          YMIN=GY(II,1)
          DO 1 I=II,NEE
          KM=I+1
          IF(YMIN-GY(KM,1))1,1,2
 2        YMIN=GY(KM,1)
 1        CONTINUE
C         TYPE*,YMIN,K
          DO 10 I=II,NEE
          IF(GY(I,1).NE.YMIN) GO TO 10
          DO 15 J=1,8
          GGY(J)=GY(K,J)
          GY(K,J)=GY(I,J)
          GY(I,J)=GGY(J)
          GGX(J)=GX(K,J)
          GX(K,J)=GX(I,J)
          GX(I,J)=GGX(J)
 15       CONTINUE
          K=K+1
 10       CONTINUE
          K=K-1
          K1=K-1
          DO 30 I=II,K1
          IN=I+1
          DO 20 KK=IN,K
          IF(GX(I,1)-GX(KK,1)) 20 ,20,11
 11       DO 25 J=1,8
          GGX(J)=GX(I,J)
          GX(I,J)=GX(KK,J)
          GX(KK,J)=GGX(J)
          GGY(J)=GY(I,J)
```

```
          GY(I,J)=GY(KK,J)
          GY(KK,J)=GGY(J)
   25     CONTINUE
   20     CONTINUE
   30     CONTINUE
          K=K+1
          IF(YMIN.GE.0)GO TO 45
          GO TO 35
   45     NEE=NE
          DO 407 I=1,NB
          DO 407 J=1,8
          GX((ZX+I),J)=GLX(I,J)
          GY((ZX+I),J)=GLY(I,J)
   407    CONTINUE
            RETURN
          END
C          SUBROUTINE NNODE
          SUBROUTINE NNODE(NEE,NLN,PLOAD,NF)
          COMMON/AA/XO,YO,GX(900,8),GY(900,8),XN(3000),YN(3000)
          COMMON/BB/PX,PY,NC
          DIMENSION PLOAD(300,2),NF(300)
          IF(NC.GT.1) GO TO 8
          DO 1 I=1,7
          NEE=NEE+1
          DO 1 J=5,7
          M=8-J
          GX(NEE,M)=GX((NEE-7),J)
          GY(NEE,M)=GY((NEE-7),J)
   1      CONTINUE
   8      NLN=0
          L=0
          K=1
   18        I=K
          DO 10 J=1,3
          L=L+1
          XN(L)=GX(I,J)
          YN(L)=GY(I,J)
```

```
 10     CONTINUE
        M=I+1
        IF(GX(M,1).EQ.0)GO TO 100
         N=0
        DO 11I=M,NEE
        K=K+1
        N=N+1
        IF(GX(I,1).GT.GX((I-1).1)) GO TO 12
        GO TO 14
 12        DO 20 J=2,3
        L=L+1
        XN(L)=GX(I,J)
        YN(L)=GY(I,J)
 20     CONTINUE
        GO TO 11
 14     NLN=NLN+1
        NF(NLN)=L
C       TYPE*,NLN
        K1=K-N
        DO 15 J=4,8,4
        IF(J.EQ.8) GO TO 19
        N=8
        GO TO 30
 19        N=4
 30     L=L+1
        XN(L)=GX(K1,N)
        YN(L)=GY(K1,N)
 15       CONTINUE
        K=K-1
        K2=K1+1
        DO 16 II=K2,K
        L=L+1
        XN(L)=GX(II,4)
        YN(L)=GY(II,4)
 16     CONTINUE
        NLN=NLN+1
        NF(NLN)=L
```

```
           IF(GX(I,1).NE.GX((I-N).7))GO TO 25
           GO TO 17
   25      DO 21 N=K1,K
           IF(GX(I,1).EQ.GX(N,7)) GO TO 17
           DO 21 J=1,2
           N1=8-J
           L=L+1
           XN(L)=GX(N,N1)
           YN(L)=GY(N,N1)
  21       CONTINUE
  11       CONTINUE
  17       K=K+1
            IF(I.LT.NEE)GO TO 18
           GO TO 28
  100      DO 101 I=M,NEE
           DO 101 J=2,3
           L=L+1
           XN(L)=GX(I,J)
           YN(L)=GY(I,J)
  101      CONTINUE
  28       NLN=NLN+1
           NF(NLN)=L
           DO 40 I=1,NLN
           IF(I.EQ.1) GO TO 23
           IF(I.EQ.NLN) GO TO 26
           NP=NF(I-1)
           NZ=NF(I+1)
           GO TO 24
  23       NP=NF(1)
           NZ=NF(2)
           GO TO 24
  26       NZ=NF(I)
           NP=NF(I-1)
  24       J=2
           PLOAD(I,J)=-PX*(XN(NP)-XN(NZ))/2
C          TYPE*,PLOAD(I,J)
           J=1
```

```
      PLOAD(I,J)=-PY*(YN(NZ)-YN(NP))/2
 40   CONTINUE
      RETURN
      END
C     SUBROUTINE(NELE(NEE))
      SUBROUTINE NELE(NEE)
      INTEGER GN
      COMMON/AA/XO,YO,GX(900.8),GY(900,8),XN(3000),YN(3000),GN(610,
      M=1
      DO 10 I=1,NEE
      K=M+240
      DO 30 J=1,8
      DO 30 L=M,K
      IF(GX(I,J).EQ.XN(L).AND.GY(I,J).EQ.YN(L))GO TO 20
      GO TO 30
 20   GN(I,J)=L
C     TYPE*,GN(I,J)
 30   CONTINUE
      M=GN(I,3)
 10   CONTINUE
      DO 21 I=1,NEE
     GN(I,9)=1
 21   CONTINUE
      RETURN
      END
      SUBROUTINE NARCH(NEE)
      COMMON/AA/XO,YO,GX(900.8),GY(900.8),XN(3000),YN(3000),GN(610,
      NEE=120
      DO 10 I=79,120
      DO 10 J=1,8
      GX(I,J)=GX(I+48,J)
      GY(I,J)=GY(I+48,J)
 10   CONTINUE
      DO 20 I=1,78
      DO 20 J=1,8
      GY(I,J)=GY(I,J)+20
 20   CONTINUE
```

```
      RETURN
      END
C      SUBROUTINE NCTWO
      SUBROUTINE NCTWO(NEE,NLN,PLOAD,NF)
      INTEGER GN
      COMMON/AA/XO,YO,GX(900,8),GY(900,8),XN(3000),YN(3000),GN(610,
      COMMON/BB/PX,PY,NC
      DIMENSION GGX(610,8),GGY(610,8),GNR(610,8),PLOAD(100,2),NF(1(
      OPEN(UNIT=23,DEVICE='DSK',FILE='NDN,DAT')
      READ(23,*) (I,(GN(I,J).J=1,8),MI=1,NEE)
C      TYPE*, (I,(GN(I,J),J=1,8),I=1,NEE)
      DO 10 I=418,NEE
      READ(23,*)NOR,MOR
      DO 10 J=1,8
      GGX(MOR,J)=GX(NOR,J)
      GGY(MOR,J)=GY(NOR,J)
      GNR(MOR,J)=GN(NOR,J)
 10   CONTINUE
      DO 20 I=418,NEE
      DO 20 J=1,8
     GX(I,J)=GGX(I,J)
      GY(I,J)=GGY(I,J)
      GN(I,J)=GNR(I,J)
 20     CONTINUE
      DO 101 I=1,NEE
      TYPE*,I
      DO 101 J=1,8
      L=GN(I,J)
      XN(L)=GX(I,J)
      YN(L)=GY(I,J)
C     TYPE*,I
 101      CONTINUE
      READ(23,*)NLN,(NF(I).I=1,NLN)
      DO 40 I=1,NLN
      IF(I.EQ.1) GO TO 23
      IF(I.EQ.NLN) GO TO 26
      NP=NF(I-1)
```

```
            NZ=NF(I+1)
            GO TO 24
   23       NP=NF(1)
            NZ=NF(2)
            GO TO 24
   26       NZ=NF(I)
            NP=NF(I-1)
   24       J=2
            PLOAD(I,J)=-PX*(XN(NP)-XN(NZ))/2
C           TYPE*,PLOAD(I,J)
            J=1
            PLOAD(I,J)=-PY*(YN(NZ)-YN(NP))/2
            TYPE*,I
   40      CONTINUE
            CLOSE(UNIT=23)
            RETURN
            END
```