THREE DIMENSIONAL ANALYSIS OF SURFACE POWER STATION BY FINITE ELEMENT METHOD

A DISSERTATION

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

of

MASTER OF TECHNOLOGY

in

WATER RESOURCES DEVELOPMENT

By

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CANDIDATE'S DECLARATION

I do hereby declare that the dissertation entitled "**THREE DIMENSIONAL ANALYSIS OF SURFACE POWER STATION BY FINITE ELEMENT METHOD**" is being submitted by me in partial fulfillment of requirement for the award of degree of "**Master of Technology** in **WATER RESOURCES DEVELOPMENT (CIVIL)**" and submitted in the Water Resources Development and Management Department, Indian Institute of Technology, Roorkee, is an authentic record of my own work carried out during the period from July, 2004 to June 2005 under the guidance of **Dr. B.N. Asthana**, Visiting professor, **Prof. Gopal Chauhan,** Professor, Water Resources Development and Management Department, and **Dr.Yogendra Singh,** Assistant Professor, Earthquake Engineering Department, Indian Institute of Technology, Roorkee.

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

Vivek Tripathi

Dated: 29 June, 2005
Place: IITR, Roorkee

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

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Notations		Details
1.	ρ	Density of concrete
2.	ν	Poison's Ratio
3.	cm.	Centimeter
4.	DOF	Degrees of freedom
5.	E	Young's Modulus
6.	El.	Elevation
7.	FX	Force in x-direction
8.	FY	Force in y-direction
9.	FZ	Force in z-direction
10.	kg.	Kilogram
11.	S1 or σ_1	First Principal stress
12.	S2 or σ_2	Second Principal stress
13.	S3 or σ_3	Third principal stress
14.	SX or σ_x	Normal stress in X-direction
15.	SXY or σ_{xy}	Shear stress in X-Y plane
16.	SXZ or σ_{xz}	Shear stress in X-Z plane
17.	SY or σ_y	Normal stress in Y-direction
18.	SYZ or σ_{yz}	Shear stress in Y-Z plane
19.	SZ or σ_z	Normal stress in Z-direction
20.	UX	Displacement in X-direction
21.	UY	Displacement in Y-direction
22.	UZ	Displacement in Z-direction

SYNOPSIS

A surface hydro power station houses generating equipment having rotating parts mounted on a vertical shaft like rotor of electric generator and runner of hydro turbine, which transmit typical forces, like short circuit torque from generator and water pressure in scroll case around the runner, to large mass of concrete around the scroll case. In addition, the forces resulting from horizontal and vertical loads transferred from gantry columns of electric overhead traveling (EOT) crane, used for erection and maintenance of the generator and turbine, to the mass concrete around draft tube connected to bottom of the runner for conveying water to tail race.

The large mass of the concrete around the scroll case and the draft tube is considered basically a mass concrete structure acted upon by a number of horizontal and vertical forces. This concrete mass is generally heavily reinforced. For structural design the structure is usually divided into two parts (i) substructure, which houses the draft tube of hydro turbine (ii) intermediate structure, which encases scroll case and supports the generator.

The substructure and intermediate structure have complex shape and loading arising from water pressure in scroll case, short circuit torque in generator besides loads due to rotating parts of turbine and generator, as well as loads transferred from gantry columns of the super structure of the power station complicates the structural design.

Although a large number of power houses have been constructed both in India as well as in other countries, no exact analysis or standards for the design of Power House Structure are yet available. The design engineer considers each power house as an individual problem. Thus, due to its complex shape and force system acting upon it, the determination of stress distribution and displacements of different parts of the structure in the Power House structure is still an unsolved question, leaving much to the personal judgment of the design engineers

The prevalent design practices are based on the two dimensional analysis approaches. Each structural component of the power station is analyzed and designed in both transverse and longitudinal direction. However, this approach does not represent correctly the behavior of the structure under loads and forces but yields a safe design expeditiously.

For simulating the true structural behavior a three dimensional analysis and design approach is required.

This thesis presents stresses and displacements in concrete around the draft tube and spiral casing in surface power station on account of the various loading conditions by analysing it as a three-dimensional structure, using ANSYS package of Finite Element Method. The results reveal that the two-dimensional design approach gives a conservative design.

CHAPTER -1

INTRODUCTION

1.1 GENERAL

Energy is the backbone of overall economic development of a country. Energy consumption per capita is a recognised parameter for the economic development status of a country and in this respect India with a consumption of 500 kwh falls at the bottom of the list of countries. Out of several sources of energy, Electrical energy is most sought after because of the ease to carry it to long distances in no time. There are several conventional and non conventional sources of electrical energy but most common are thermal and hydro. Hydropower has inherent advantages of using annually replenishable sources free of cost and supplying peak energy whenever required in no time.

In last fifty years the installed capacity in India has increased from 1360 MW to about 1.1 lac MW out of which about 25000 MW is hydropower developed through run of river and storage projects. The hydropower potential of the country is estimated as 84000 MW at 60% load factor. Hence there is vast unharnessed hydropower potential in the country and the national policy is to harness it speedily and economically.

The initial cost of hydro development is high (Rs 4 to 7 crores per MW). Every type of hydro power scheme requires either an underground or a surface power house. The civil structure of a power house is generally costlier than the electrical equipment. The structure of a surface power house is complex in shape and has complicated loading pattern, not amenable to normal structural design practices. A reasonably accurate and speedy analysis of the structure is required to make the power house structure safe and economical. An attempt in this direction to carryout 3-D analysis using FEM.is made in this study.

1.2 EARLIER STUDIES

A 3-D analysis using FEM is carried out for generator barrel foundation of Power House structure using FEM (Vidyarthi Umashanker[14],2001). Three dimensional analysis of stresses in concrete around spiral case has been attempted by P.Kumar[8], (2004). A three dimensional frame analysis of super structure is carried out by Chand Puri[3], (1973). The stress conditions in the substructure have been studied by Khalid[7] in (1970). Efforts have also been made to investigate the behaviour of substructure and superstructure of the surface and under ground Power house by several researchers (Nigam[11], 1976; Ashim[2], 2002 ;).

1.3 PROBLEM IDENTIFICATION

It is seen from the review of literature over past three decades that the present knowledge about the actual structural behaviour of the power house is inadequate and prevalent two dimensional design practices are only approximate. Much attempt has not been made by the earlier researchers for the combined study of the substructure and intermediate structure together, which act monolithically.

In this context, endeavour has been focussed in the present work to investigate stresses and deflections in concrete around spiral casing and draft tube under various loading conditions by analysing it as three dimensional structure, using Finite Element Method. ANSYS software version 7.0 has been used to analyse the structure.

1.4 STUDY AREA

Koteshwar Power house has been taken as the case study. It is a surface power house located at the toe of Koteshwar dam across river Bhagirathi in the Tehri Garhwal, Uttaranchal state of India. The Dam is located at, 22 kms. downstream of the Tehri dam.

1.5 SCOPE OF THE STUDY AND OBJECTIVE

Based on the review of literature, the following objectives were set for the present study:

- Analysis of the substructure and intermediate structure to find out the stresses and deflections around the spiral casing and draft tube due to various loading conditions.
- Analysis of the superstructure for working out the support reactions at the top of intermediate structure.
- Analysis of the substructure and intermediate structure against two categories of loads i.e., one accruing from self weight, stator, rotor, short circuit torque and another from self weight, stator, rotor, short circuit torque, water pressure in the spiral casing and superstructure load.

1.6 FINDINGS OF THE STUDY

This present endeavour encompasses the behaviour of the structures with different rigidity of the foundation under two different loading conditions, resulting in significant findings, important observations as briefly narrated below.

1. The study has revealed that the structure of the power house is essentially a low stressed stable structure. There are some tensile stresses around the openings and the location of point load

2. It was seen that with different values of rock modulus of foundation, structure does not show appreciable change in tensile and compressive stress unless the rock modulus is less than Ec/10.
3. The stresses and deformations of the structure with foundation of modulus of elasticity, $E=10\ E_C$ resembled the results of the structure considered fixed at the base, indicating that the structure may be assumed fixed with foundation rock when rock modulus is about ten times that of concrete.
4. For economical foundation design the rock modulus of foundation should be about $E_C/10$ or more...

The study has revealed that no rigorous analysis of the power house structure is required. Nominal reinforcement is required in both directions around the openings such as spiral case and draft tube and the location of concentrated load application.

1.7 ORGANISATION OF THE DISSERTATION

This dissertation is organised in to the chapters as follows:

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

Chapter 2: Description of literature review.

Chapter 3: Presentation of details of structure and model with assumptions, dimensions and different loading conditions.

Chapter 4: Description of results and discussions.

Chapter 5: Conclusions and scope for future study.

References

LITERATURE REVIEW

2.1 GENERAL

The practice to analyse the structure of the Power house by different organizations of the world is generally based on the two dimensional approach which is based on various simplifying assumptions. To carry out simple 2-D analysis the structure is analysed along the transverse and the longitudinal direction. Various practices adopted by the different organisations of the world are briefly discussed in the chapter.

The actual structural behaviour of the power house under different loading conditions can be perceived only by the three dimensional analysis. The methods of 3-D analysis are also described in this chapter.

2.2 PRACTICES OF TWO DIMENSIONAL ANALYSIS

For two dimensional analysis the power house structure is divided vertically in to three parts viz, sub structure, intermediate structure and super structure. The sub structure and super structure are analysed both in transverse (along flow) and longitudinal (perpendicular to flow) directions .These are briefly described below.

2.3 SUBSTRUCTURE

Besides the hydraulic function, the substructure containing draft tube has structural functions as follows

(i) It safely supports the superimposed machinery loads over the cavities

(ii) It acts as a transition foundation member distributing the heavy machine loads on the soil such that the obtainable ground pressures are within safe limits.

2.3.1 Rock Foundation

In the case of rocky foundations, it does not require a large base area, since the bearing capacity of the rock is high. The piers and divide walls of the draft tube rest directly on the rock and structurally these act as portals. In order to provide smooth surface for the flow of water, a lining is provided over the rock. This thin slab is separated from the piers. In order to safeguard against uplift, proper weep holes and anchors are provided.

2.3.2 Soil Foundation

For soil foundations, a larger base area is required and the draft tube is generally in the form of boxed structure with thick bottom and top slabs.

From the above it is evident that design of substructure founded on rock as more easily amenable to analysis than that on soil. Attempts to economise on concrete and steel should be made by fully utilising the strength of foundation. The substructure on soil needs careful analysis. Its two dimensional analysis is briefly described below.

2.3.3 Two Dimensional Analysis

Shape of the draft tube structure is such that its structural analysis can not be carried out as a whole and it has to be resolved into simpler elements which might be amenable to a more accurate structural analysis. It is seen that this structure may further be divided into simpler sub divisions, which are supposed to have a similar structural behaviour (Figure 2.1). These subdivisions are

(i) Cantilever portion out side the power house

(ii) The elbow portion above the draft tube.

(iii) The bottom slab with in the power house.

(iv) The vertical members

(v) The solid mass up stream of the tube.

The analysis can be made simpler by splitting it in two parts:

- In transverse direction (along the direction of the flow)
- In longitudinal direction.

In the transverse direction superimposed load vary considerably from one point to another. Transverse analysis is meant to see that the sub structure acts as a true transition foundation member.

In the longitudinal direction the superimposed loads are assumed more or less uniform and in that direction no appreciable horizontal forces act (except the differential water pressure on account of one unit being closed or due to seismic forces in the longitudinal direction). Longitudinal analysis is thus meant to see that the structure performs well in supporting the superimposed loads of machine.

2.3.4 Transverse Analysis

In this direction sub structure can be analysed in a number of ways in order that it my act as a foundation transition member:

(i) As a flat plate or a raft

(ii) As a continuous footing

(iii) As a cantilever projecting from a mass structure.

(iv) As a gravity structure

These are briefly described below.

2.3.4.1 Flat plate or raft

It could be seen that the superimposed loads introduced by the plant equipment etc do not act below the level of the top of the sub structure (Figures 2.1and 2.2). This level may therefore be considered as the top of the raft foundation through which the loads are transmitted.

The only deference between an ordinary raft foundation and this structure is that, while the former is solid, the latter has openings of different kinds. In a raft foundation provided in the normal buildings the point of application of the downward loads are well defined.

However, in the case of the substructure of the Power house the point of application of the various loads and their intensities at the top of the substructure are not well defined and cavities galleries complicate the problem still further (Figure-2.1). Thus it becomes difficult to find out the structural behavior of the sub structure and the distribution of forces in the transverse and longitudinal direction. Since the loads are assumed to vary only in the transverse direction and remain more or less uniform in the longitudinal direction, this analysis is possible and serves the purpose. It is simpler and takes into account one direction at a time.

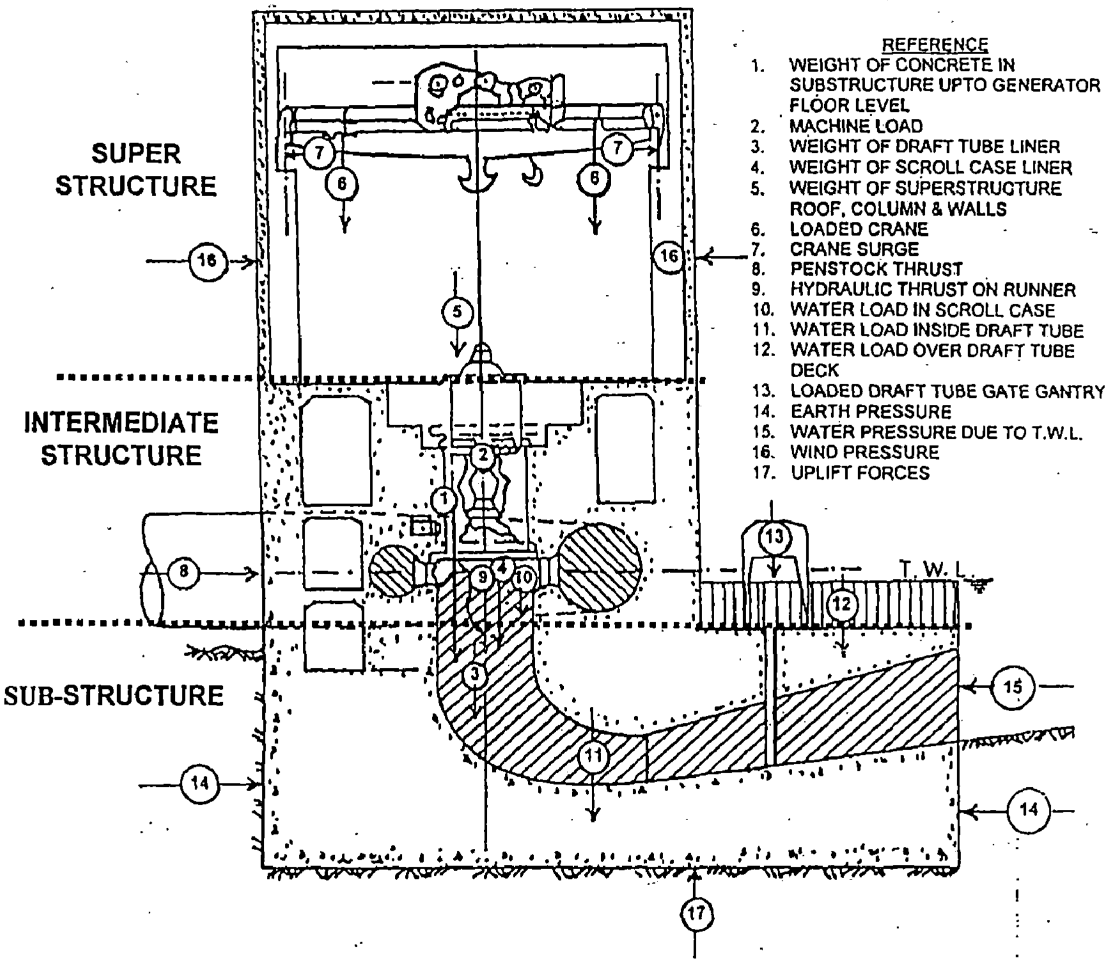
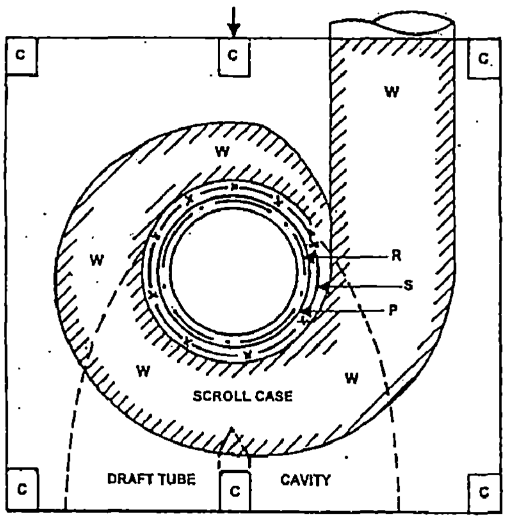
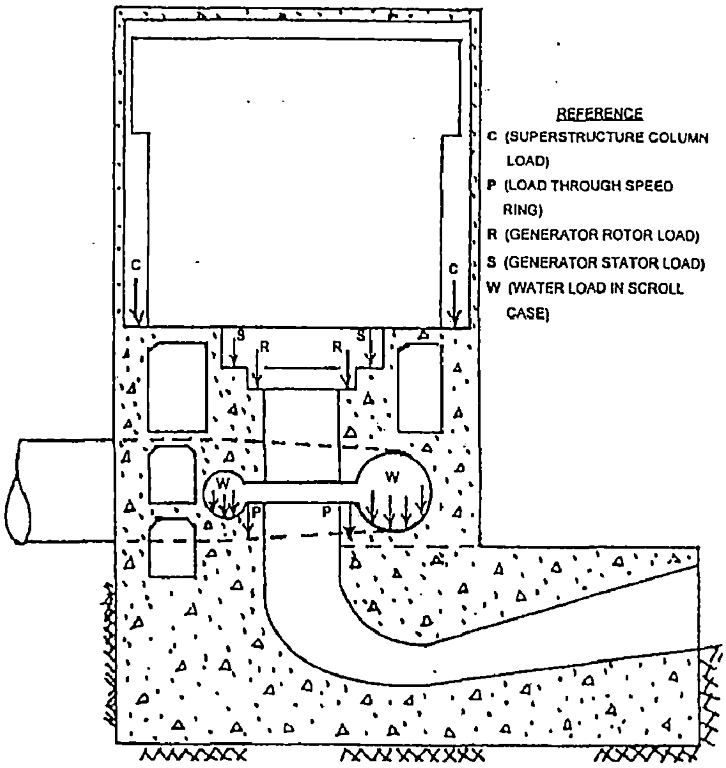


Figure 2.1 Forces acting on a power house



(a) Internal plan of machine hall showing relative position of loads



b) Position of Loads at various levels and concrete in a power house

(Equipment and plant not shown)

Figure 2.2 Location of different loads is a power house

2.3.4.2 Continuous footing

The substructure may be designed in the transverse direction as a continuous footing which is supposed to be an end less cantilever. The moment at any section of the footing is the algebraic sum of moments of forces about that particular section. The salient features of the various steps involved in the complete design of the sub structure as footing are as below:

a) Computation of loads at top of the substructure

b) Determination of soil reaction

c) Finding bending moments and shearing forces in the transverse direction

d) Actual design of section and determination of reinforcement

2.3.4.3 Cantilever projecting from mass concrete

Some organisations opine that inside the power house walls the structure may be considered as mass concrete on account of its great depth and may provide sufficient fixity to the draft tube portion projecting out the downstream wall, which may, thus, be treated as a cantilever for which reinforcement may be provided in the projecting portion and continued upon the draft tube opening inside the power house.

Such a treatment does not fulfil the condition of substructure as a fully transition member because it does not consider the interaction of the loads inside the power house walls in relation to the projecting cantilevers.

2.3.4.4 As a gravity structure

The power house sub structure remains essentially a mass concrete structure with a very high degree of structural strength and integrity. According to one organisation the correct design of the entire structure can be accomplished by an analysis similar to that used for gravity dam.

The sub structure may be divided into a number of hypothetical blocks by conveniently chosen horizontal planes. Evaluation of active forces should include

a) All vertical forces (dead and live)

b) All horizontal forces (Soil pressure, hydrostatic, hydro dynamic, and earthquake and wind etc.).

c) Generator and other plant equipment.

Just as in a gravity dam, normal and principal stress in concrete can be computed by analytical methods after determination of the resultant of all forces and computation of sectional properties for the unit block at various elevations. This analysis is very difficult and time consuming.

On account of the difficult analysis a modified gravity analysis combining the ideas of continuous footing and gravity structure as outlined below may be made.

Considering any gravity dam as a continuous footing acted upon by horizontal forces and its own dead load, it will be found that at any section a considerable bending moment and shear force occur which would require sizable amount of steel as reinforcement in order that no cracks develop. However in a gravity structure, the horizontal shear stress also has the considerable value and it is this horizontal shear stress which prevents a vertical section of the dam from separating with the adjacent section. On the same analogy, it would not be wrong to take into account the horizontal stress in the power house sub structure also.

From the above facts it is evident that the provision of steel in the transverse direction will considerably reduce if horizontal shear is taken in to account.

2.3.5 Longitudinal Analysis

The analysis in the direction (at right angle to the flow of water) is required in order to check and provide for structural strength of sub structure, so that it may support the equipment and other superimposed loads in spite of the various cavities. This analysis can be understood if this structure is further subdivided into the following smaller subdivisions such that they can be treated as simpler elements.

- Cantilever portion out side the power house wall
- Draft tube portion inside the power house

It can be treated as a frame/box for the out side portion but the thickness of frame members being large it is appropriate to consider the elements of the box individually for the portion inside the power house the elements are as below

a) Elbow portion above the draft tube

b) Bottom slab with in the power house.

c) Vertical members

d) Solid mass upstream of draft tube

The details of analysis of the above parts are discussed below.

2.3.5.1 Cantilever portion out side the power house wall

In longitudinal direction this portion can be analysed as a multi bay boxed or framed structure. If foundation is laid on the soil it is taken as boxed structure, if foundation is on the rock then this structure is treated as framed structure. If the members are considerably thick these are considered or analysis as shown in the Figure 2.3

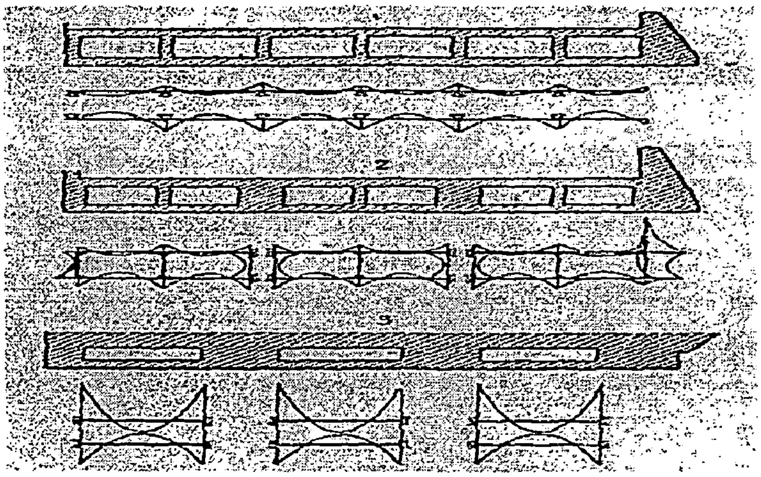


Figure 2.3 Longitudinal sections at draft tube

It is possible that the minimum reinforcement provided on account of the temperature and shrinkage stress would be in considerably large quantities to warrant any other consideration. Obviously in such situation it would be of no use to make any more accurate calculations on account of the superimposed loads. Any reinforcement provision based on these would give a false impression of the safety of the structure. Hence it is always good practice to reduce the parasite (shrinkage and creep) stresses as much as possible that is by reducing the thickness of members, by using construction joints and by employing appropriate construction materials

N.K.A.G. Consulting Engineers, Switzerland, recommended that this projected portion should be separated from the main machine hall by means of a joint for the stress analysis. The section could be considered as a closed frame work (Box frame).In case the depth of foundation slab is large as compared to the thickness of vertical members then it may be treated as continuous beam as suggested by Mosonyi, which has been depicted in Figure 2.3. U.S.B.R. recommends that the analysis be done treating the structure as a continuous frame. Since the thickness of members are considerably higher than those are ordinarily met with in commonly used frames, special care has to be taken for computing stiffness and carryover factors and the bending moments can be found in the usual manner applying the column analogy method. U.S.ARMY also recommends that this portion may be analysed as a multibay framed structure. The moments as found from the centre line diagrams will be more than the actual values if the members of the frame are more than 300 mm thick and in these cases the moment should be reduced to moments at the face of the support by allowing for shear relief as recommended by the Portland Cement Association.

2.3.5.2 Draft tube portion inside the power house

a) Elbow portion above the draft tube

Unlike the first sub division, this portion has to be necessarily thick in order to give the required shape to the draft tube. On account of the complex shape of the structure exact analysis is not possible.

It is interesting to note that while for cantilever portion there is almost consensus of opinion among different organisations about its treatment yet for this elbow portion there are quite divergent opinions regarding its design as given below.

(a) Mosonyi stated that this roof slab can be treated as slab fixed at both the ends with outer walls of draft tube .Loads on this portion are high on account of it lying below the spiral case and machine support. But the structural height of the roof is also considerably greater. On account of its great depth this slab can be treated as a deep girder.

(b) In the Miller Ferry project constructed by the U.S. Army it has been assumed that the superimposed loads are transferred by means of parabolic arches within mass concrete, which span over the opening. The series of arches can be suitably reinforced. The vertical and horizontal component of arch action should also be taken into account. The mass concrete lying below the hypothetical arches can be treated as supported on this arch.

(c) Another variation of this idea can be of treating this portion as lintel spanning between the opening and on which the load of concrete lying within 45° isosceles triangles need only be taken.

(d) An Italian organisation of repute suggests that for this portion as the floor and sides the hypothesis of a massive block with tunnel shaped cavities in it(consisting of the turbine draft tubes) is to be considered more realistic than that of too short and high beams..

(e) Central Water and Power Commission (CWPC) recommended the following method for analysis of this portion. The portion is divided into two zones as shown in Figure 2.4. Zone -2 is directly below the gantry columns and is designed as a deep girder for the superimposed and the dead load. Zone-I is divided into three strips, strip *A* is just down stream of the throat ring of the draft tube liner and is supposed to carry the 1/4th generator load, 1/3rd the weight of embedded liner etc. and the dead load of the concrete below the turbine floor. This is supported on strip B on either side of the throat of the draft tube. Strip *B* is supposed to carry the live load on the floor and concrete dead load besides the reaction of strip.

Strip *C*, which carries the reaction of strip *B* and the concrete dead load, has been analysed by consistent deflection as below.

(a)- Load transmitted by the deep beam is such that the deflection of the beam is equal to the pier shortening.

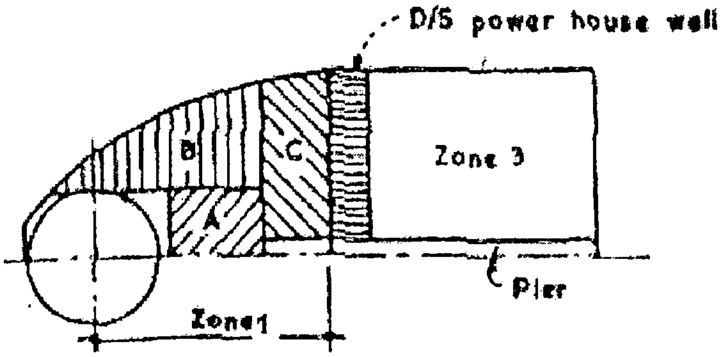


Figure 2.4 Sub division of zone 1 into strips for structural support

(b) The support of the strip *C* is provided partly as a cantilever beam from zone II and partly as beam supported on pier, it being supported that 75% is the cantilever action and 25% beam action.

b) *Bottom slab within the power house*

The method of design of this portion of the substructure is more or less interconnected with design of the elbow portion and these are described here..

(a) Masonyi stated that foundation slab may be treated as slab fixed at both ends in the separation or outer walls (divide wall). If the splitter (intermediate) pier is short, this slab may be treated as a two way reinforced slab. In the longitudinal direction it may be treated as fixed in the wide separating walls (divide walls). In the transverse direction it may be treated as fixed with the mass concrete at the up stream on one side and the adjacent slab strips of short span on the other.

(b) Miller Ferry Project constructed by U.S. Army this slab has been designed as spanning between a heavy cantilever members (projecting out of the mass concrete) near center line and theoretical beam near the upstream end of the intermediate piers. The theoretical beam is of the slab depth and transmits the reaction of slab and beam to the piers and the side walls of the draft tube.

c) Vertical members

So, far as the vertical members of the outside cantilever portion are concerned they can be designed as veridical members of the frame work. Inside the machine hall the projection of the intermediate pier/ piers is very small and the wide side walls are the only vertical members. Their method of analysis should obviously depend upon the type of analysis adopted for the elbow portion and the bottom slab.

On account of the massiveness of these members, it appears justifiable to provide only temperature reinforcement at the face of the draft tube. However they may also be tested structurally for the vertical loads and bending moments. On account of lateral loads, treating them to be fixed at top and bottom is necessary.

d) Solid mass upstream of draft tube

In the longitudinal direction there is no design problem for the solid mass of concrete upstream of the draft tube. However the provision of reinforcement from the consideration of construction constraints should be made.

2.4 INTERMEDIATE STRUCTURE

The intermediate structure of a powerhouse is a part, which extends from the top of the draft tube to the top of the generator foundation. It contains two important elements of the powerhouse. One is the generator foundation, galleries and other is the scroll case. Intermediate structure has mainly two functions, firstly it is to safely support the barrel and superstructure loads and transmits the same to the substructure and secondly to provide support to concrete encased the spiral case.

The scroll or spiral case is that part of turbine which distributes water from the penstock uniformly and smoothly through the guide vanes to the runner. The spiral case is required only in case of Francis and Kaplan types of turbine. The design of the spiral case is somewhat complicated from both hydraulic and structural considerations, due its hydraulic function and irregular shape.

2.4.1 Types of Spiral Case

There are two types of spiral case. One is concrete spiral case, which is used in low head power station having a shape of rectangular or trapezoidal in section with an angle of envelopment varying from 200^0 to 250^0. Another is steel spiral case, which is generally used in medium and high head power station having a shape circular in section with an angle of envelopment varying from 300^0 to 360^0 (.Figure-2.5).

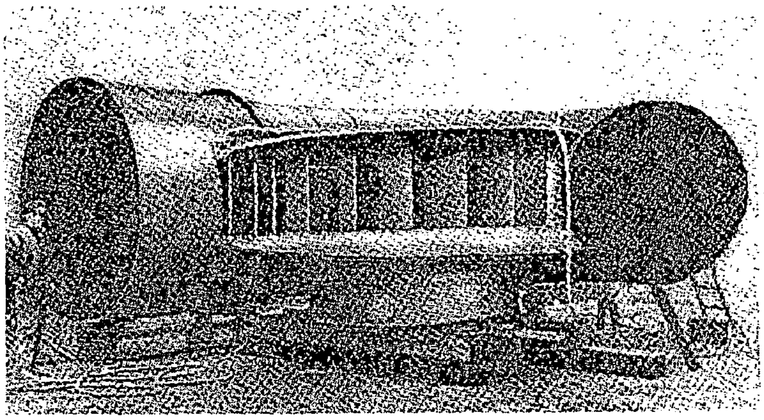


Figure 2.5 Steel spiral casing

2.4.2 Loads and Forces on the Intermediate Structure

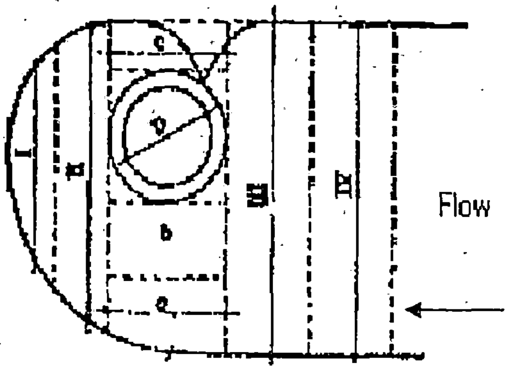
A section of power house and forces acting on it is shown in Figures 2.1 and 2.2. The arrows indicate the centre of gravity of the various forces and loads and may not give the exact point of application of these loads. The figure itself indicates that neither the geometry of intermediate structure nor the loads acting over it are symmetrical.

Generator support loads are the major loads. Floor load, dead load, Tangential force due to short circuit and radial loads due to unequal magnetic attraction are others loads acting on generator foundation are transmitted on the intermediate structure. Beside it, hydrostatic force including water hammer effect is also a major part of load acting on the inner face of spiral case.

2.4.3 Concrete Spiral Case

The load on the spiral case roof may either be carried by the roof itself or transmitted (at least partially) to the substructure by the stay vanes.

The roof of the spiral case shall carry the machine load and the structural design shall be carried out in two ways.



I, II, III and IV are primary beams and
a, b and c are secondary beams

Figure 2.6 Arrangement of primary and secondary beams

a) The roof of the spiral case is considered as a reinforced concrete slab with a large opening for the turbine, which rests upon or is fixed into the side walls whose structural system is composed of a system of beams and cross beams as shown in the Figure 2.6.

b) Beams a, b, and c are secondary beams which are partially fixed to the primary beams II and III. In the case of relatively wide spiral case walls, the primary beams I, II, III, and IV may be designed as fully fixed. Sometimes it may be advantageous to adopt a radial arrangement of beams as shown in Figure 2.7.

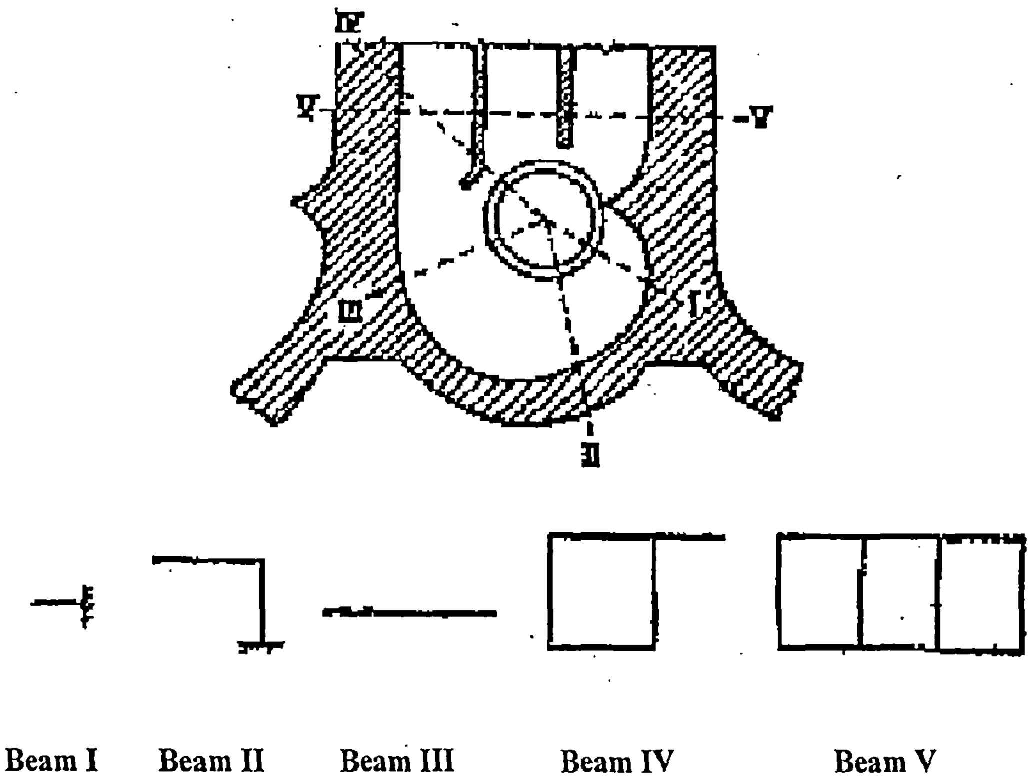


Figure 2.7 Arrangement of beams in concrete spiral casing

The downstream side of the spiral case is assumed to be a cylinder and therefore, circular reinforcement fixed into the upstream section is provided. If the spiral case is not too high, the sidewalls may be designed as fully or partially fixed vertical walls, depending upon their relative stiffness. The load may be divided into two parts, one including tensile stresses in the circular direction, i.e. in the horizontal elements, and the remaining partial load including bending moment in the vertical direction.

ii) <u>Load transmitted by stay vanes</u>

The steel stay vanes are fixed at their lower and upper ends with the steel speed rings. This stay vane should be well embedded into the concrete in order to ensure reliable load transmission and the structural arrangement should be such as to prevent the crushing of the relatively thin spiral case cone by the heavy loads transmitted through the stay vanes. These elements carry not only downward loads but also the resultant of the pressure tending to lift the roof of the deeply set spiral case as anchorage also.

2.4.4 Steel Spiral Casing

Generally steel spiral casing is fully encased by concrete; some times, they are half encased and occasionally un-encased also. The problem of designing the concrete encasing the steel spiral case is complicated due to the difficulty in predicting its structural behaviour on account of the irregular shape as well as because of the expansion of the steel liner due to the internal water pressure.

The forces acting on the concrete section depend upon the mode of placing concrete in the case of fully encased spiral casing and the section of scroll case which varies from place to place. As the boundary of concrete around the spiral case is rectangular in shape, the width of the wall surrounding the spiral case varies from one section to the next. On account of these varying factors the scroll case is divided into a number of sections as shown in the Figure-2.8 for purpose of design.

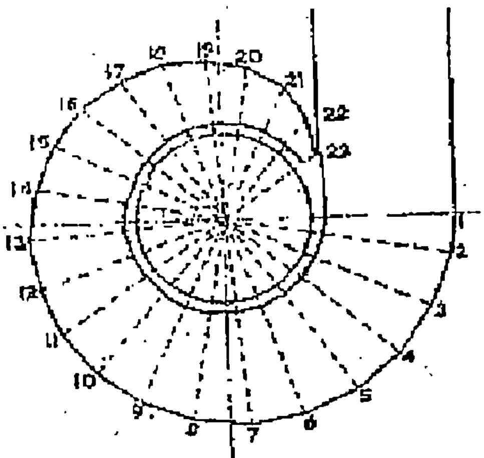
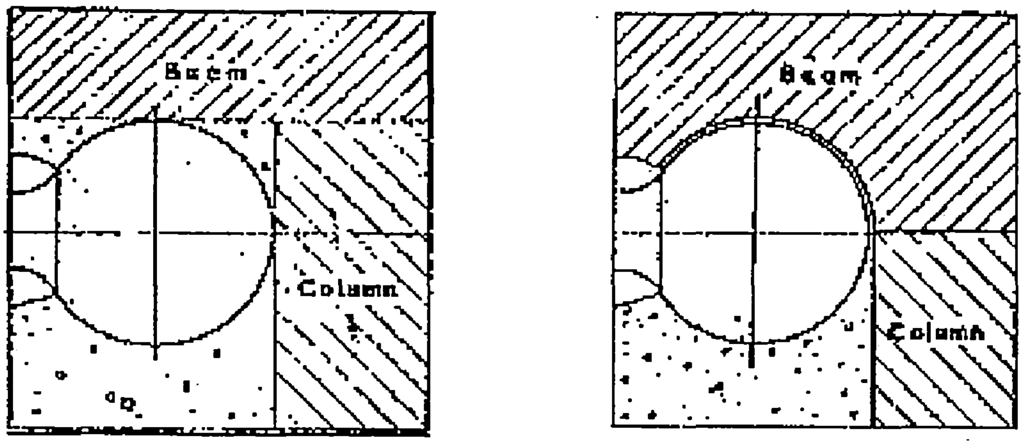


Figure 2.8 Plan of steel spiral case

Since the exact behaviour of concrete is unpredictable these section may be designed in one of the following ways:

2.4.4.1 Beam and column arrangement

In this arrangement the top of the concrete is considered as a beam. After deciding the level of generator floor and hence the level of the top of the beam, there are two alternatives for considering the depth of beam.



(a) Beam with fixed depth (b) Beam with variable depth

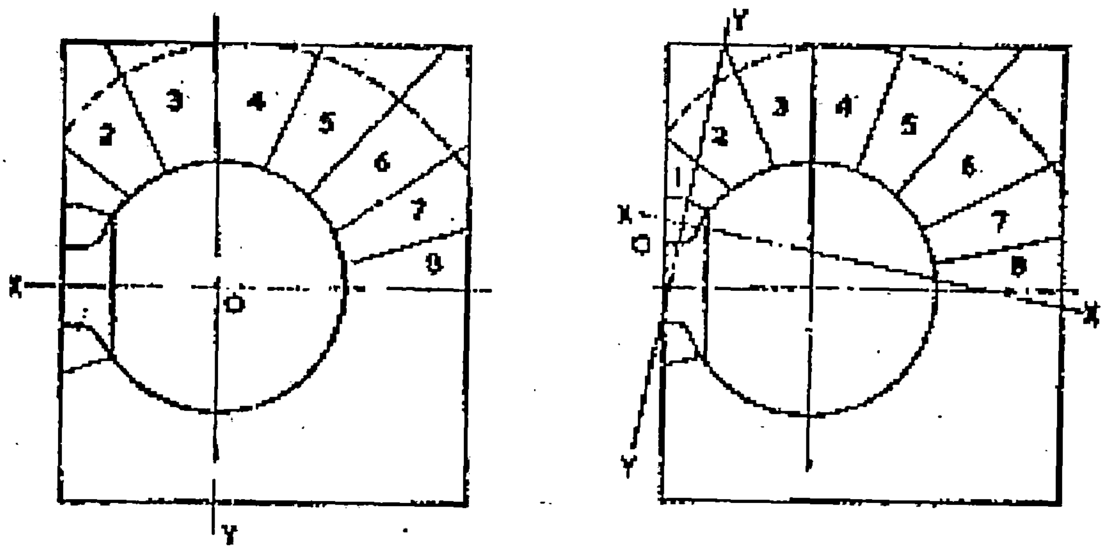
Figure 2.9 Beam and column arrangement

- The depth of the beam is equal to the minimum depth at the crown as shown in the Figure 2.9 (a). The concrete at the side is considered as a column having the width available at the centre line of the runner.
- As shown in the above Figure 2.9 (b) the beam is of variable depth and follows the profile of the scroll case in section. Since the concrete will be laid in such a manner as to fill the entire space above the spiral liner, it will be economical to take into account the full area of concrete as well.

The end conditions at the columns support will be altered since the depth of the beam and the width of the column is varying at every section. So if the width of the column is comparable to the depth of the beam, the end will be considered as fixed; otherwise it is taken as hinged and the top slab may be treated as fixed at the outer end & hinged at the inner end (speed ring). Since there is an element of doubt in the support condition on the speed ring, it may be taken as hinged.

2.4.4.2 Arch arrangement

In this arrangement the top concrete is treated as an arch. The inner boundary of the arch is circular while the outer boundary of the arch may also be drawn circular with a different centre so as to keep the boundary within the concrete to be poured monolithically as shown in the Figure 2.10.



(a) Fixed arch (b) Hinged arch

Figure 2.10 arch arrangements at speed ring

The outer end of the arch is taken as fixed at the center of runner, the fixity being provided by the concrete below the scroll case. Since the degree of fixity provided at the speed ring is not very definite, this end may be treated either as fixed or hinged. The analysis is carried out with both the end conditions and to design the section the maximum moments obtained by the two methods should be taken.

2.4.4.3 Ring arrangement

In this arrangement the section is considered as a ring between two hinged speed ring supports.

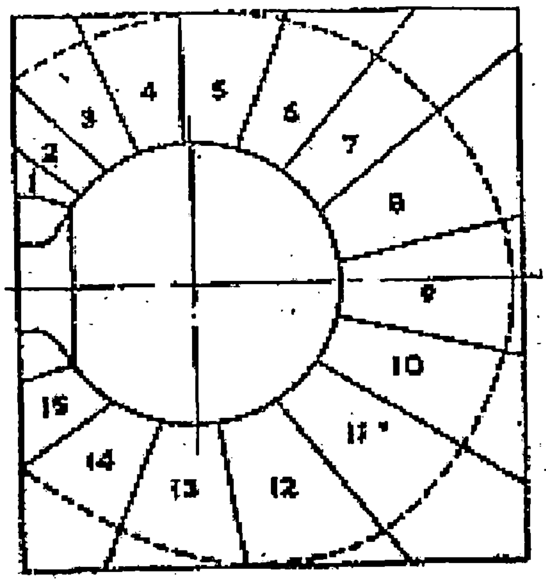


Figure 2.11 Ring arrangement

As shown in the Figure 2.11, an arch is inscribed with a different radius within the monolithically laid concrete. The ring is divided into a convenient number of sections. This method has more importance when some horizontal thrust, say, on account of high tail water is to be resisted by the encasing concrete.

2.4.4.4 Hollow cylinder

In this type of arrangement, each section of the encasing concrete may be treated as part of hollow cylinder which is subjected to uniform pressure on the inner (for internal hydrostatic plus water hammer forces) and outer surfaces. In this method the distribution of external forces on account of superimposed loads of machinery will have to be converted into equivalent uniform pressure.

2.4.4.5 Mass concrete

Many of the design organizations do not make any special analysis for this part and provide only nominal reinforcement around the openings for shrinkage and creep. It is supposed that this concrete is space filler and provides foundation for generator. It would behave like a block in which some openings have been scooped out, and the superimposed loads are transmitted to speed ring or concrete encasing the scroll case.

2.5 SUPER STRUCTURE

Super structure of the power house generally consists of beams columns, walls, roof slabs etc, which acts as a framed structure, having three-dimensional behaviour. Earlier the transverse frame used to be analysed as a portal and u/s and d/s longitudinal frames as multi bay-multi storey frame. The column sections are designed for biaxial bending. Now the structural analysis as a space frame is possible using standard computer packages.

2.6 THREE DIMENSIONAL ANALYSIS TECHNIUES

Since behaviour of the power house structure is three-dimensional, a three dimensional stress analysis by photo-elastic method can be employed to identify the areas of critical stresses

With the availability of high speed and large memory computers, it is possible to carry out the rigorous analysis of the structure. Some of the available numerical method such as Finite Difference Method, Finite Element Method etc. render it possible for the whole unit to be considered as the three dimensional structure. These can be regarded as an accurate analytical procedure. Now a days various softwares for three dimensional FEM analysis are available. ANSYS is one of them to analyse the structure as three dimensional problem.

2.6.1 Photo Elasticity Technique

Stress freezing method is employed in 3-D model photo elasticity technique to find out the stresses in the model. It is based on the diphase behavior of polymer materials when heated. The polymeric materials are composed of long chain hydrocarbon molecules some of which are well bonded into a 3-D network of primary bonds. However a large number of molecules are less solidly bonded together into shorter secondary chains. At room temperature both primary and secondary molecular bonds act to resist deformation due to applied load. However as the temperature of the polymer is increased the secondary bonds breakdown and the primary bonds in effect carry the entire applied load. Since the secondary bonds constitute a very large portion of the polymer, the deflections which the primary bonds undergo are quite large yet elastic in character.

If the temperature of the polymer is lowered to room temperature while the load is maintained on the model, the secondary bonds will be reformed between the highly elongated primary bonds and serve to lock them into their extended positions.

When the load is removed the primary bonds relax to a very modest degree but the main portion of their deformation is not recovered. The elastic deformation of the primary bonds is permanently locked into the model by the reformed secondary bonds.

As these deformations are locked-in on a molecular scale, the accompanying birefringence is maintained in any small section cut from the original model. The cutting or slicing process may relieve the molecular layer on each face of slice cut from a model but this relieved layer is so thin relative to the thickness of the slice that the effect is not observed.

Different steps for the stress freezing are as follows:

(i) Place the model into the stress-freezing oven.

(ii) Heat the model relatively rapidly until the critical temperature is attained.

(iii) Apply the required loads.

(iv) Soak the model for at least two to four hours until a uniform temperature throughout the model is obtained.

(v) Cool the model sufficiently slowly that temperature gradients are minimized at the rate 1^0 C per hour.

(vi) Remove the load and slice the model.

Basic Photo-elastic Equation

The basic photo elastic equation is

$$\sigma_1 - \sigma_2 = \frac{Nf_\sigma}{h}$$

Where, h = thickness of the slice in inch.

σ_1, σ_2 = principal stresses along two principal axes.

$$N = \frac{\Delta}{2\pi}$$

Where, N = relative retardation in terms of complete cycle of retardation, 2π

Δ = Relative retardation,

$f_\sigma = \frac{\lambda}{c}$ = the material stress fringe value in psi-in,

Where, λ = wave length,

$c = c_1 - c_2$ = Relative stress optic coefficient,

Where c_1 and c_2 are stress optic coefficients along the two principal axes.

According to Maxwell:

"The changes in the indices of refraction are linearly proportional to the stresses induced in the model" i.e.

$$n_1 - n_0 = c_1 \sigma_1 + c_2 \sigma_2$$

$$n_2 - n_0 = c_1 \sigma_2 + c_2 \sigma_1$$

Where n_0 = index of refraction of the model in the unstressed state, n_1 and n_2 are indices of refraction along the two principal axes associated with σ_1 and σ_2 respectively.

The component stresses computed from the above experimental method can be presented in the form of stress contours at different planes. Analysis of spiral casing (intermediate structure) of a powerhouse using photo-elastic technique has been carried out by Nigam (1985) for machine load and stress contours are given in his book "Hand book of Hydro Electric Engineering".

2.6.2 Three Dimensional Finite Element Method

The structure of powerhouse is essentially a three dimensional structure with complex shaped cavities and loading conditions are also varying in all directions and levels. So the assumptions of two-dimensional analysis for the structure will not give the exact picture of nature of stresses developed at different locations. A three dimensional analysis will give a picture of sufficiently reliable and correct stress distribution pattern. In three dimensional models, number of elements can be increased to get better accuracy. But for large number of elements, more computer capacity and computation time is required so there should be a compromise between accuracy and computational time and computer capacity. Several three dimensional of various structural components of a power house have been carried in the past (Vdyarthi Umashanker, 2001; kumar, 2004; Nigam, 1976; Ashim, 2002 ;).

2.7 CONCLUDING REMARKS

From the review of the literature, it is seen that no study has been carried out towards the analysis of stresses around the spiral casing and draft tube, considering the substructure and intermediate structure as a monolithic concrete structure supporting the super structure

In the present study, a 3-D model of Koteshwar power house, substructure and intermediate structure together, supporting the superstructure frame, have been developed and analysed with the help of ANSYS software to understand behaviour of the structure and the results worked out have been presented.

STRUCTURE AND MODEL

3.1 GENERAL

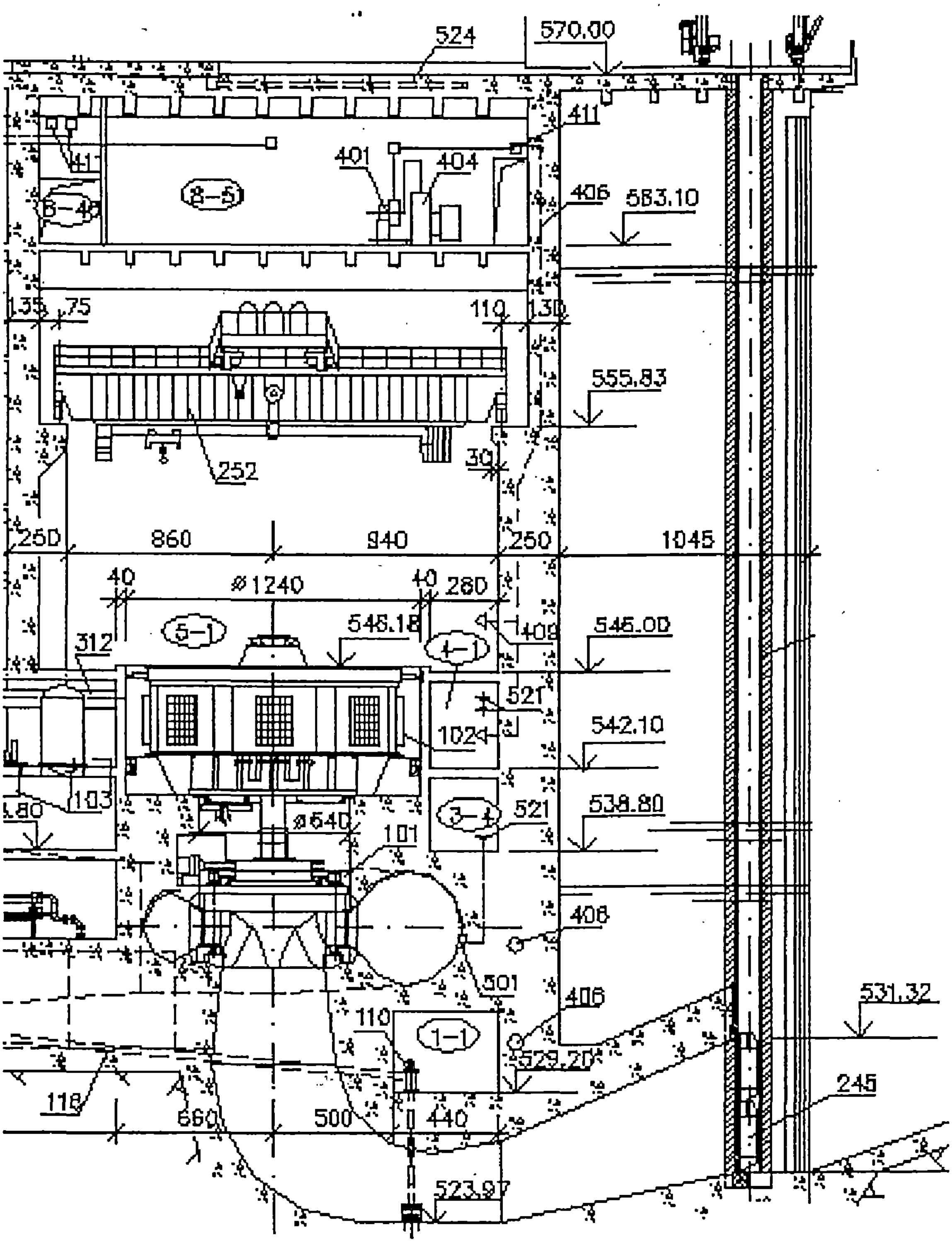
The Koteshwar hydroelectric power house of has been selected as a case study of this dissertation. This power house at the toe of the dam utilises the head for the generation of hydroelectric power of 400 MW (4 units of 100 MW each).

A three dimensional model of one unit of the Koteshwar power house has been prepared to determine the deflections and stresses. Finite Element Method (FEM) using ANSYS software is used to analyse the model under different loading and foundation conditions to get an insight to the structural behaviour of the concrete surrounding the spiral casing and the draft tube.

3.2 DIMENSIONS OF THE PROTOTYPE AND THE MODEL

The transverse and the longitudinal sections of the power house are shown in the Figures 3.1.and 3.2, The Units of the power house are 18.0 m c/c when measured from the turbine axis and draft tube of each unit is having one pier of 2.0 m thickness. The side walls of the draft tube are having a thickness of 2.0 m, providing two openings of 6.0 m each. The load from the super structure is considered at the elevation of 542.46 m .Which is the level of stator support. Figure 3.3 shows the dimensions of the spiral casing. In the prototype spiral casing diameter reduces to zero at the angle of 360^0 but in the model, spiral casing has been modelled up to 330 ^{0} to avoid the degeneracy during the meshing of the model. Radial and circular dimensions of spiral casing and dimensions of the draft tube have been shown in Tables 3.1 and 3.2.

The superstructure is having two walls one in u/s and the other in d/s direction running along the longitudinal direction of the structure. There are three columns, placed at the c/c distance of 9.00 m connected to the u/s wall having the height of 13.37 m and section of 1.15 m x 1.15 m. At the elevation of 555.84 m a longitudinal beam to give track to the gantry beam is provided having a length of 18.00 m. A projection of 1.10 m is in the d/s wall for the support of the gantry beam. Structure is having two slabs of the dimension 23.00 mx18.00 m at the elevation of 563.10m and 570.00 m .Each slab is supported by the three transverse beams spacing at 9.00 m having cross section of 1.15 m x 1.15 m and length of 23.00 m. There are ten longitudinal beams also to support each slab but to avoid the complexity in the modelling the average thickness of the slab is calculated considering the longitudinal beams as the part of the slab. The average thickness of each slab has been taken as 0.35 m. The model of the super structure has been shown in the Figure 3.9.



3.1(a) Transverse section of the Power House

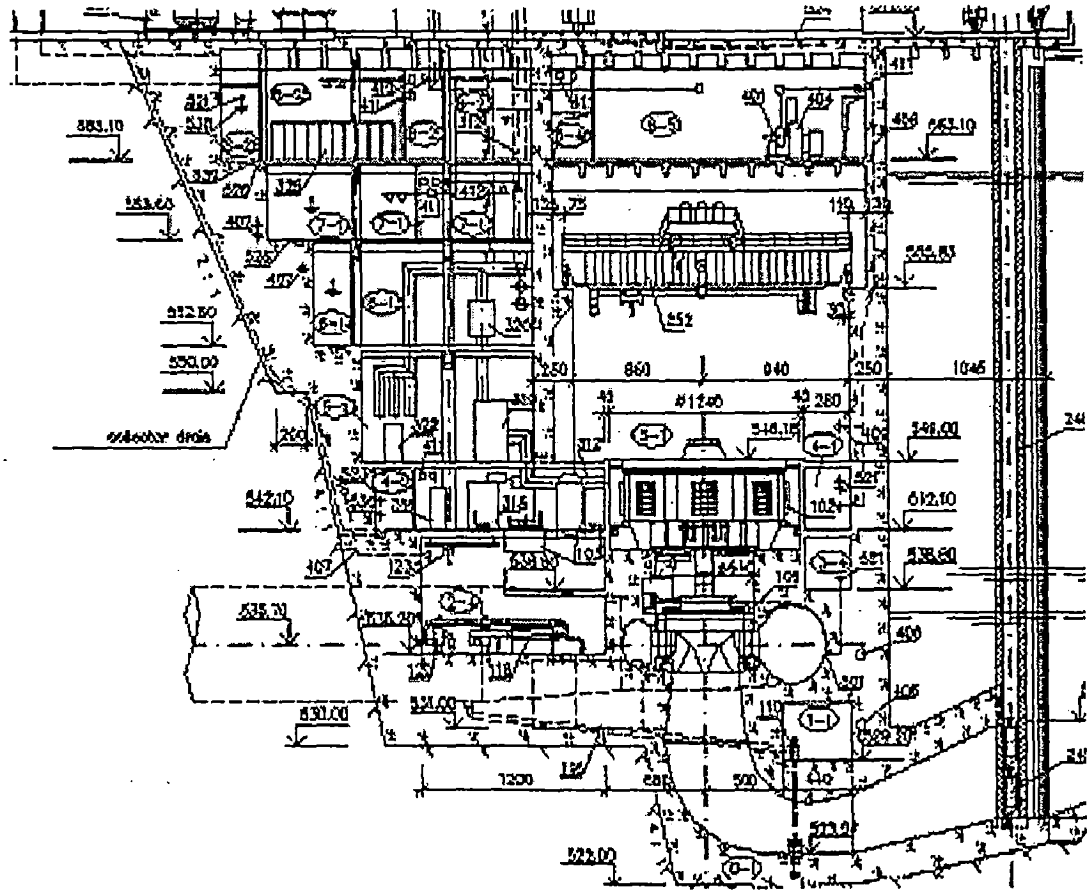


Figure 3.1 (b) Transverse section of a unit bay of the Power House

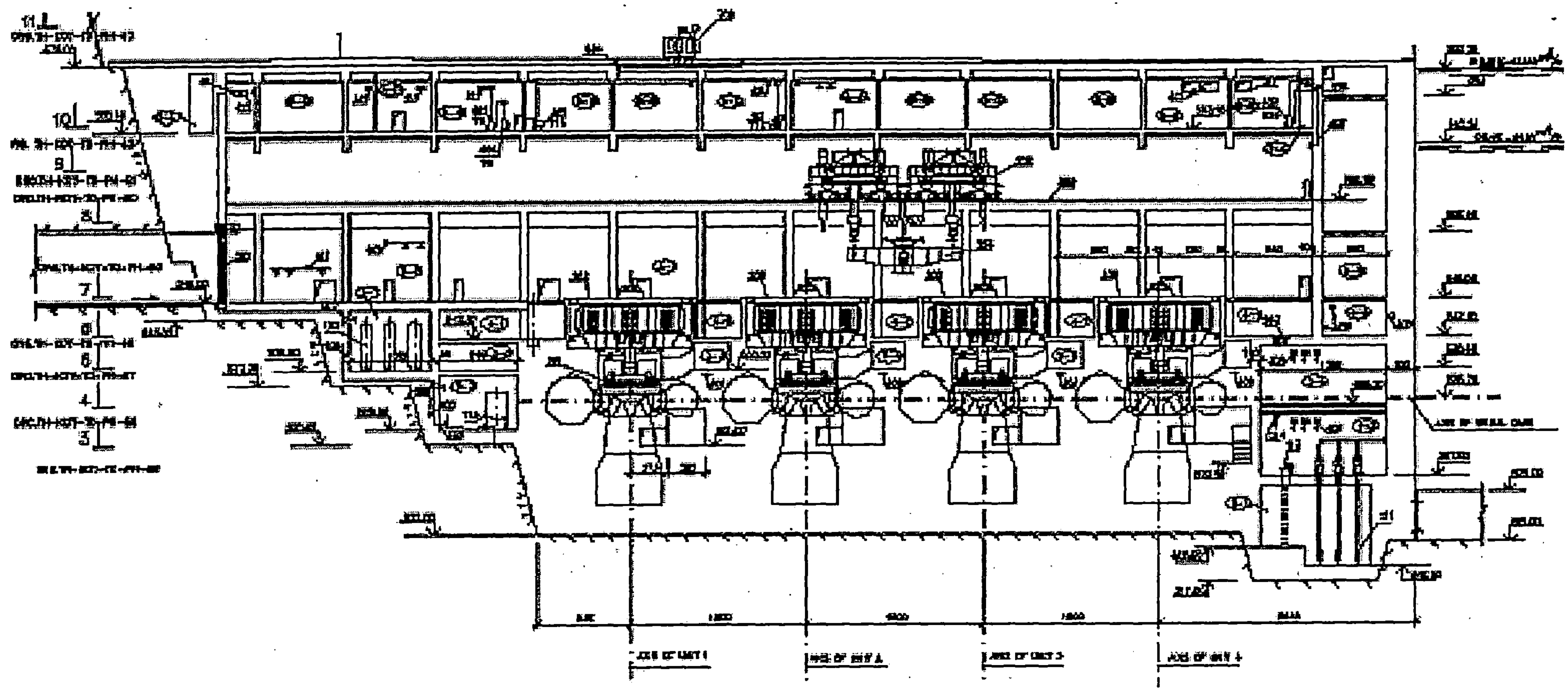


Figure 3.2 (a) Longitudinal section of the Power House

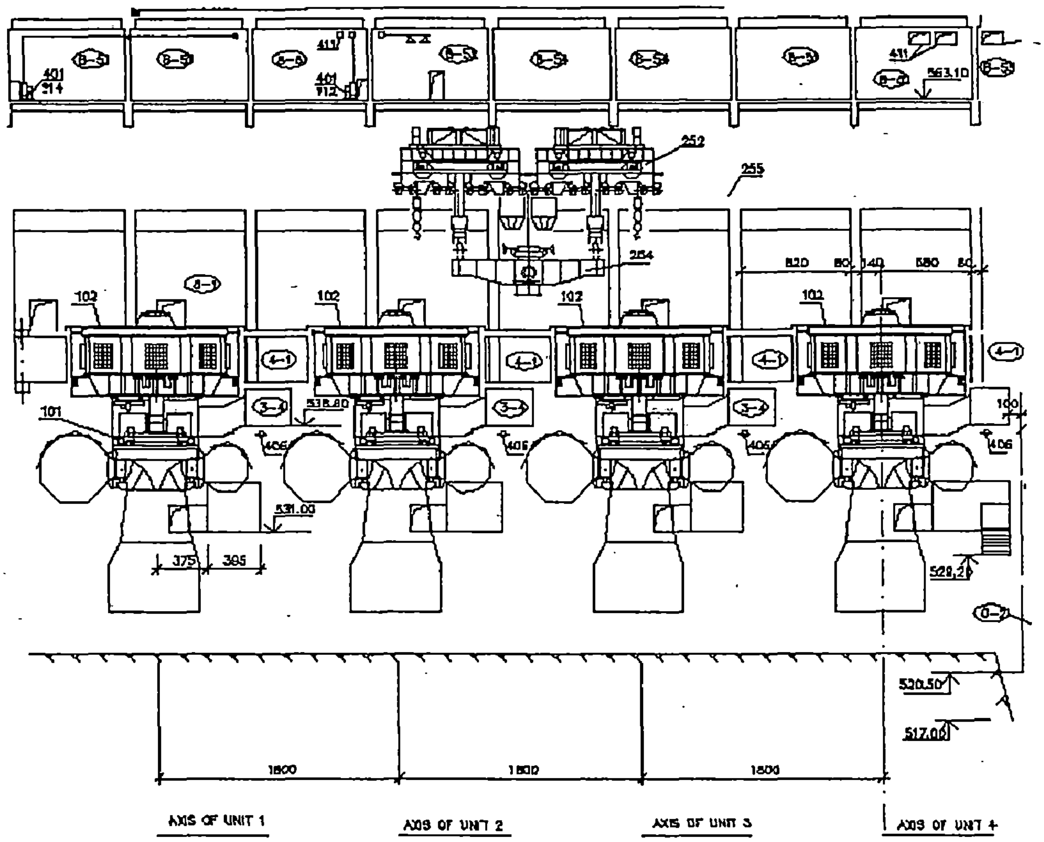


Figure 3.2 (b) Longitudinal section of the unit bays

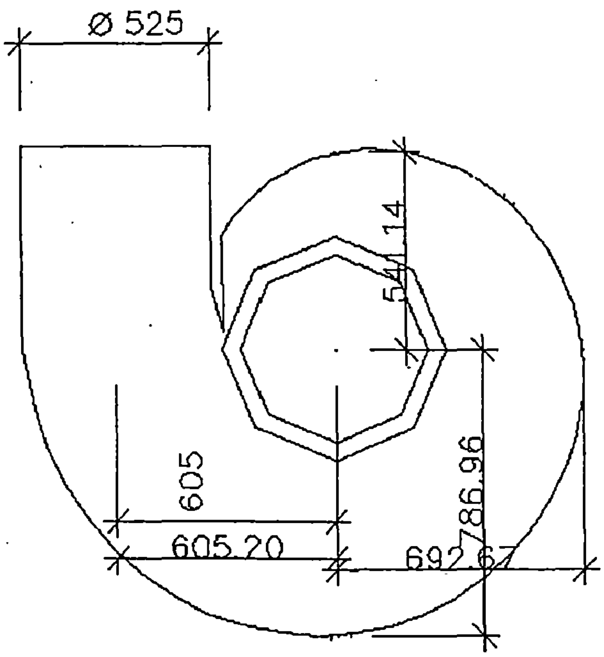


Figure 3.3 Dimensions of the spiral

3.3 ASSUMPTIONS IN THE MODEL

The following simplified assumptions and concrete properties are used for analysis are as follows:

a) The material is within its elastic limit.

b) Concrete of the structure is homogeneous.

c) The material is isotropic.

d) Model does not provide for any gallery or cavity in encasing concrete.

e) Loads of stator foundations and lower brackets are distributed on six brackets of generator barrel

f) Young's Modulus of Elasticity of concrete (M20), E=2.05 x 10kg/sq.cm.

g) Poisson's Ratio of concrete, $\nu = 0.2$

3.4 SUB STRUCTURE AND INTERMEDIATE STRUCTURE

The dimensions of different components of the model are as below and these are shown in Figures 3.4 to 3.9

a. The maximum diameter of scroll case is 5.25 m at the inlet. The radius of other sections are tabulated in the Table 3.1
b. The inner diameter of the barrel is 6.40 m at turbine floor level and 12.40 m at generator floor level.
c. The outer diameter of the generator barrel is 14.44 m.
d. The discharge diameter of the turbine is 5.20 m.
e. Length of the Penstock considered in the model is 5.85 m
f. Total length (in X-direction) of the unit considered is 33.45 m.
g. Total width (in Z-direction) of the unit considered is 18.00 m
h. Total height (in Y-direction) of the model is 21.49m
i. Length of the foundation (in X-direction) considered is 100.50 m.

j. Width (in Z-direction) of the foundation considered is 45.00 m

k. Height (Y-direction) of the foundation is 10.75 m

l. Radius of the curvature of the draft tube is 6.28 m

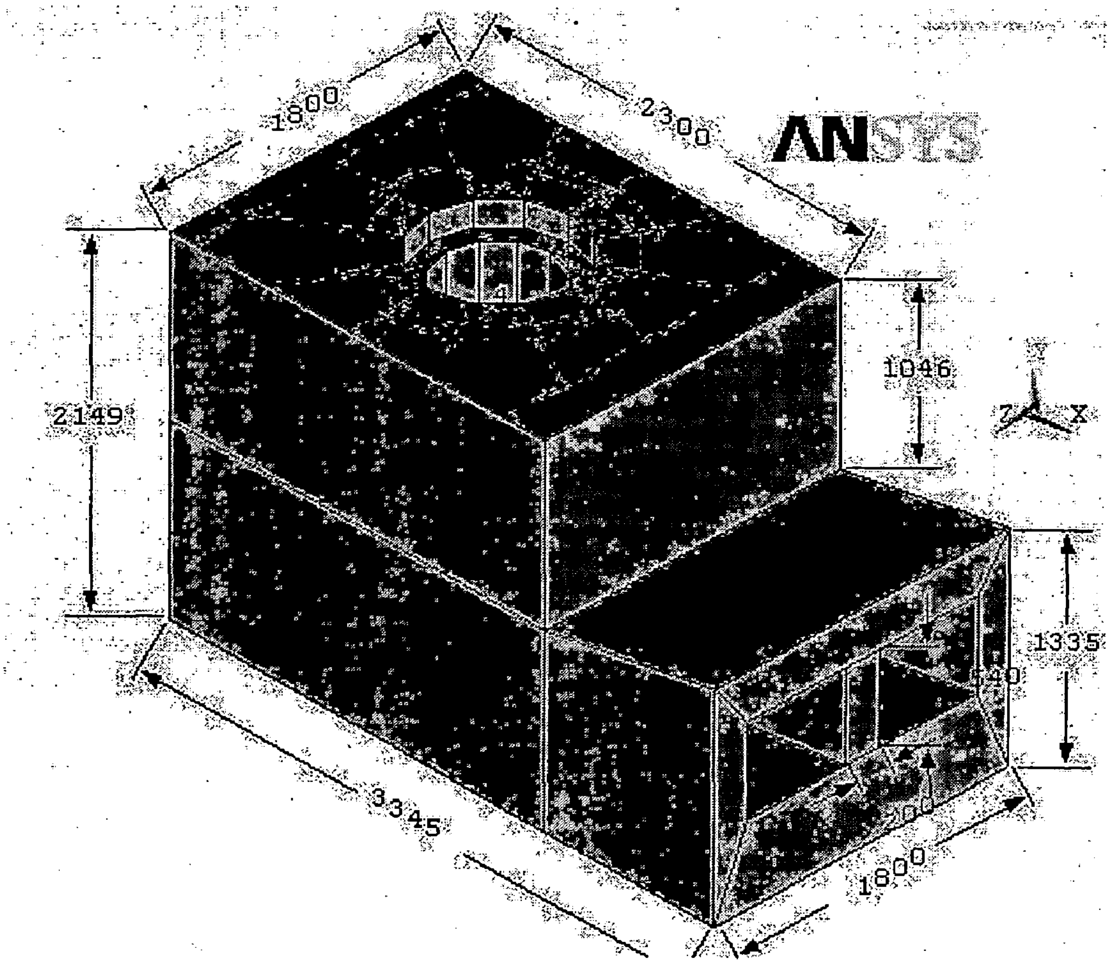


Figure 3.4 Model of the structure (Dimensions in cm)

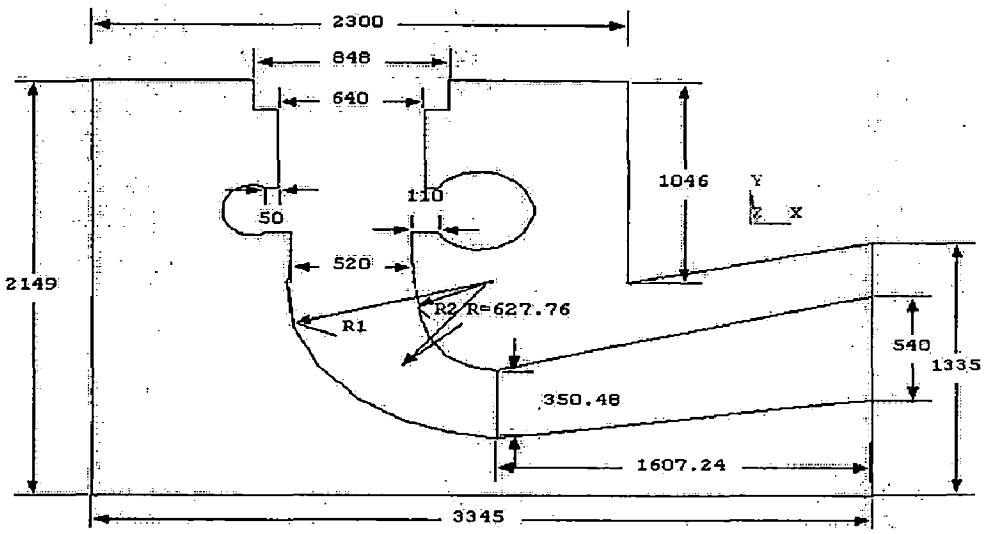


Figure 3.5 Transverse section of the structure (Dimensions in cm)

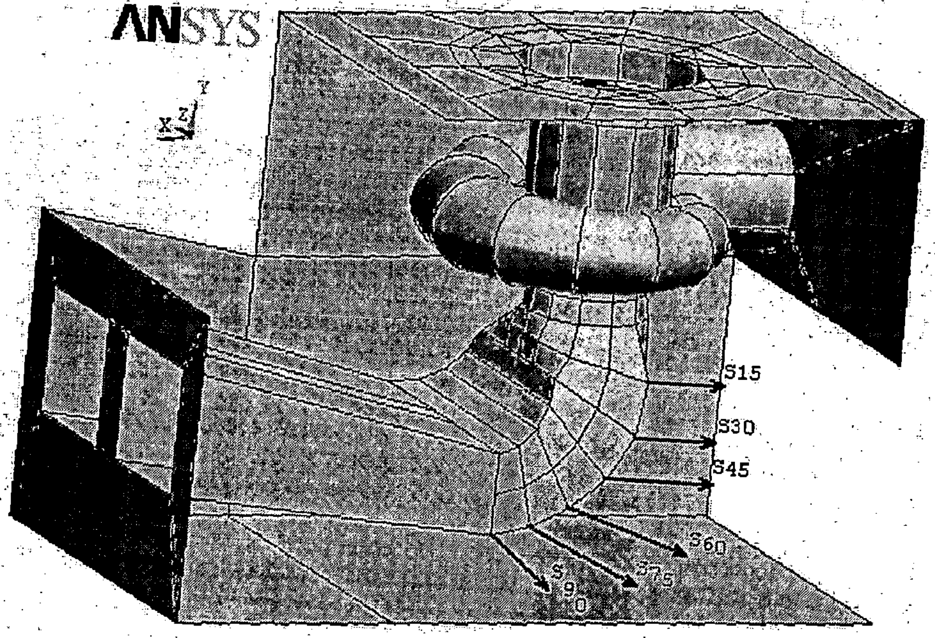


Figure 3.6 Sections of the draft tube bend

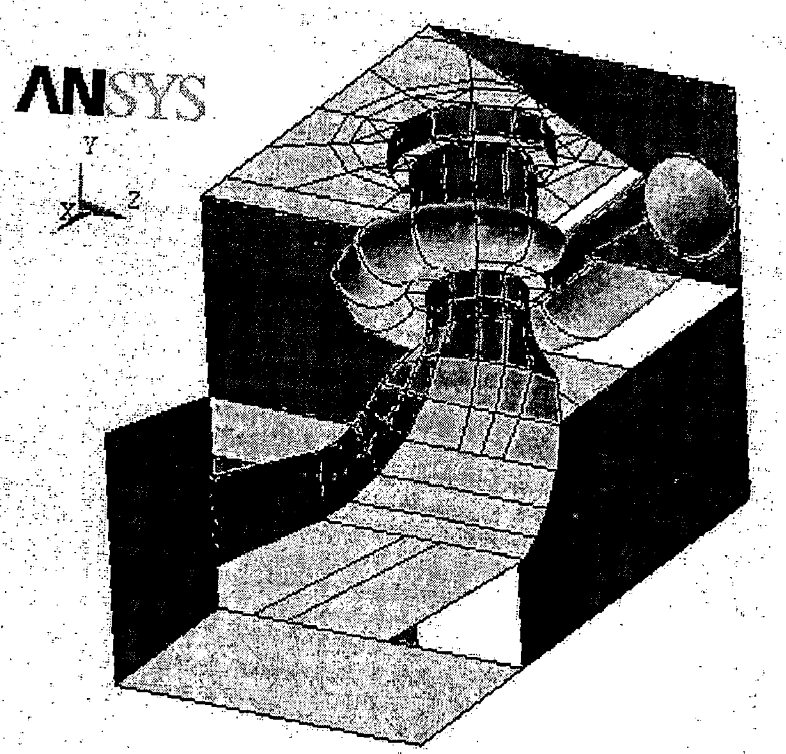


Figure 3.7 Draft tube and Spiral casing

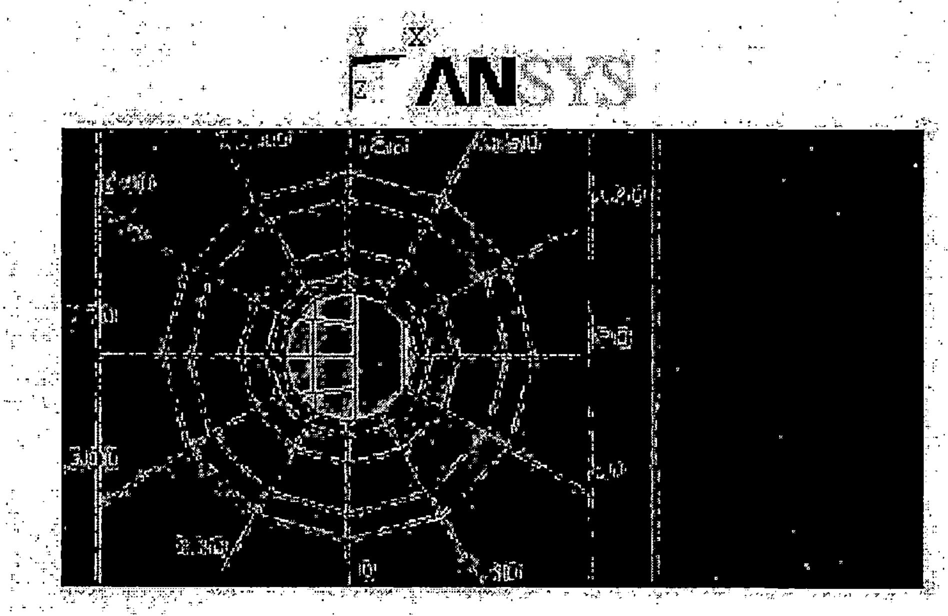


Figure 3.8 Angles for the section of Spiral casing (Top view of the structure)

Table 3.1 Radial and circular dimensions of Spiral casing

Angle (Degree)	**0**	**30**	**60**	**90**	**120**	**150**
R1 (cm)	867.50	840.65	813.81	786.96	755.52	724.08
R2 (cm)	262.50	254.08	245.66	237.23	221.52	205.81
R (cm)	605.00	586.57	568.15	549.73	534.00	518.27
Angle (Degree)	**180**	**210**	**240**	**270**	**300**	**330**
R1 (cm)	692.67	642.16	591.65	541.14	523.21	487.00
R2 (cm)	190.09	164.84	139.58	125.56	121.28	117.00
R (cm)	489.95	877.37	452.07	415.58	401.93	370.00

R1 -Outer radius of the Spiral Casing (from turbine axis)

R2 -Circular radius of the Spiral Casing

R - Canter line radius of the Spiral Casing

Table 3.2 Draft Tube dimensions (Centre line Radius (R=627.76)

Sections	Outer Radius (R1) (cm)	Inner Radius (R2) (cm)	Depth (Z) (cm)
S 15	905.08	350.32	350
S 30	879.68	376.32	400
S 45	848.48	407.52	450
S 60	821.96	434.04	500
S 75	805.84	450.16	550
S 90	803.00	452.52	600

3.5 SUPER STRUCTURE

Super structure has been modelled as described in the section 3.3. The model of the superstructure has been shown in Figure 3.9. This model has been analysed separately and the reactions have been transferred at the intermediate structure at the locations shown in Figure 3.10. The reactions obtained by the analysis are listed in the Table 3.4

3.6 LOADS ON THE STRUCTURE

Different loads on the Intermediate structure are given below except the load from the super structure, which has been calculated separately and transferred on the top of the Intermediate structure at the El. 542.46 m

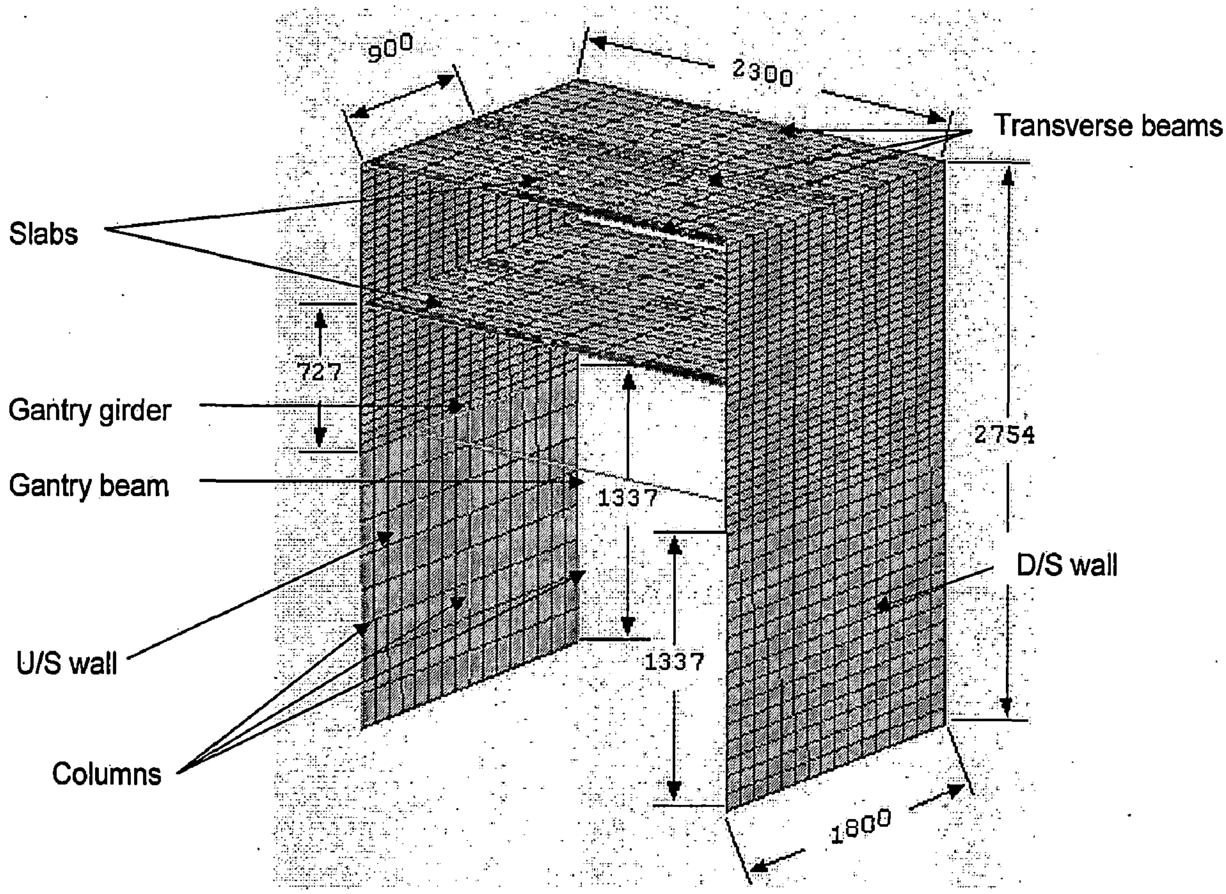


Figure 3.9 Superstructure of the Power House (dimension in cm)

Loads on the Intermediate Structure

Load of the stator on the upper brackets	=	362.84 tonnes
Load of the rotor on the lower brackets	=	615.93 tonnes
Number of brackets at the stator level	=	6
Number of brackets at the rotor level	=	6
Load on the single upper bracket	=	60.47 tonnes
Load on the single lower bracket	=	102.65 tonnes
Short circuit torque on the upper bracket	=	3209 tonnes-m
Tangential load on the single upper bracket	=	307.38 tonnes
Self weight of the concrete	=	2.4 tonnes / m^3
Water pressure inside the spiral casing (including water hammer)	=	69 tonnes / m^2

Short circuit torque has been experienced by the brackets at the stator level. This torque has been resolved in to the load by dividing the value by lever arm (10.44 m). This load being a tangential load has been resolved into two components Fx and Fz to simplify the application of the loads. The resolved components have been tabulated in Table 3.3 and applied at the level of the stator bracket.

Table 3.3 Tangential load components

Bracket No.	Tangential load (tonnes)	Fx (tonnes)	Fz (tonnes)
1	307.38	217.35	-217.35
2	307.38	296.90	79.55
3	307.38	79.55	296.90
4	307.38	-217.35	217.35
5	307.38	-296.90	-79.55
6	307.38	-79.55	-296.90

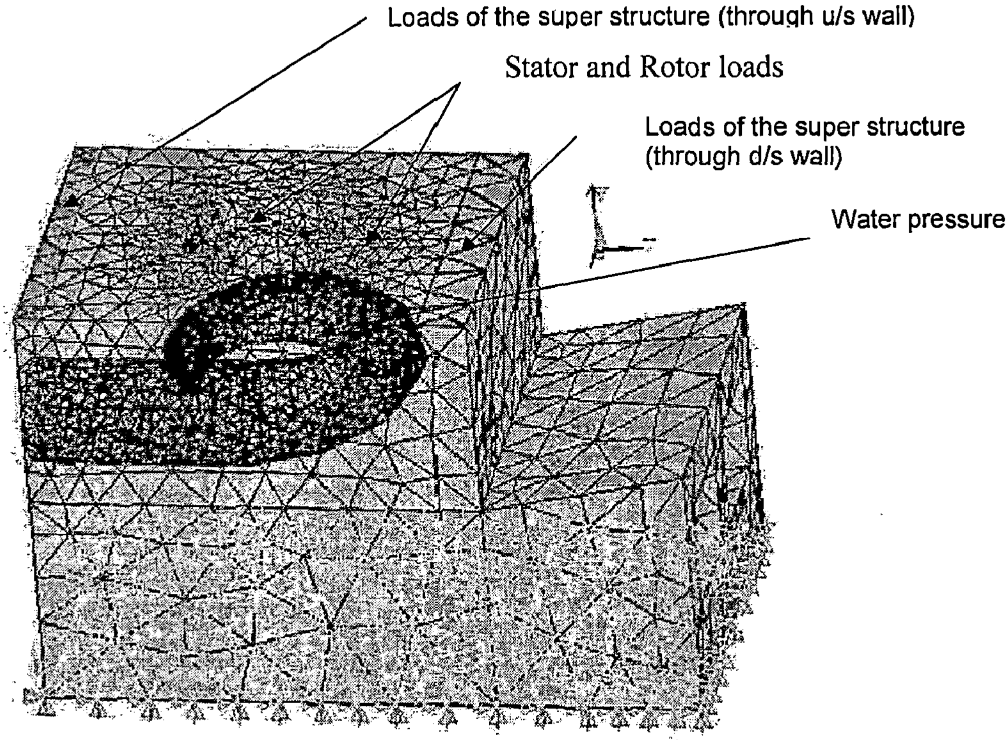


Figure 3.10 Loads on the Structure fixed at the base

3.6.1 Super Structure Loads

The weight of the components of the super structure, as described in the section 3.3 has been calculated as follows

1-Weight of the U/s wall	=	1606 tonnes
2-Weight of the D/S wall	=	2239.74 tonnes
3-Weight of the two slabs	=	695.52 tonnes
4- weight of six transverse beams	=	438.00 tonnes
5-Weight of three U/S columns	=	127.30tonnes
6-Weight of longitudinal Gantry beam	=	657.13 tonnes
7-Weight of gantry beam	=	179.4 0 tonnes
8-Weight of the trolley	=	20.00 tonnes
9-Crane capacity	=	400.00 tonnes

10-Weight of live load on the slab @)0.015 kg/cm2	=	124.20 tonnes
Total Load of the super structure	=	**5887.29 tonnes**

3.7 MODELING STEPS

The following information has been incorporated in the input data for the analysis.

3.7.1 Element type and Material Properties

Element type

A 10 nodes tetrahedral element is selected for 3-D analysis i.e. SOLID92 in the analysis of Sub Structure and Intermediate Structure and BEAM4-3 D and SHELL63 are used in the analysis of Super Structure. A brief description of element capabilities are given below.

Solid 92

It is a 3D; 10-nodes structural solid element. It has three degrees of freedom at each node; translational in nodal x, y, z direction.

Beam 4 -3D

This is a uniaxial element with tension, compression, torsion and bending capabilities. The element has six degrees of freedom at each node; translational in the nodal x, y and z direction and the rotations about the nodal x, y, z axis. The columns and beams of the super structure are meshed with this element.

Shell 63

This element has both bending and membrane capabilities, both in plane and normal loads are permitted. The element has six degrees of freedom at each node i.e. in nodal x, y,z direction

Material properties

M20 concrete is selected for analysis whose properties are as:

- Young' Modulus, E = 2.05×10^5 kg/sq.cm.
- Density of concrete, ρ = 0.0024 kg/cu.m.
- Poison's Ratio, ν = 0.2

3.7.2 Creating The Model

The model of the structure is prepared by the bottom up approach i.e. in order by making key point, line, area, and volume.

3.7.3 Meshing Of Model

Model is meshed using smart size free meshing for level 4 (1 is finer and 10 is coarser) .Sub structure and Intermediate Structure using SOLID 92 element and beams and columns in the Super Structure is meshed by BEAM 4-3D, slabs and walls are meshed with SHEL 63 element. The dimensions of the foundation are depicted in 3.11 and the mesh model has been shown in Figures 3.12 and 3.13.

3.7.4 Boundary Conditions

a) Structure with foundation

At the bottom surface at Y= *-2547.5 cm* (Ux =0, Uy =0, Uz =0)

At the side surfaces at X=1190 cm, X= -1110 cm, Z=900 cm and Z= -900cm (Ux = 0 and Uz = 0).

b) Sub structure fixed at the base

The Boundary condition is applied in such a way that the model is fixed (by seizing all DOF i.e. Ux, Uy, Uz) at the bottom of the sub structure. For that all nodes of lowest plane (Y= -1473 cm) are made fixed.

c) Super structure

Superstructure has been considered fixed at the base because Intermediate structure and sub structure has a large concrete mass resulting in greater rigidity as compared to the superstructure as shown in Figure 3.9.

3.7.5 Loading Conditions

Loads other than self weight were applied in following two conditions

A) Rotor load , stator load, short circuit torque (clock wise direction)

B) Rotor load, stator load, short circuit torque (clock wise direction), water pressure in the spiral casing and superstructure load.

Load Case - A

This loading condition has been applied for the different foundation conditions Under this load condition following values of modulus of elasticity of the foundation have been used.

a) Modulus of elasticity of the foundation material (E)is 10 times of the modulus of elasticity of concrete (Ec).i.e. $E=10^* Ec$

b) Modulus of elasticity of the foundation material (E)is equal to the modulus of elasticity of concrete (Ec).i.e. $E=Ec$.

c) Modulus of elasticity of the foundation material (E)is 1/10th of the modulus of elasticity of concrete (Ec).i.e. $E=Ec/10$.

d) Modulus of elasticity of the foundation material (E)is 1/100th of the modulus of elasticity of concrete (Ec).i.e. $E=Ec/100$.

e) Sub structure fixed at the foundation

LOAD CASE - B

Structure has been analysed in the following two conditions under this loading arrangement

a) Structure with the foundation modulus of elasticity of the foundation material (E)is equal to the modulus of elasticity of concrete (Ec).i.e. E=Ec.

b) Sub structure fixed at the foundation.

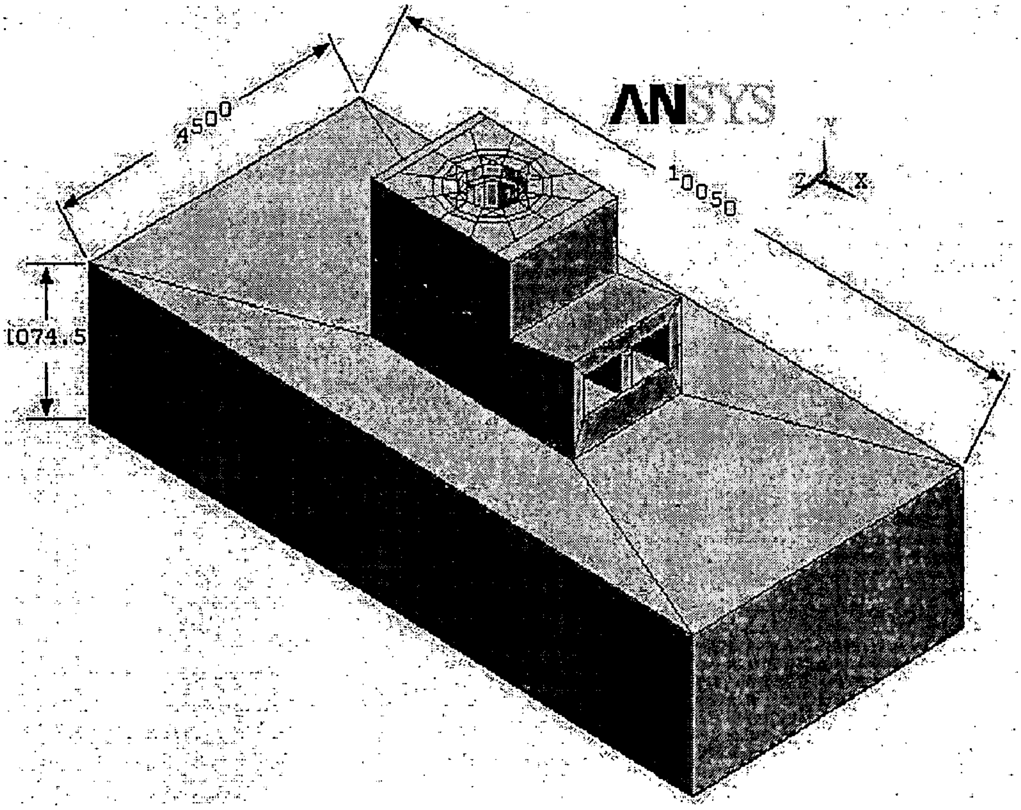


Figure 3.11 Dimensions of the foundation

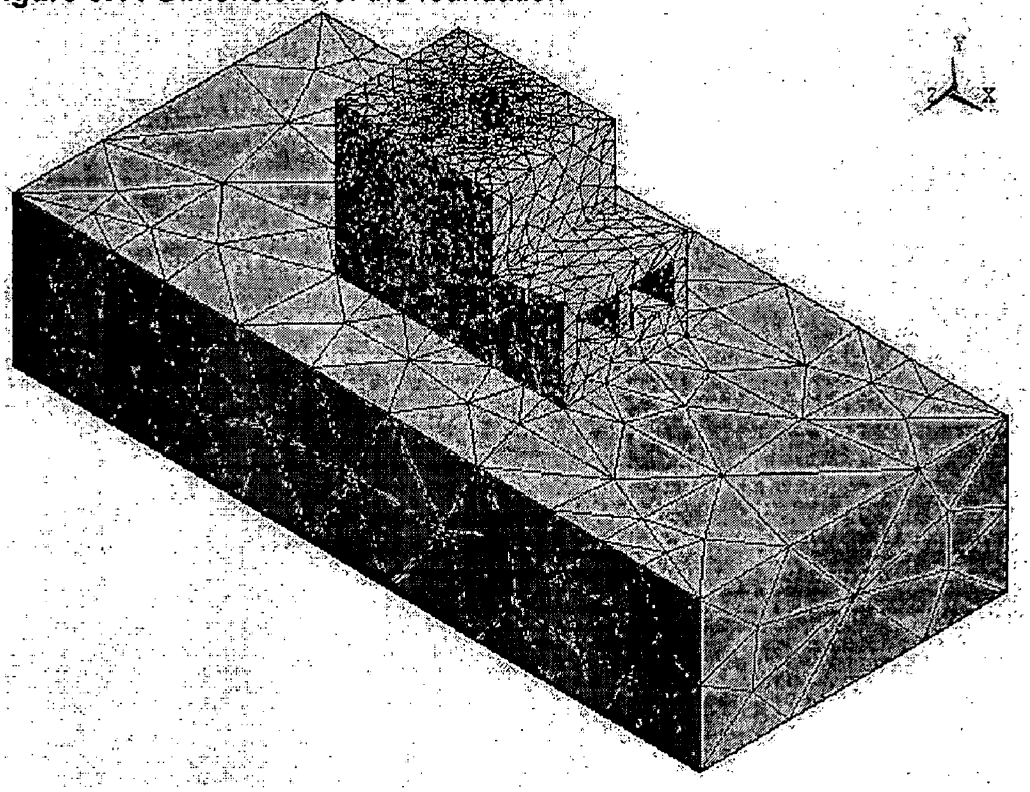


Figure 3.12 Meshed model with foundation

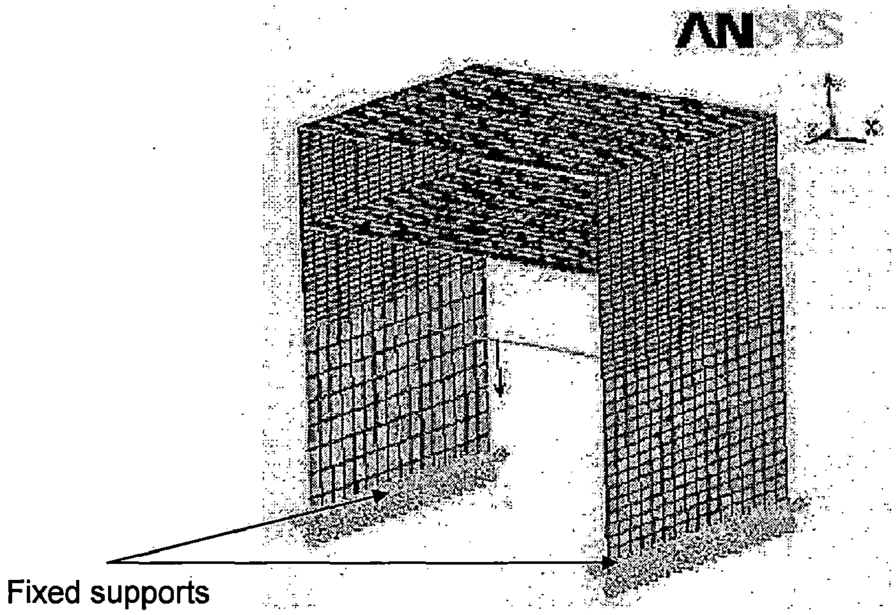
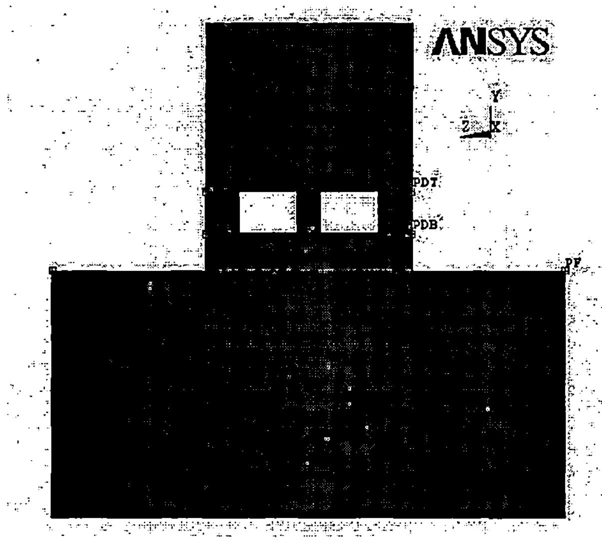


Figure 3.13 Meshed model of super structure and boundary conditions



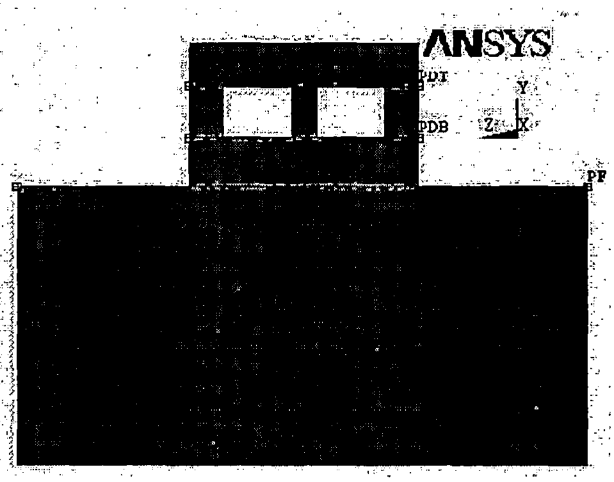


Figure 3.14 u/s section of the draft tube **Figure 3.15** d/s section of the draft tube

3.7.6 Locations For The Results

In this analysis following two sections on the draft tubes have been considered for the results.

1- At 800 cm from the axis of the turbine towards the flow, (u/s section), which passes through the elbow portion of the power house structure.

2- At 1200 cm from the axis of the turbine towards the flow, (d/s section) which is out side the d/s wall of the power house structure.

Above sections were considered in the longitudinal direction of the one bay of the power House.

Following three locations on each section have been selected, as shown in Figures 3.14 and 3.15.

i) At the top of the draft tube openings (PDT)

ii) At the bottom of the draft tube openings. (PDB)

iii) At the foundation contact of the structure (PF)

The results have also been obtained at the following sections of the model of the power house.

1- Transverse Section of the model through the turbine axis

2- Longitudinal Section of the model through the turbine axis

Table 3.4 –Reactions transferred from the super structure

RIGHT BASE REACTION SOLUTIONS PER NODE
(Unit- Fx, Fy, Fz in kg and Mx, My, Mz in kg-cm)

NODE	FX	FY	FZ	MX	MY	MZ
104	-6597.3	0.14489E+06	-29279.	932.07	0.12913E+06	0.11857E+07
105	-5655.0	0.11442E+06	23986.	-916.78	-85586.	0.88678E+06
106	-11212.	0.23163E+06	-28499.	764.51	-58541.	0.20897E+07
107	-8298.3	0.21542E+06	-20694.	-198.21	-20759.	0.17980E+07
108	-7644.5	0.20974E+06	-15032.	-40.904	-16516.	0.15983E+07
109	-7248.1	0.20554E+06	-10677.	-11.258	-17234.	0.14318E+07
110	-6659.6	0.20197E+06	-6962.6	21.520	-19194.	0.12580E+07
111	-5838.1	0.19856E+06	-3720.1	37.452	-20693.	0.10678E+07
112	-4853.9	0.19518E+06	-807.36	24.679	-20355.	0.87076E+06
113	-3826.3	0.19179E+06	1869.9	41.488	-17562.	0.68640E+06
114	-2879.0	0.18841E+06	4404.5	18.753	-12423.	0.53720E+06
115	-2111.0	0.18510E+06	6900.0	-21.787	-5475.7	0.44297E+06
116	-1589.1	0.18192E+06	9474.7	3.3547	2676.4	0.41869E+06
117	-1363.1	0.17899E+06	12285.	15.108	11610.	0.47541E+06
118	-1507.9	0.17673E+06	15483.	20.650	20979.	0.62453E+06
119	-2376.8	0.17584E+06	19565.	177.81	31877.	0.88920E+06
120	-5753.3	0.18407E+06	24935.	-756.68	67226.	0.13036E+07
TOTAL VALUES						
VALUE	-85413.	0.31802E+07	3232.1	111.77	-30834.	0.17565E+08

LEFT BASE REACTION SOLUTIONS PER NODE
(Unit- Fx, Fy, Fz in kg and Mx, My, Mz in kg-cm.)

NODE	FX	FY	FZ	MX	MY	MZ
1	-2892.1	0.19904E+06	22309.	12187.	0.17895E+06	-0.52200E+06
11	1290.6	0.26135E+06	-28056.	-15619.	-0.48757E+06	-0.20354E+07
20	5094.4	0.26336E+06	1140.0	408.93	11349.	-0.28215E+07
1665	6048.0	0.14448E+06	-1495.0	-125.66	10697.	-0.31139E+07
1666	6998.5	0.14631E+06	-3799.1	-737.17	9739.3	-0.34144E+07
1667	7837.7	0.14840E+06	-6394.0	-537.85	7455.6	-0.36906E+07
1668	8559.4	0.15059E+06	-9292.9	-308.36	4361.6	-0.39334E+07
1669	9346.2	0.15312E+06	-12622.	-88.657	3345.8	-0.41457E+07
1670	10574.	0.15632E+06	-16914.	-19.825	14810.	-0.43227E+07
1671	11840.	0.16415E+06	-21600.	-424.97	64286.	-0.43432E+07
1721	2516.4	0.12771E+06	18769.	311.21	-25015.	-0.14163E+07
1722	2707.1	0.12578E+06	15491.	482.15	-6639.1	-0.16051E+07
1723	2510.4	0.12765E+06	12362.	406.89	1055.3	-0.17221E+07
1724	2548.9	0.13012E+06	9901.7	341.17	4251.7	-0.18555E+07
1725	2860.9	0.13295E+06	7696.9	260.77	6158.9	-0.20302E+07
1726	3410.7	0.13594E+06	5624.1	291.89	7880.5	-0.22509E+07
1727	4161.9	0.13926E+06	3647.8	189.88	9398.7	-0.25135E+07
TOTAL VALUES						
VALUE	85413.	0.27065E+07	-3232.1	-2981.5	-0.18549E+06	-0.45737E+08

CHAPTER - 4

RESULTS AND DISCUSSION

4.1 GENERAL

The analysis of the substructure, intermediate structure and the superstructure has been carried out using ANSYS software of Finite Element Method. The details have been narrated below in subsequent paragraphs.

Results were obtained at the locations, whose details have been shown in Figure 3.14 and 3.15, chapter 3, stresses and displacements have been plotted on the graphs along the following three paths.

i) At the top of the draft tube openings (PDT)

ii) At the bottom of the draft tube openings.(PDB)

iii) At the foundation contact of the structure.(PF)

Stresses and deflections have also been plotted on contours

i) Along the transverse direction of the unit bay through the axis of the turbine.

ii) Along the longitudinal direction of the unit bay through the axis of the turbine

Maximum values of first principal stress S1 (tension governing stress) and the third principal stress, S3 (compression governing stress) has been tabulated in the Tables 4.1 and 4.2. Results of the analysis are discussed under the following headings.

4.2 RESULTS OF ANALYSIS (LOAD CASE-A)

Under this loading condition as described under para 3.7.5, Chapter 3, the results have been obtained for u/s section, which are shown in Figures 4.1 to 4.28.

4.2.1 U/S section (800cm from the turbine axis)

This section passed through the elbow portion of the draft tube. The results were obtained at the following three locations of the section under different foundation conditions

4.2.1.1 At the top of the draft tube openings

At the top of the opening of the draft tube in all the foundation conditions it was noticed that the first principal stress, S1 is tensile in nature having maximum tensile value of 3.55 kg/cm^2 in the foundation condition, E=10*Ec and a value 3.04 kg/cm^2 in the foundation condition, E=Ec/100..This tensile stress is at 540cm from right edge of the section (Figures-4.1 and 19)..Other values are 3.47, 3.25 kg/cm^2 for the foundation condition E=Ec and E=Ec/10 respectively (Figure-4.7and 4.13). The value of tensile stress is found increasing with the decrease in the rigidity of the foundation.

Maximum compressive stress S3 occurred in the region above the pier in every foundation condition (Figures 4.1, 4.7, 4.13 and 4.19) The value of the compressive stress varies from 8.36 kg/cm^2 in the foundation condition (E=Ec) to 7.14 kg/cm^2 in the foundation condition (E=Ec/100). The compressive, stress, S3 decreased with the decrease in the rigidity of the foundation and did not show the major change among the values (Table 4.1)

Maximum vertical displacement increased with the decrease in the stiffness of the foundation and it is noticed at 630 cm from edges of either side of the structure. The minimum value of the order of 0.03, 0.05, 0.22, and 1.84 cm in the foundation conditions, E=10*Ec, E=Ec, E=Ec/10 and E=Ec/100 respectively. (Figure 4.2, 4.8, 4, 14, and 4.20).This variation has also been tabulated in the table 4.1.The maximum values of Ux and Uz are always less than the value of Uy in all foundations under consideration. The maximum value of Uz was observed of the order of 0.009cm in positive Z direction and of Ux was observed 0.003 in negative X direction in the foundation condition E=10Ec(Figure-4.2).With the decrease of rock modulus of the foundation Uz did not show a uniform pattern of deflection(Figure-4.20).As Values of displacement in X and Z direction is not so significant as compared to displacement in Y direction.

4.2.1.2 At the bottom of the Draft Tube Openings

Maximum tensile stress was noticed at the opening at 540 cm from the left edge of the section At this location the value of the tensile stress, S1, decreased with the decrease in the rigidity of the foundation (Figure-4..3, 4.9, 4.15, and 4.21) varying from 2.58 kg/cm^2 for foundation condition, E= 10*Ec to 2.30 kg/cm^2 for the foundation condition, E=Ec/100.Other values were 2.47, 2.39 kg/cm^2 for the foundation condition E=Ec and E=Ec/10 respectively. This variation has also been tabulated in the table 4.1.

Maximum compressive stress S3 occurred below the pier of the draft tube, varying from 8.46 kg/cm^2 for the foundation condition, E=10*Ec to 7.13 kg/cm^2 for the foundation condition, E=Ec/100. (Table4.1). At this location the compressive stress S3 decreases with the decrease in the foundation modulus. The values of the order of 8.31$kg/cm^2$and 7.74 kg/cm^2 were noticed for the foundation condition E=Ec and E=Ec/10 respectively The values did not show much variation in the values with the change in the stiffness of the foundation. .This maximum value occurs at 540 cm from the left edge of the section (Figure-4.3, 4.9, 4.15 and 4.21)

Maximum vertical displacement increased with the decrease in the stiffness of the foundation. The values for the vertical deflections was noticed of the order of .0.01, 0.03, 0.20 and 1.80 cm for the foundation condition E=10*E, Ec,Ec/10and Ec/100 respectively Maximum displacement in each case of different foundation was found at the left edge of the section (Figure-4.4, 4.10,4.16,4.22). The variation in the vertical deflection with the different foundation condition has also been tabulated in table-4.1 Maximum values of UX and Uz have been observed much less than the Uy in all the foundation conditions.

4.2.1.3 At the foundation contact

Maximum tensile stress was observed approximately 6.75m from the left edge of the foundation. Variation in the values did not show definite pattern but the values of this stress did not show major difference in the values from the foundation condition, E=10*Ec to Ec/10 but the maximum value of tensile stress changes its location for the foundation condition, E=Ec/100, having maximum value 1.96 kg/cm2 at the middle of the section (Figure 4.5, 4.11, 4.17, 4.23).This variation has also been tabulated in Table-4.1

Maximum compressive stress, S3, at the section occurred at the left edge of the structure. But not very appreciable variation is noticed among the values of different foundation conditions. There was no definite variation pattern observed as shown in the Table-4.1.

Maximum vertical displacement is found 225 cm inside from the both edges of the structure in all other foundation conditions except in the foundation condition, E=Ec/100, where the maximum value of displacement occurs at the right edge of the structure. The values of the vertical displacement were of the order of.002, 0.02, 0.19and 1.82cm for the foundation condition E=10* Ec, Ec, Ec/10 and Ec/100 respectively as shown in the Figures-4.6, 4.12, 4.18 and 4.24. It was observed that maximum displacement increases with the decrease in the rigidity of the foundation. This is also tabulated in the Table 4.1 Vertical settlement of the draft tube base for the foundation condition E=Ec/100 is excessive and is not desirable for the structure of a power house. Ux andUz were having the lesser maximum values as compared to Uy in all the foundation conditions.(Figure-4.34)

4.2.1.4 Sub structure fixed at the base.

The maximum values of S1, S3, Ux,Uy and Uz (Figures 4.25,4.26,4.27,4.28and in the table-4.1) in this case are found very close with the values when foundation material is assumed such that E=10*Ec at all the three locations as discussed above. This reveals that the sub structure may be assumed fixed at base if the foundation rock has a modulus ten times of concrete.

LOAD CASE –A

(A) FOUNDATION CONDITION (E=10*Ec)

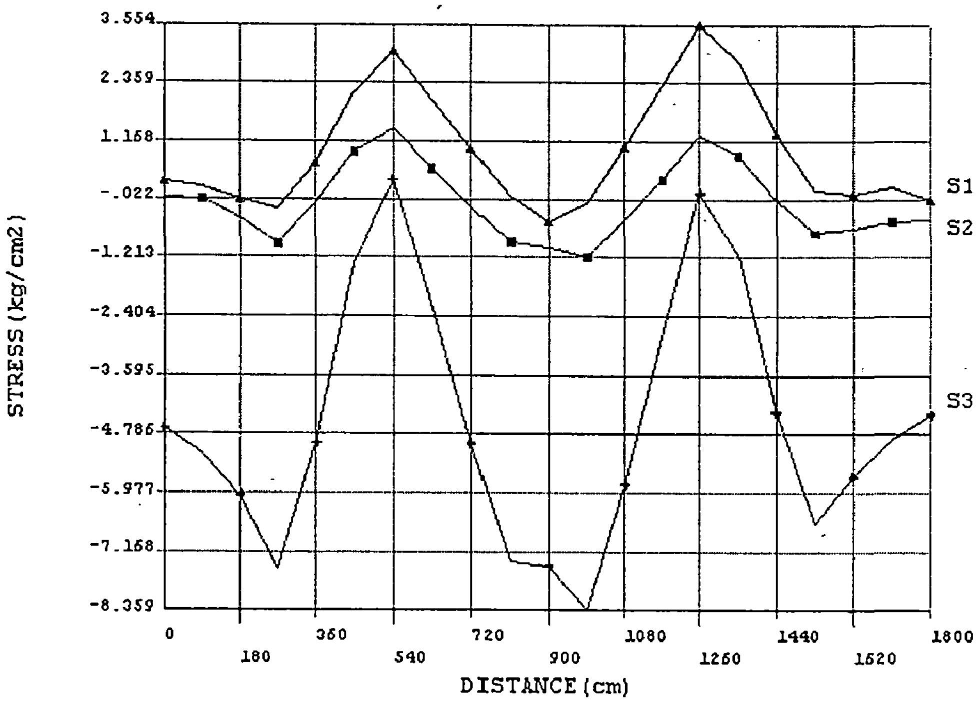


Figure 4.1 Principal Stress at top of the draft tube openings

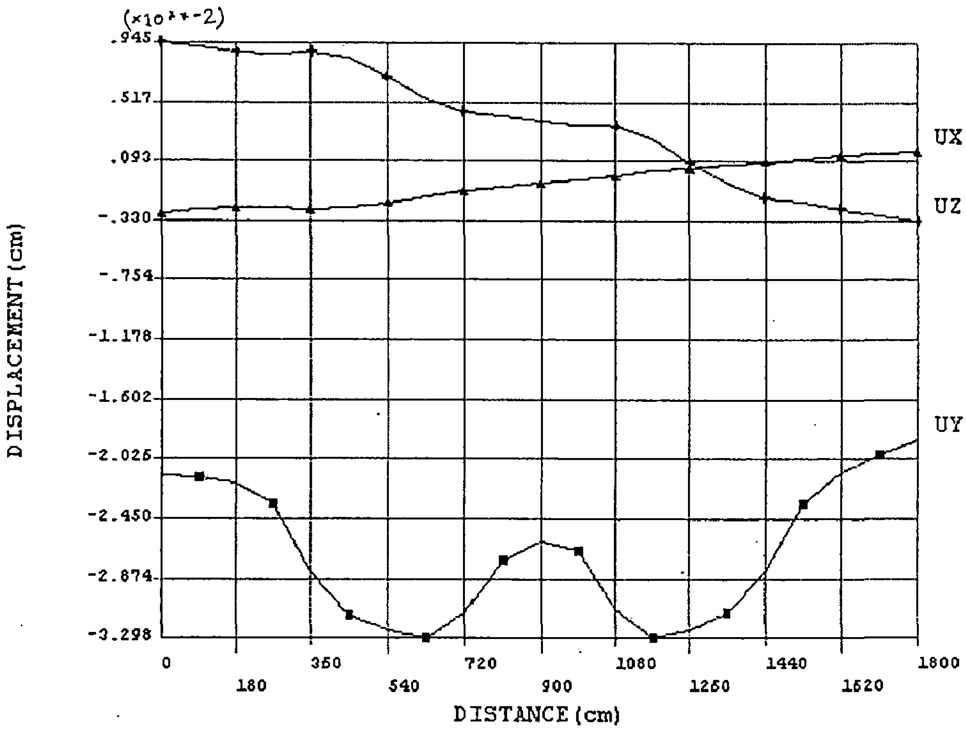


Figure 4.2 Displacement at top of the draft tube openings

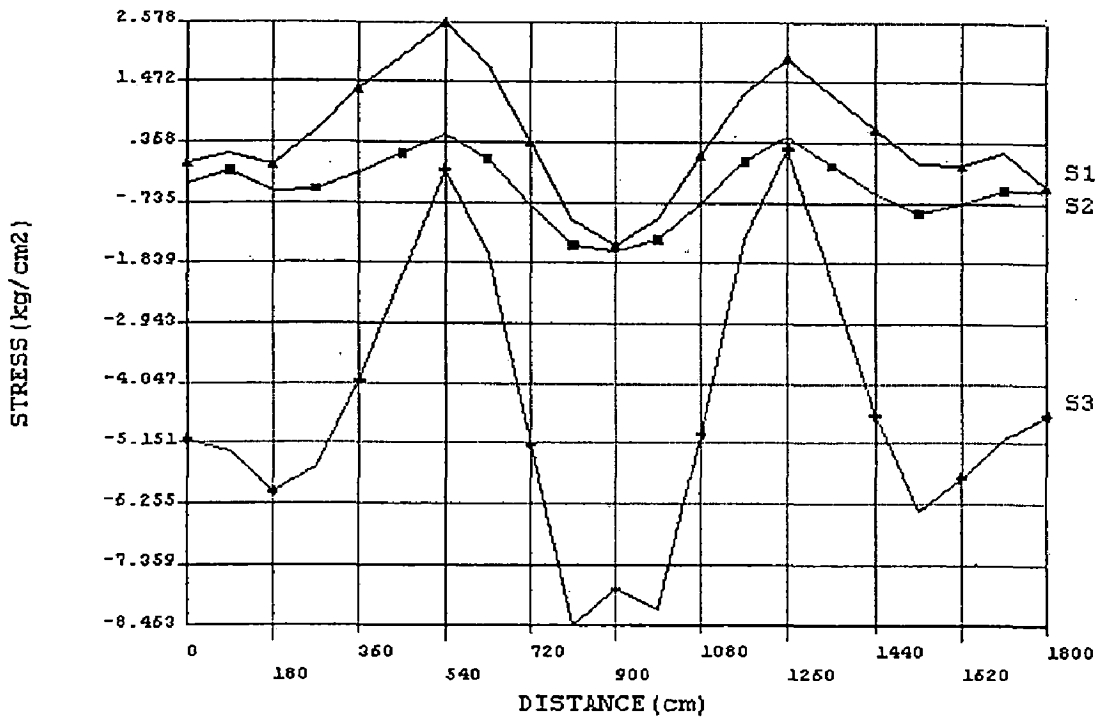


Figure 4.3 Principal Stress at bottom of the draft tube openings

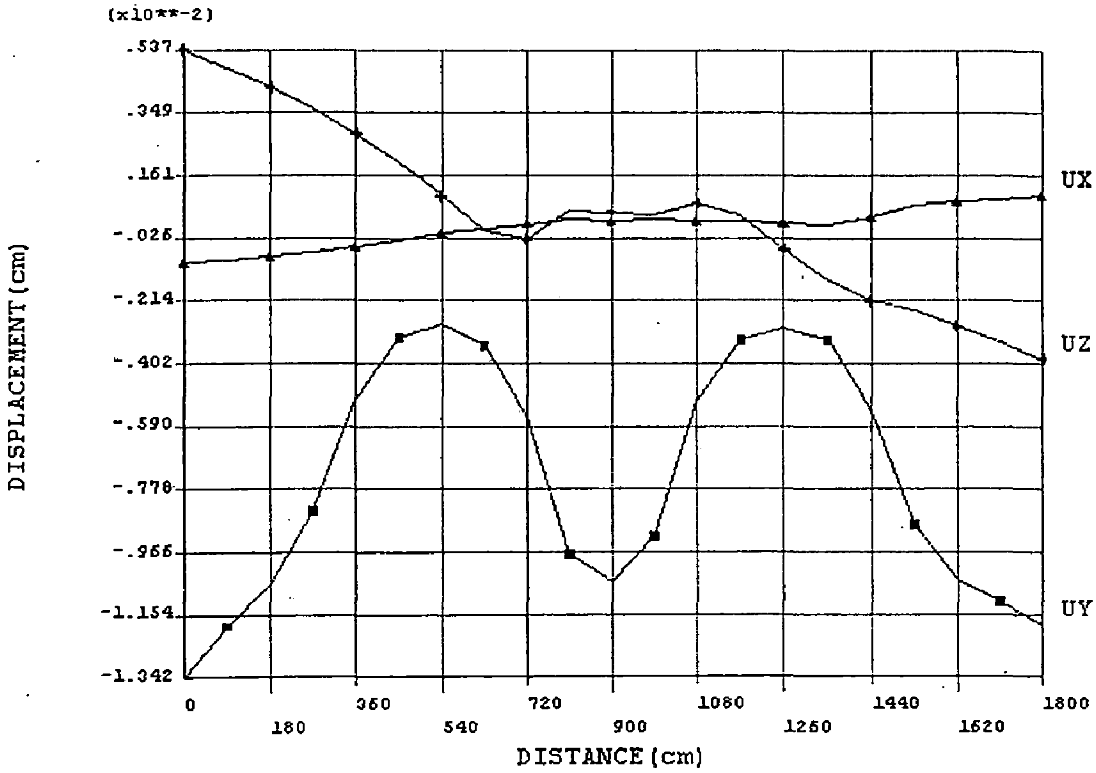


Figure 4.4 Displacement at bottom of the draft tube openings

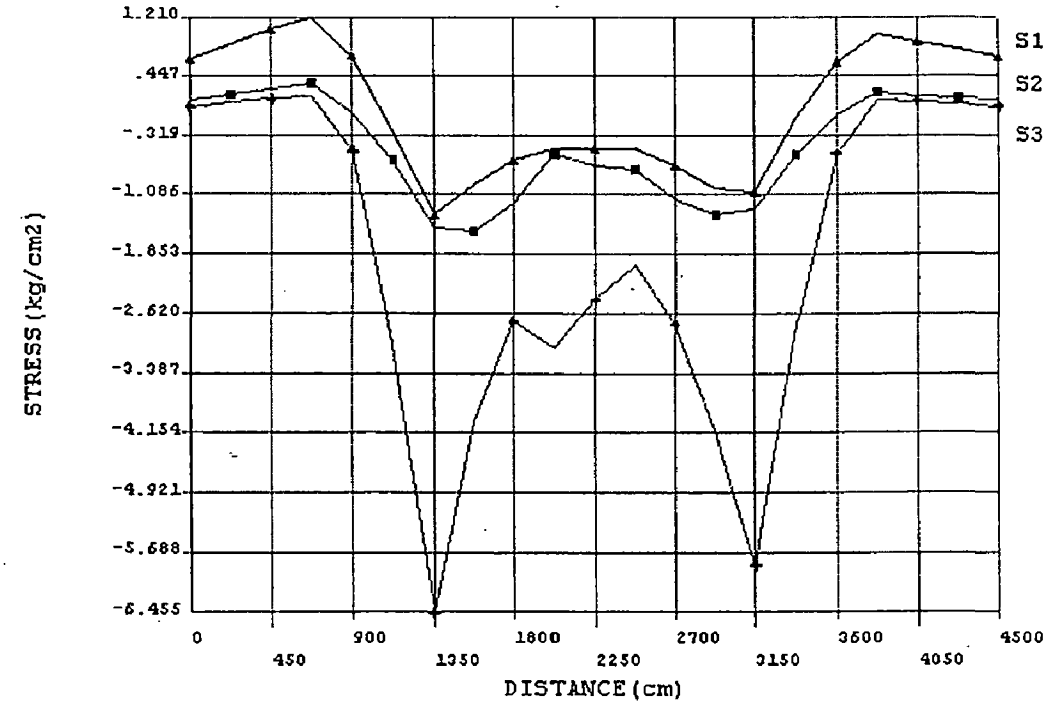


Figure 4.5 Principal Stress at foundation contact

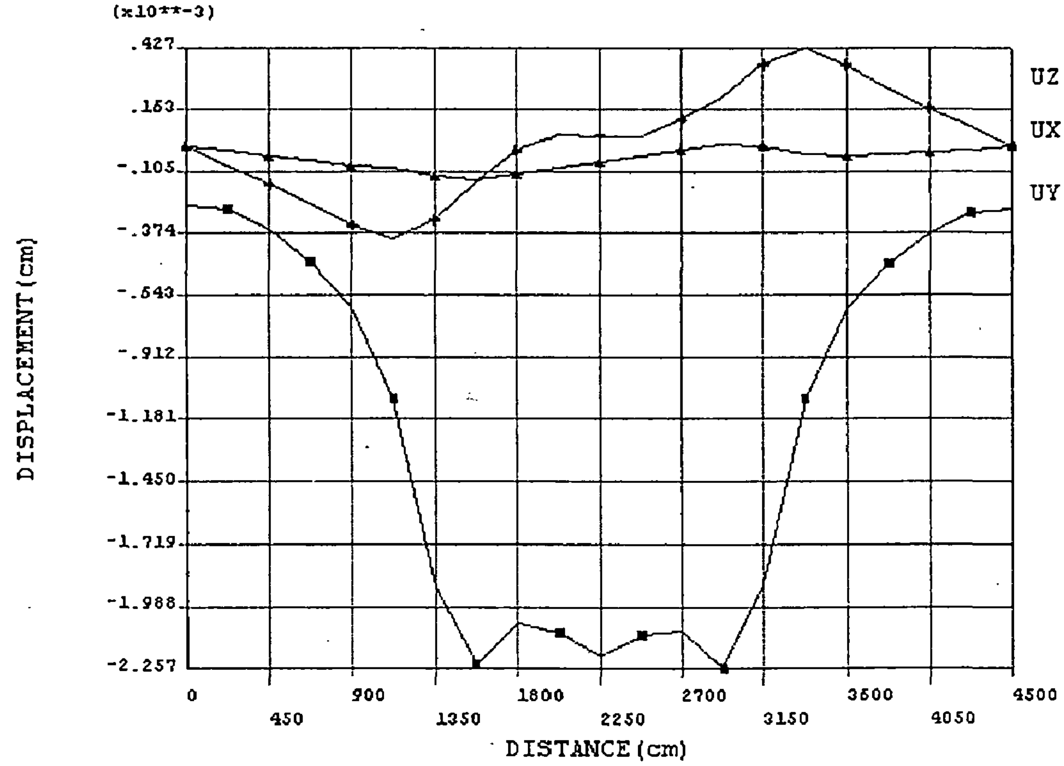


Figure 4.6 Displacement at foundation contact

(B)FOUNDATION CONDITION (E=Ec)

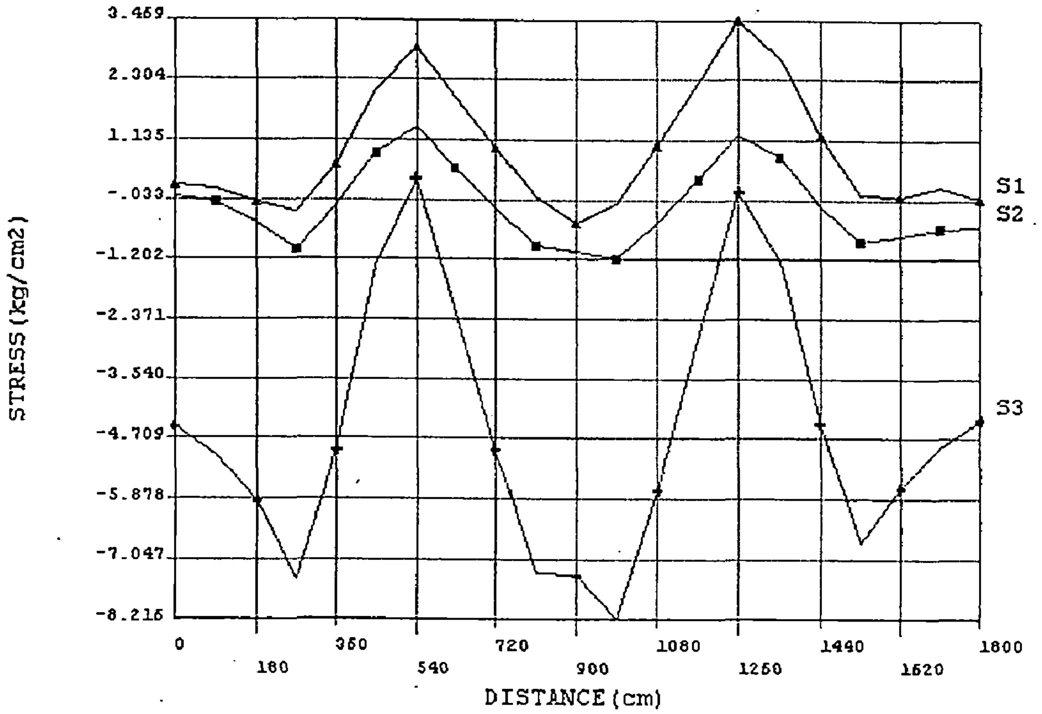


Figure 4.7 Principal Stress at top of the draft tube openings

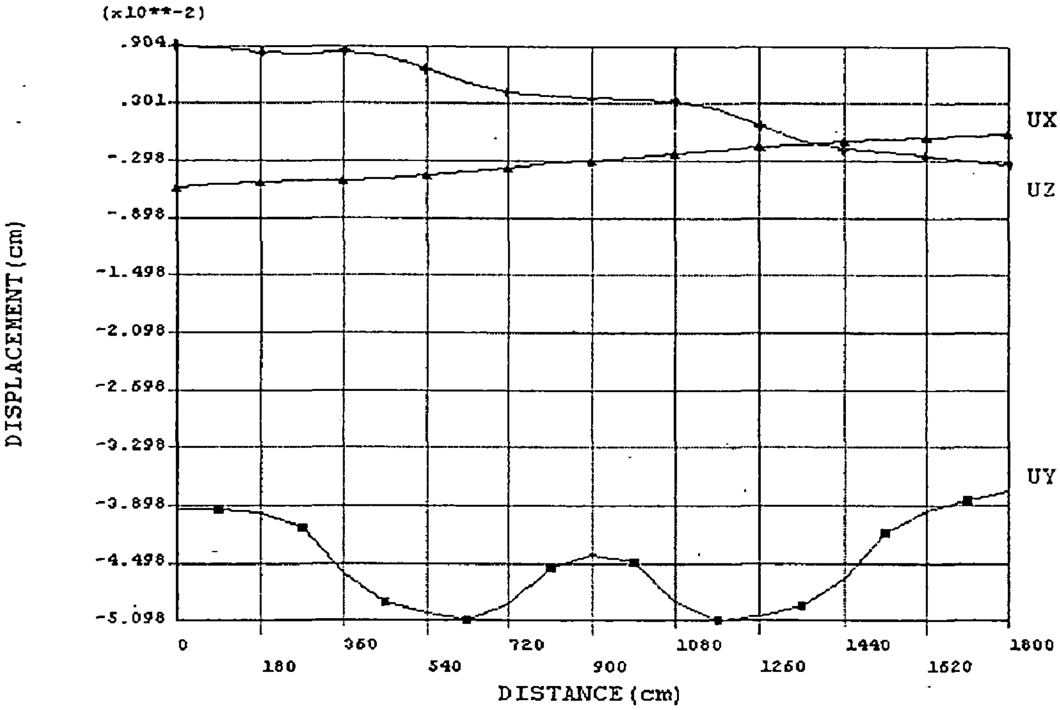


Figure 4.8 Displacement at top of the draft tube openings

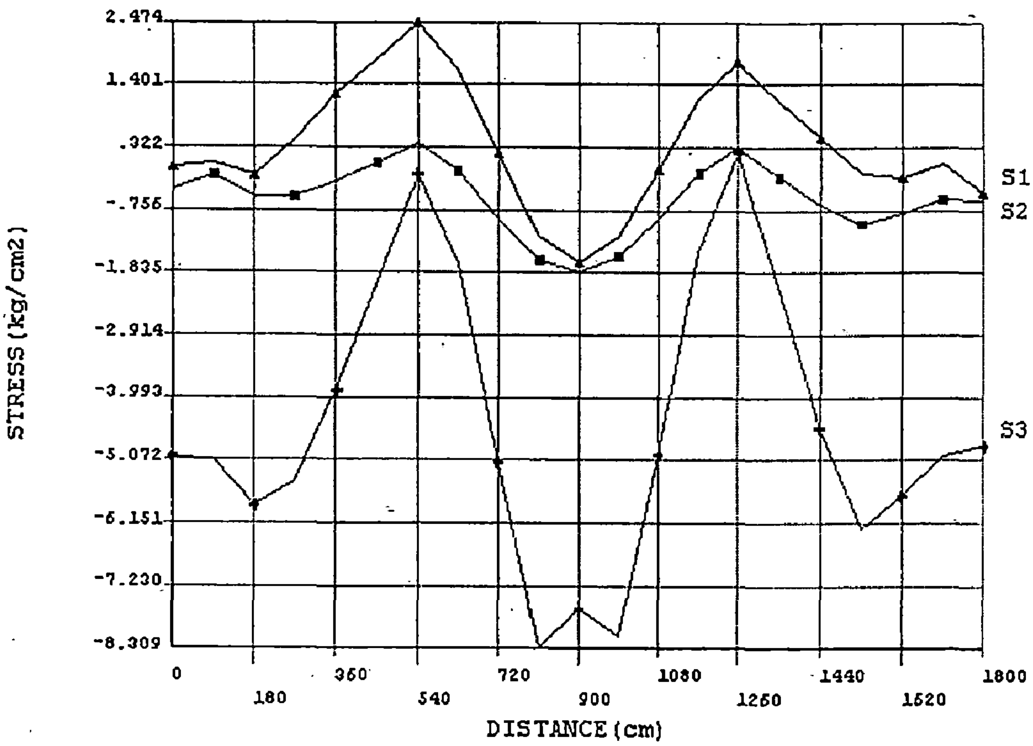


Figure 4.9 Principal Stress at bottom of the draft tube openings

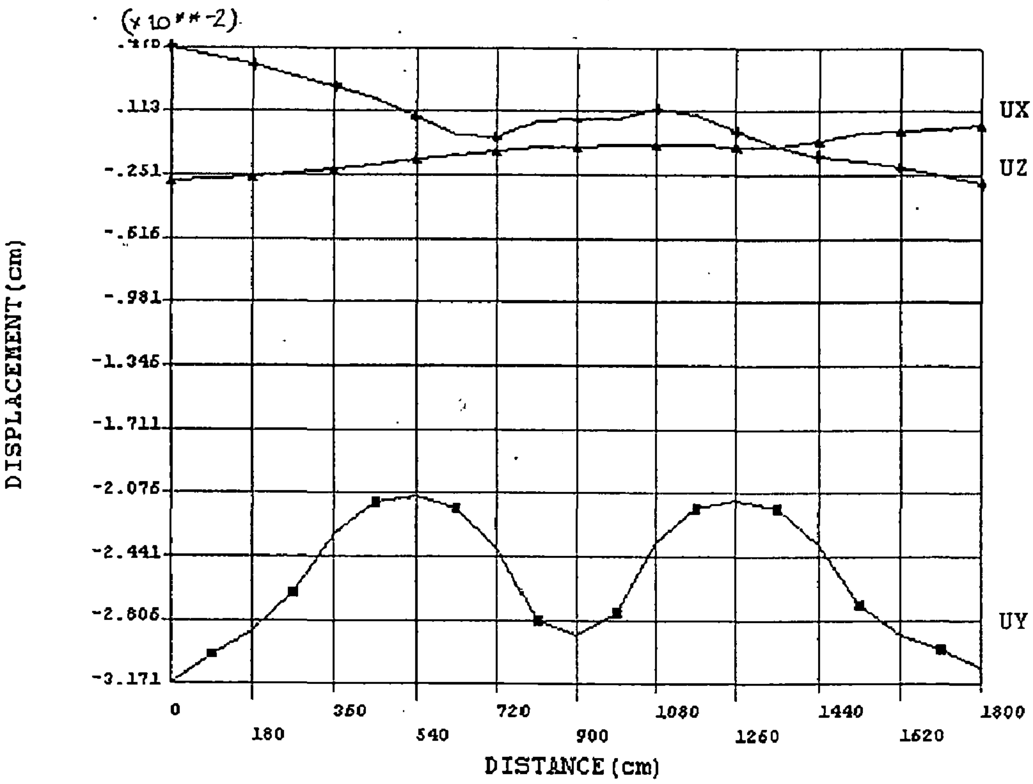


Figure 4.10 Displacement at bottom of the draft tube openings

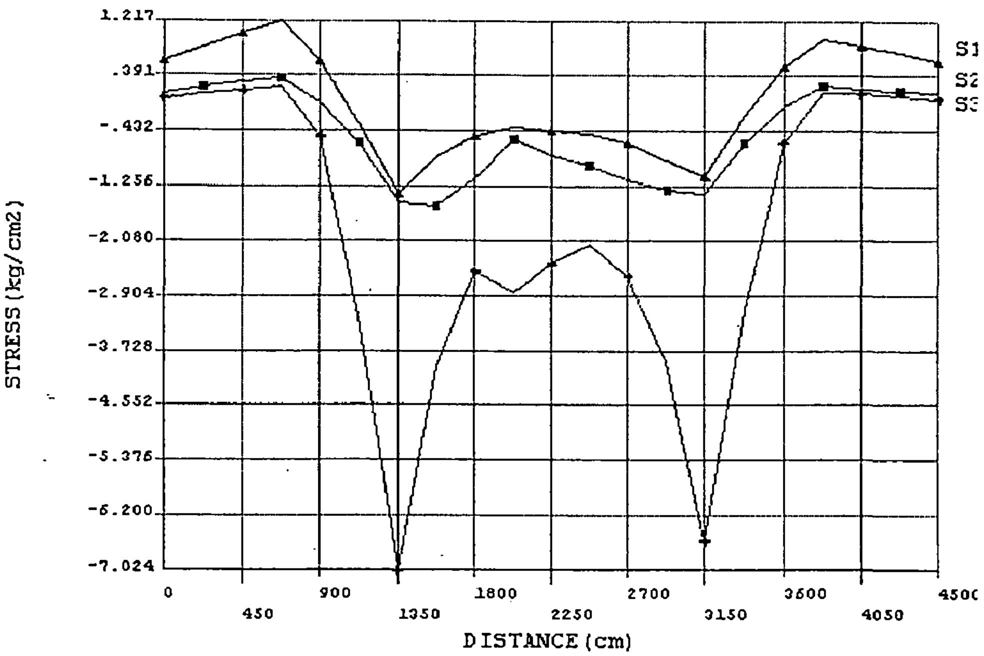


Figure 4.11 Principal Stress at foundation contact

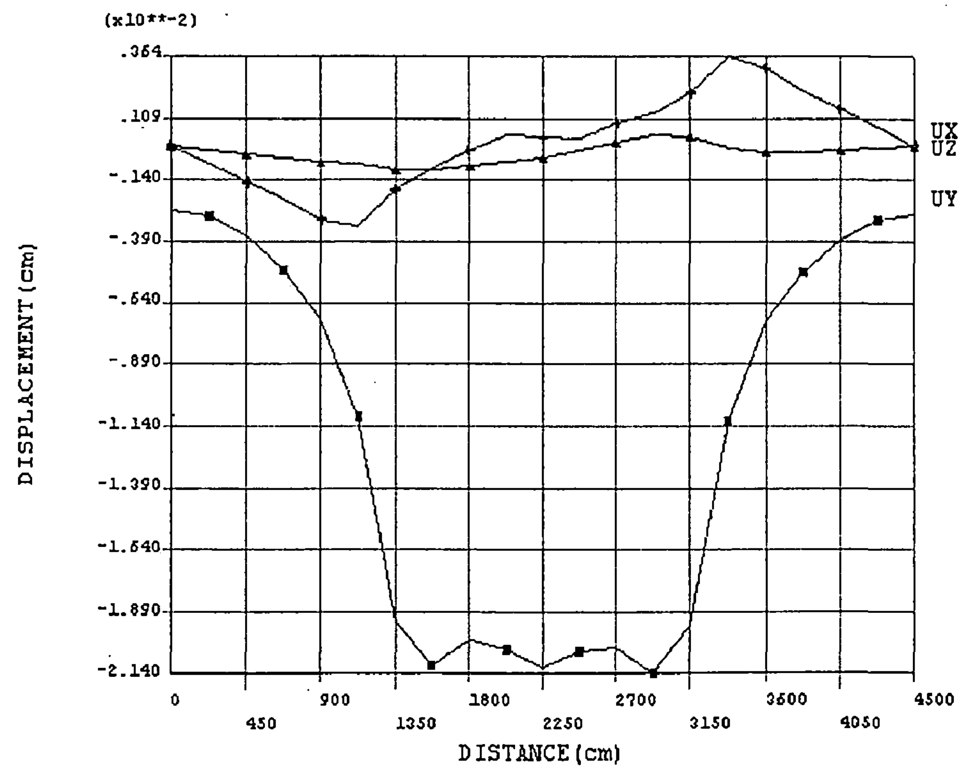


Figure 4.12 Displacement at foundation contact

(C) U/S SECTION AT THE DRAFT TUBE (E=Ec/10)

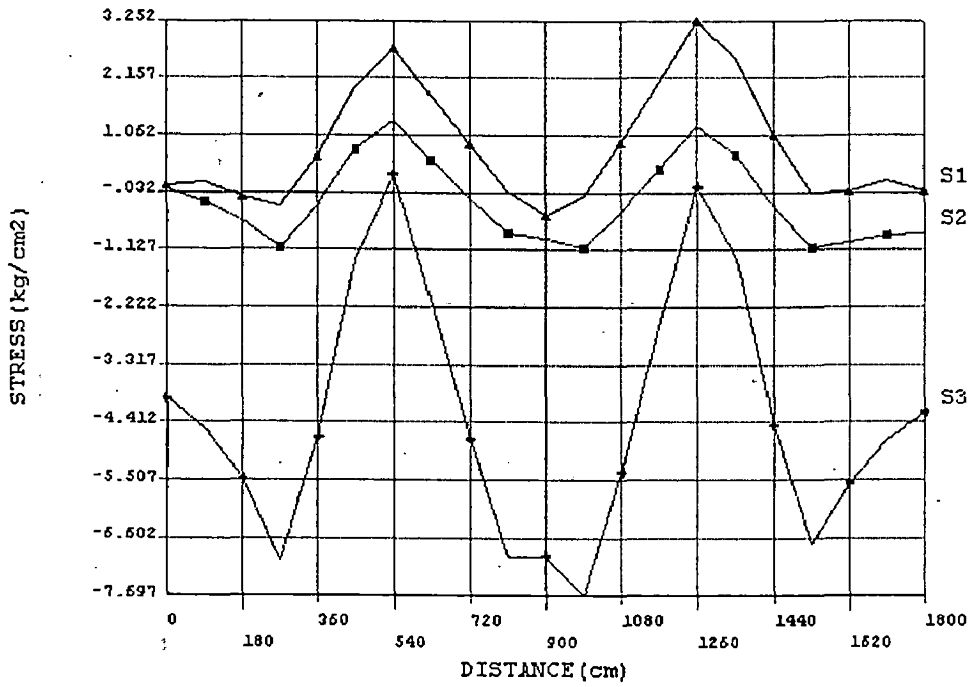


Figure 4.13 Principal Stress at the top of the draft tube openings

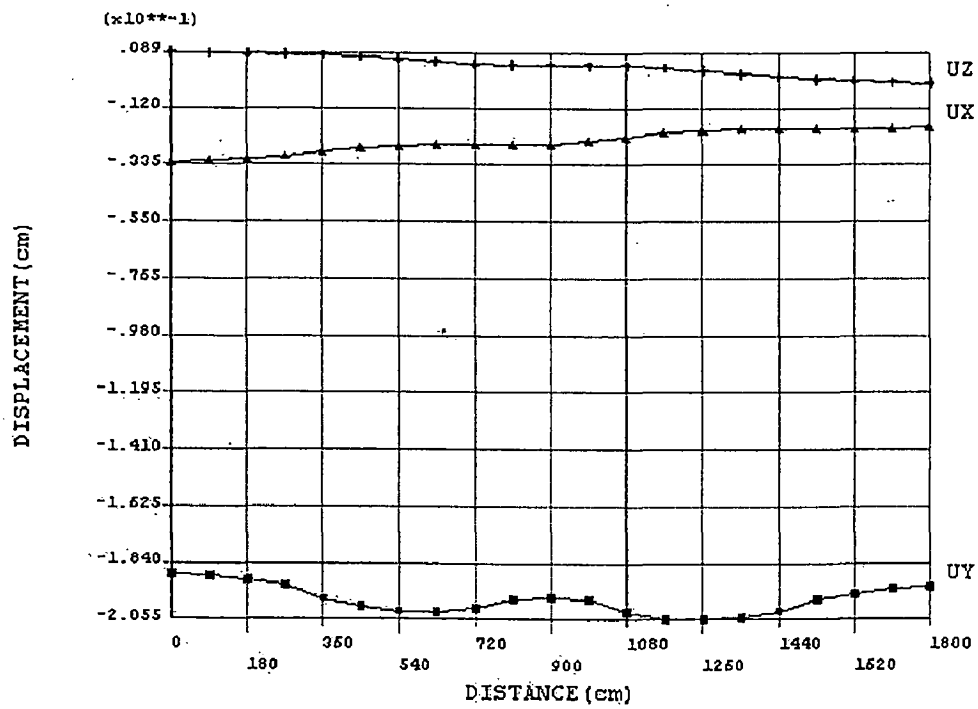


Figure 4.14 Displacement at the top of the draft tube openings

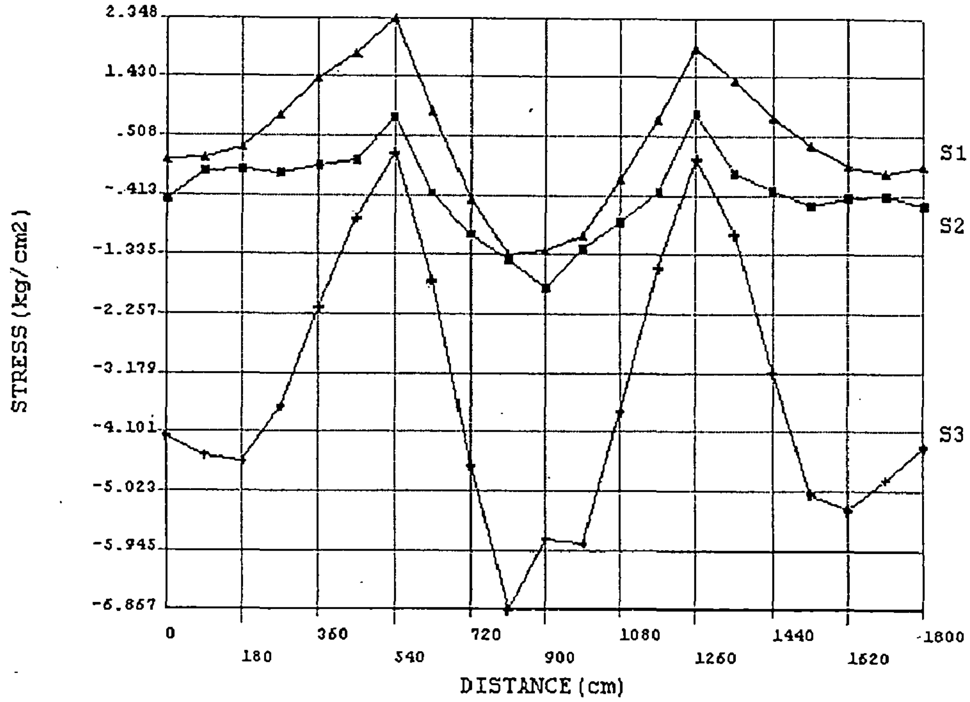


Figure 4.15 Principal Stress at the bottom of the draft tube openings

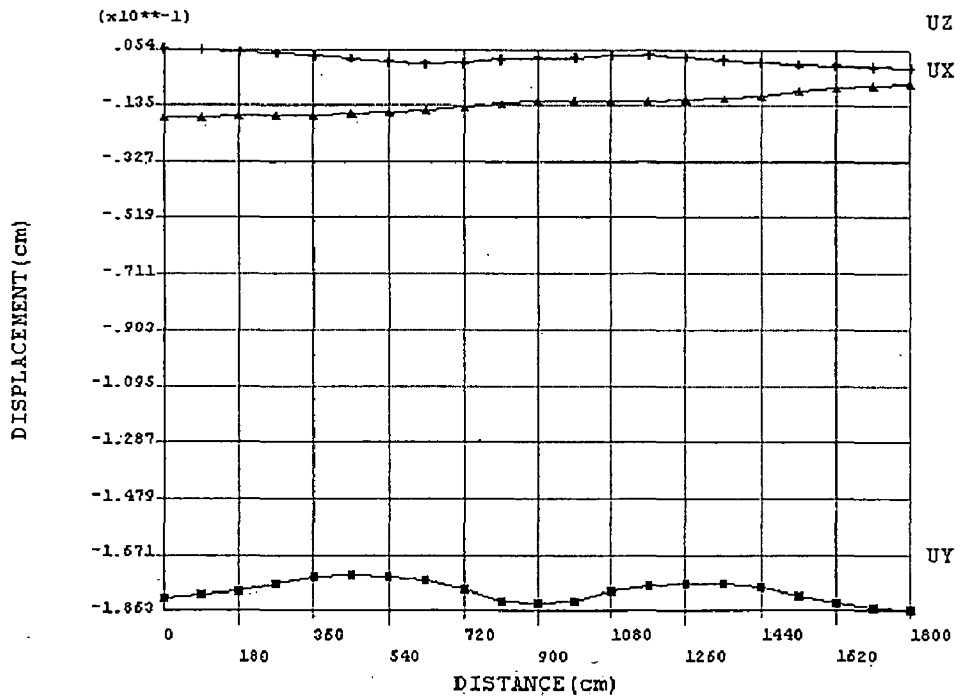


Figure 4.16 Displacement at the bottom of the draft tube openings

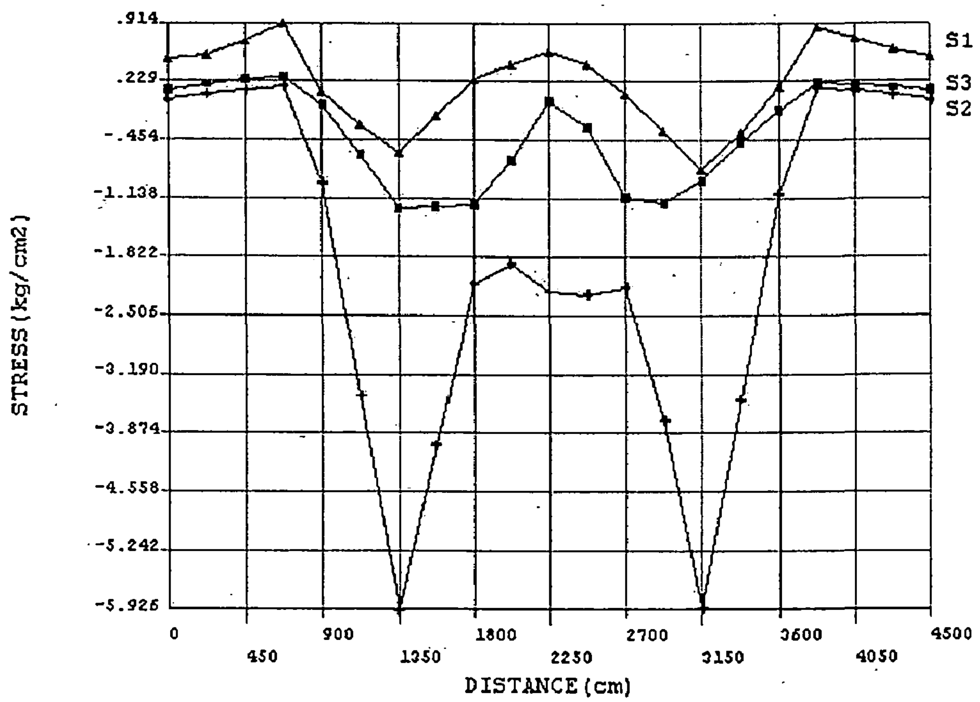


Figure 4.17 Principal Stress at the foundation contact

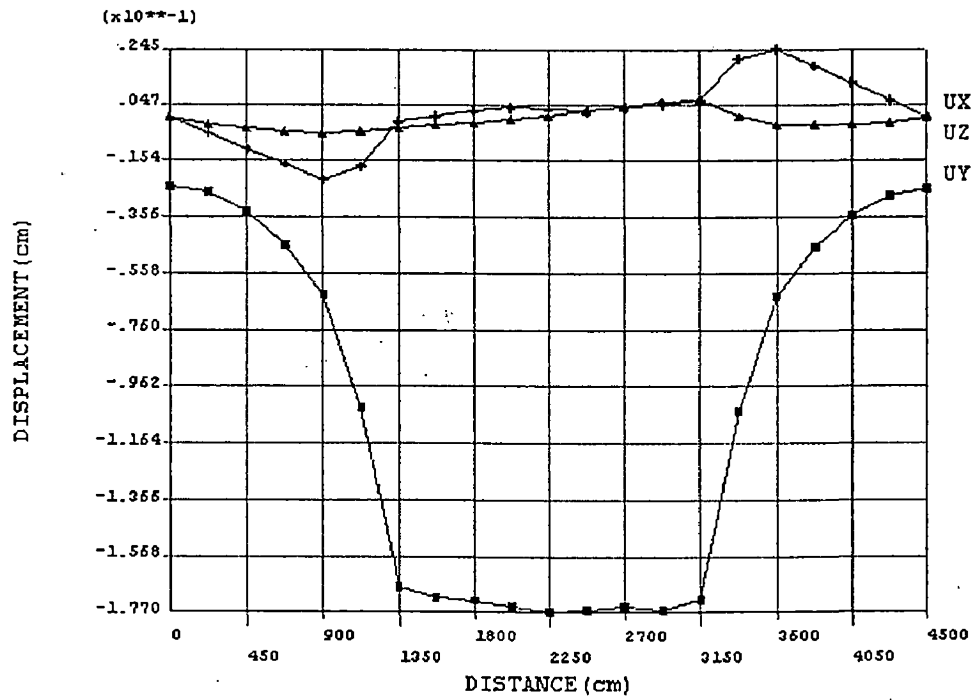


Figure 4.18 Displacement at the foundation contact

(D) U/S SECTION AT THE DRAFT TUBE(E=Ec/100)

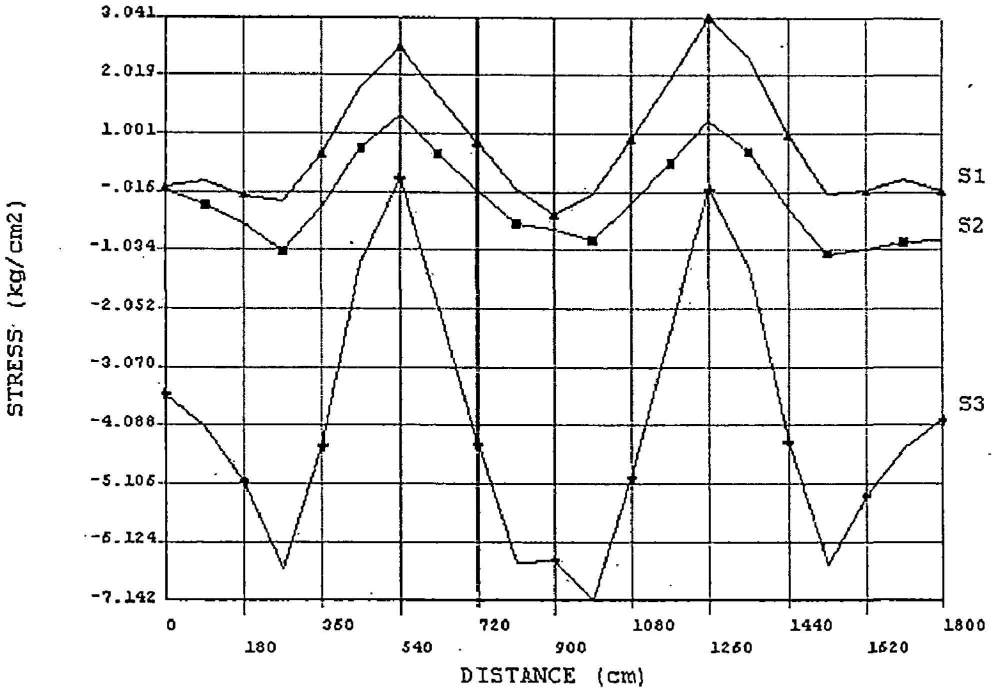


Figure 4.19 Principal Stress at the top of the draft tube openings

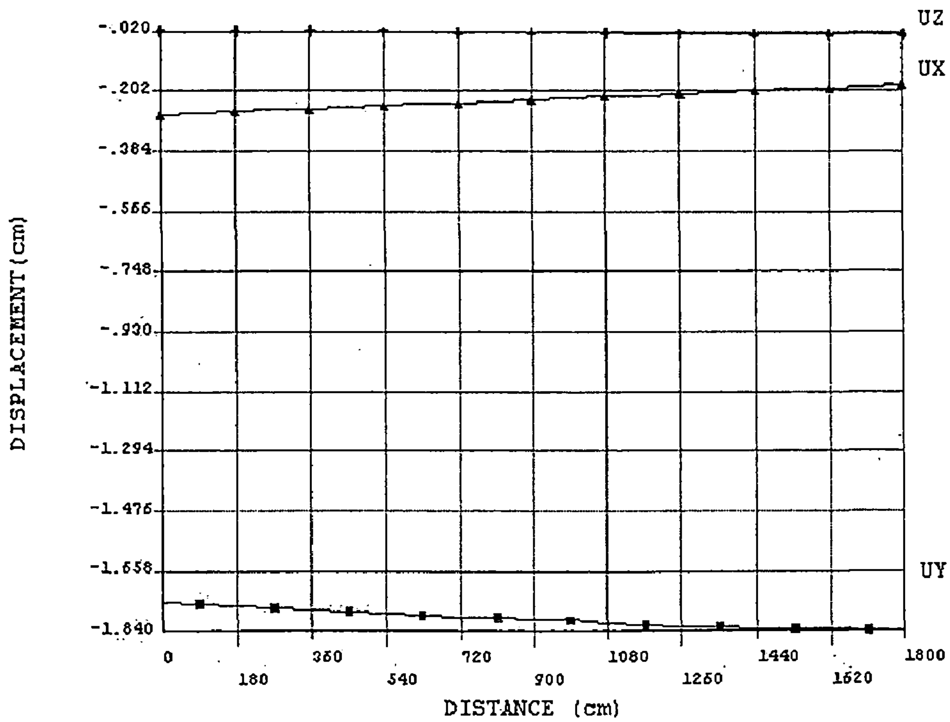


Figure 4.20 Displacement at the top of the draft tube openings

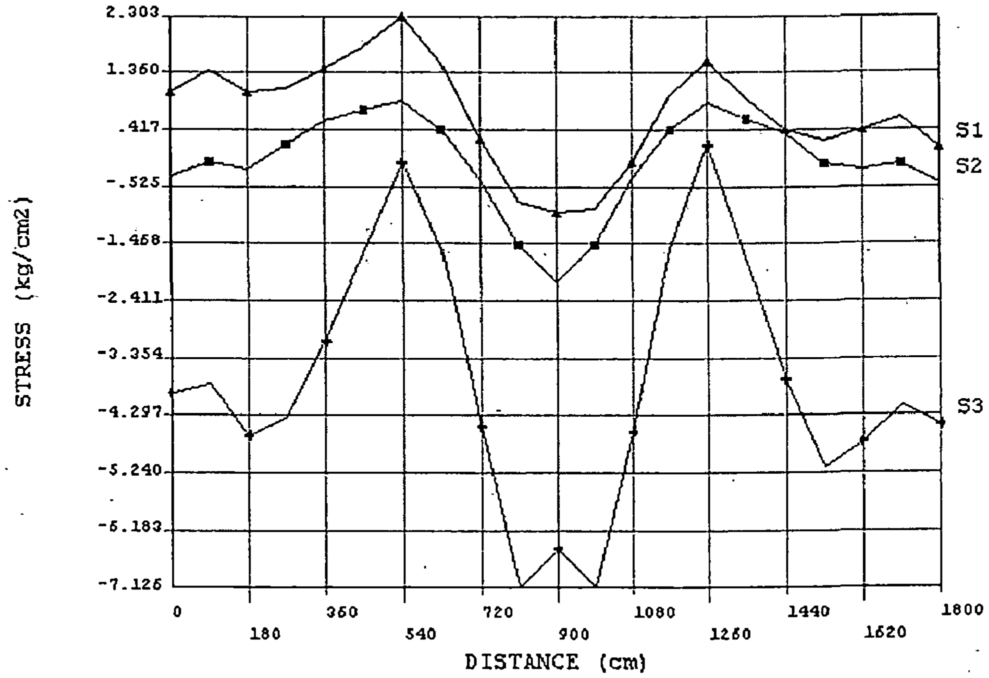


Figure 4.21 Principal Stress at the bottom of the draft tube openings

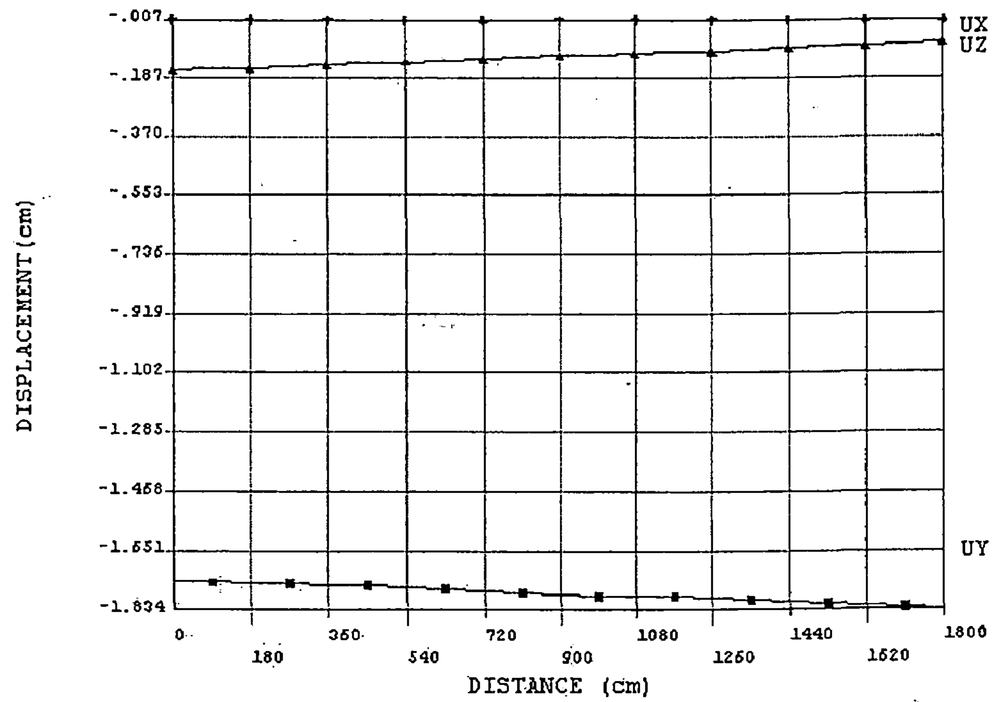


Figure 4.22 Displacement at the bottom of the draft tube openings

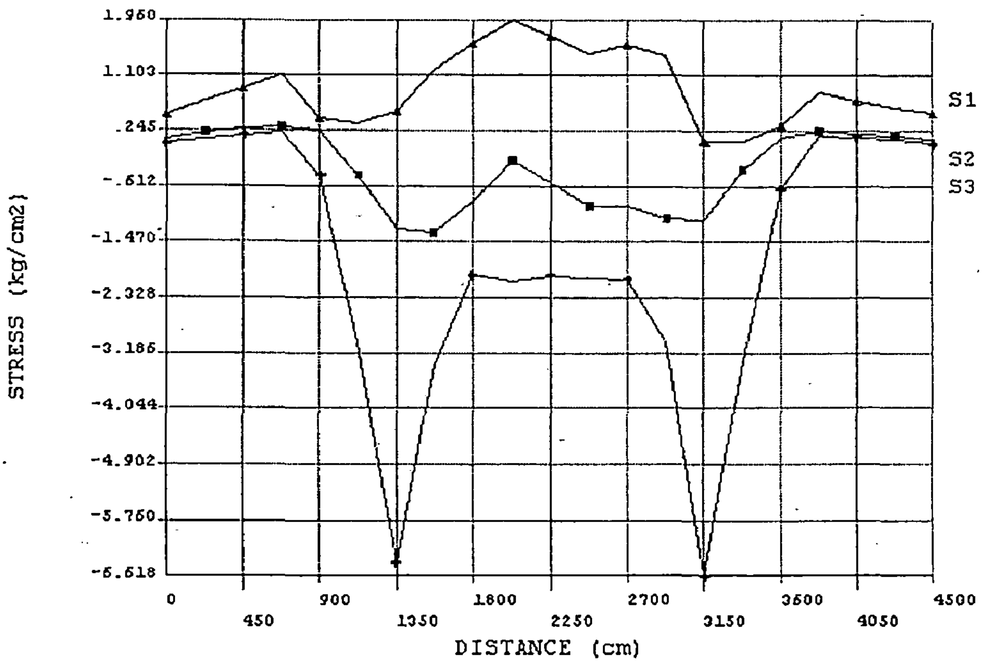


Figure 4.23 Principal Stress at the foundation contact

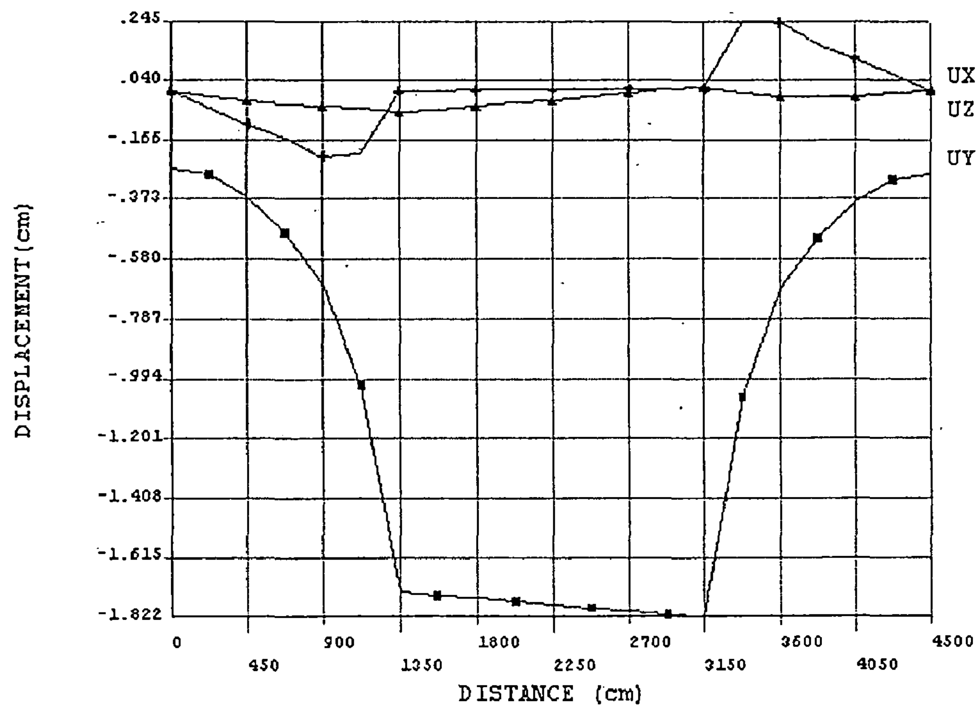


Figure 4.24 Displacement at the foundation contact

(D)SUB STRUCTURE FIXED AT THE BASE

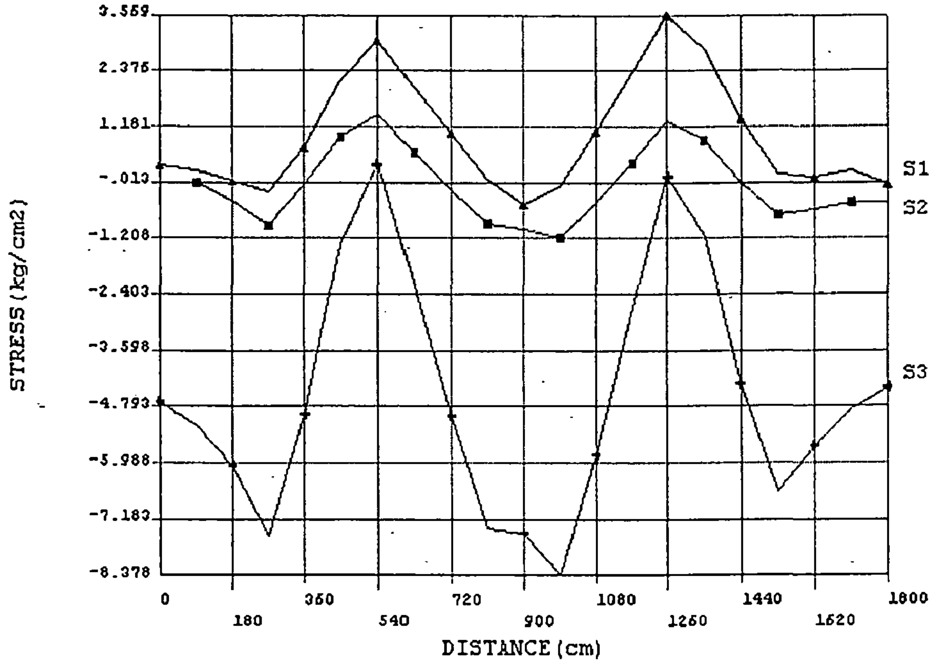


Figure 4.25 Principal Stress at the top of the draft tube openings

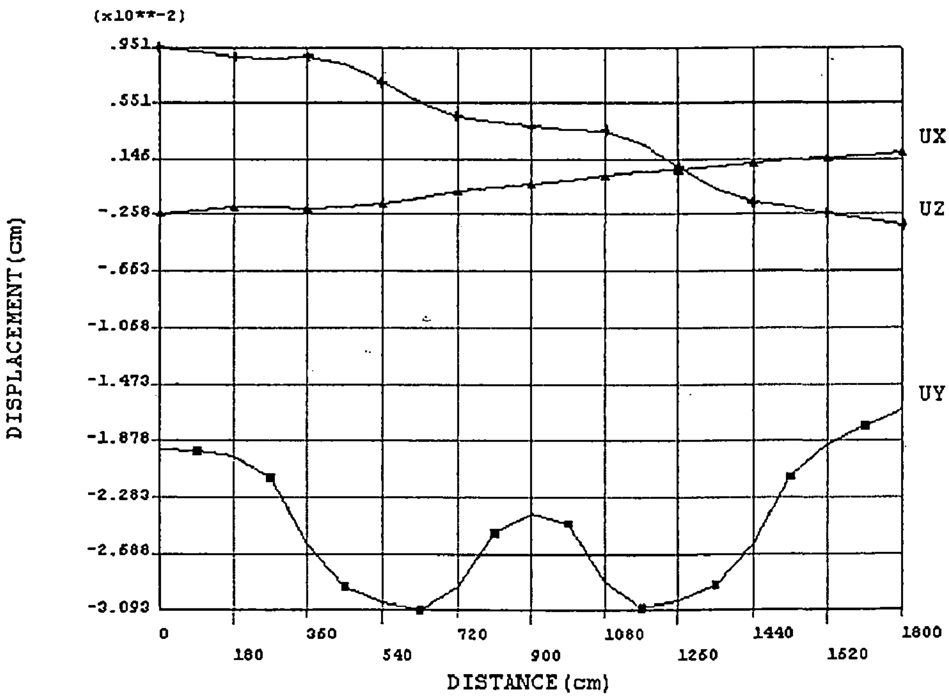


Figure 4.26 Displacement at the top of the draft tube openings

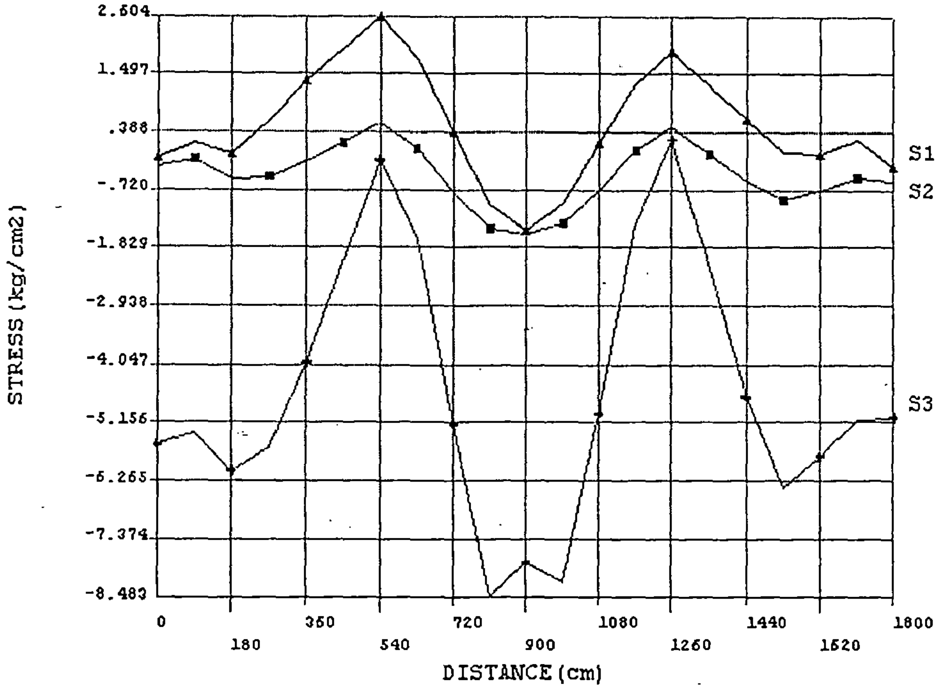


Figure 4.27 Principal Stress at the bottom of the draft tube openings

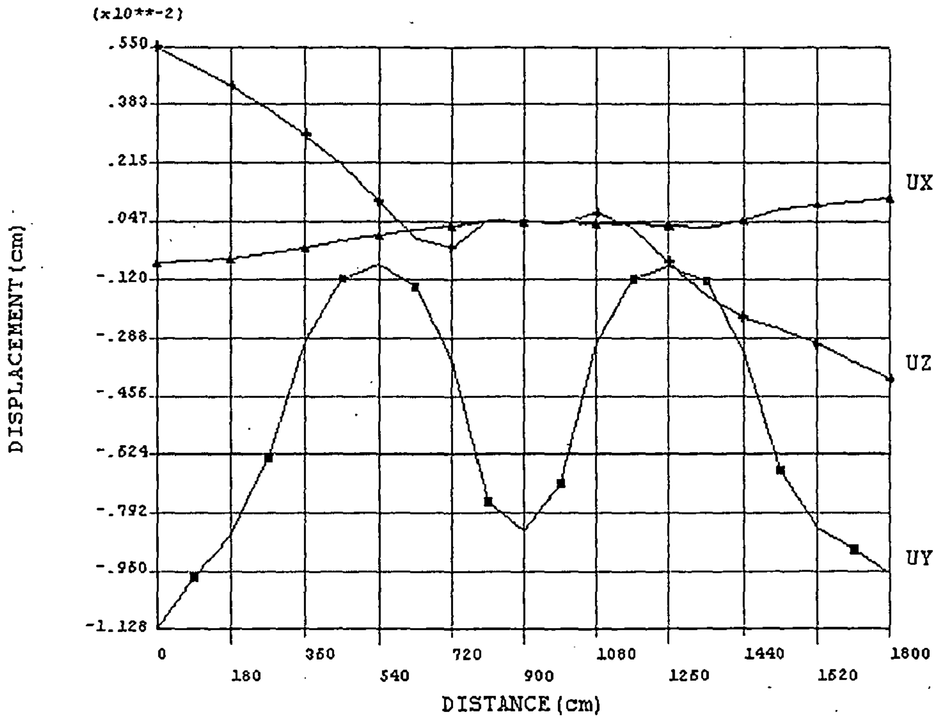


Figure 4.28 Displacement at the bottom of the draft tube openings

4.2.2 D/S Section (1200cm from the turbine axis)

4.2.2.1 At the top of the draft tube openings

Maximum tensile stress, S1, occurred at the openings at 630 cm. from each edge of the section. Its value varied from 1.70 kg/cm2 when foundation condition is E=10*Ec and 2.18 kg/cm^2, when foundation condition is E=Ec/100, values of the order of 1. 71 and 1.87kg/cm^2 have been observed for the foundation condition E=Ec and E=Ec/10 respectively (Figures-4.29, 4.35, 4.41 4.47). There has been observed increasing trend in the values with the decrease of rigidity of the foundation. The values are also tabulated in the Table-4.1

Maximum compressive stress, S3, was observed above the pier and it varied from 5.02 kg/cm^2 for foundation condition, E=10* Ec to 7.31 kg/cm^2 for the foundation condition, E=Ec/100. Other values were 5.04, 6.48 kg/cm^2 were observed for the foundation condition E=Ec and E= Ec/10 respectively. Which has shown an increase in the stress with the decrease in the rigidity of the foundation As shown in the Figures -4.29,4,35,4,41 4,47, and in the Table-4.1.

Maximum vertical displacement occurred at the openings, 540 cm from the both the edges of the section. The values of the order of 0.03, 0.05, 0.2, 1.72 cm were observed for the foundation conditions, E=10*Ec, Ec, E/10 and E/100 respectively (Figures-4.30, 4.36, 4.42, 4.48).Displacement increased with the decrease in the rigidity of the foundation as shown in the table-4.1

4.2.2.2 At the bottom of the draft tube openings

Maximum tensile stress, S1, occurred at 540 cm from the left edge of the section. Its value varied from 2.80 kg/cm2 in the foundation condition, E=10*Ec to 2.14 kg/cm^2 in the foundation condition, E=Ec/100. The values of the order of 2.61kg/cm^2 and 2.24 kg/cm^2 were observed for the foundation condition E=Ec and E=Ec/10 respectively. (Figures-4.31, 4.37, 4.43, 4.49).The values of the stress increases with the decrease in the rigidity of the foundation. As shown in the table-4.1

Maximum compressive stress occured below the pier of the Draft tube. The values vary from 7.31 kg/cm^2 in the foundation condition, E=10*Ec to 6.48 kg/cm^2, in the foundation condition=Ec/100.Other observed values were 7.21 and 6.87 kg/cm^2 for the foundation condition, E=Ec and E=Ec/10 respectively. The values of the compressive stress have decreased with the decrease in the rigidity of the foundation. As shown in the Table-4.1.

Maximum vertical displacement occured at the left edge of the section. The values were of the order of 0.01, 0.03, 0.18and 1.71cm for the foundation condition E=10*Ec, Ec, Ec/10 and Ec/100 respectively as shown in the Figures-4.32, 4.38., 4.44,4,50 The displacement in creases with the decrease in the rigidity of the foundation as shown in the table-4.1.

4.2.2.3 At the foundation contact

Maximum tensile stress were observed in the foundation zone it values were ot the order of 0.98,0.96,0.91kg/cm^2 for the foundation condition E=10*Ec, Ec and Ec/10 For the foundation condition E=Ec/100. The position of the tensile stress changes and had a value of 1.83 kg/cm2 at the center of the section (Figures-4.33,4.39,4.45,4.51). The maximum tensile stress occurred at 675 cm from the either side if the edges of the foundation. for the first three conditions of the foundation and maximum at the center of the structure in the foundation condition E=Ec/100.

Maximum compressive stress has been experienced at the left edge of the structure. The values were of the order of 5.18, 5, 63, and 5.93, 5.82 Kg

/cm2 for the foundation condition E=10*Ec, Ec, Ec/10 and Ec/100 respectively (Figures-4.33,4.39,4.45,4.51)

Maximum vertical displacement has increased with decrease in the rigidity of the foundation as shown in the Figures-4.34, 4.40, 4.46, 4.52. the values are tabulated in the table-4.1

4.2.2.4 Substructure fixed at the base

The values of stresses and displacements resembled with the case when foundation condition is, E=10*Ec as shown in the Figures-4.53, 4.54, 4.55 and 4.56 and in the Table 4.1.

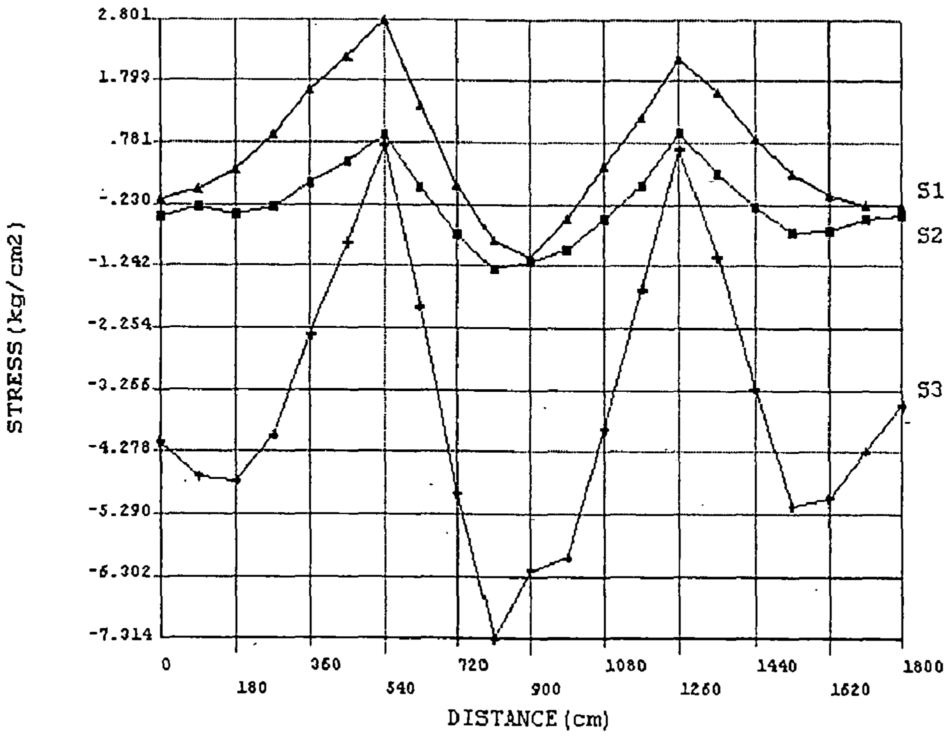


Figure 4.31 Principal Stress at the bottom of the draft tube openings

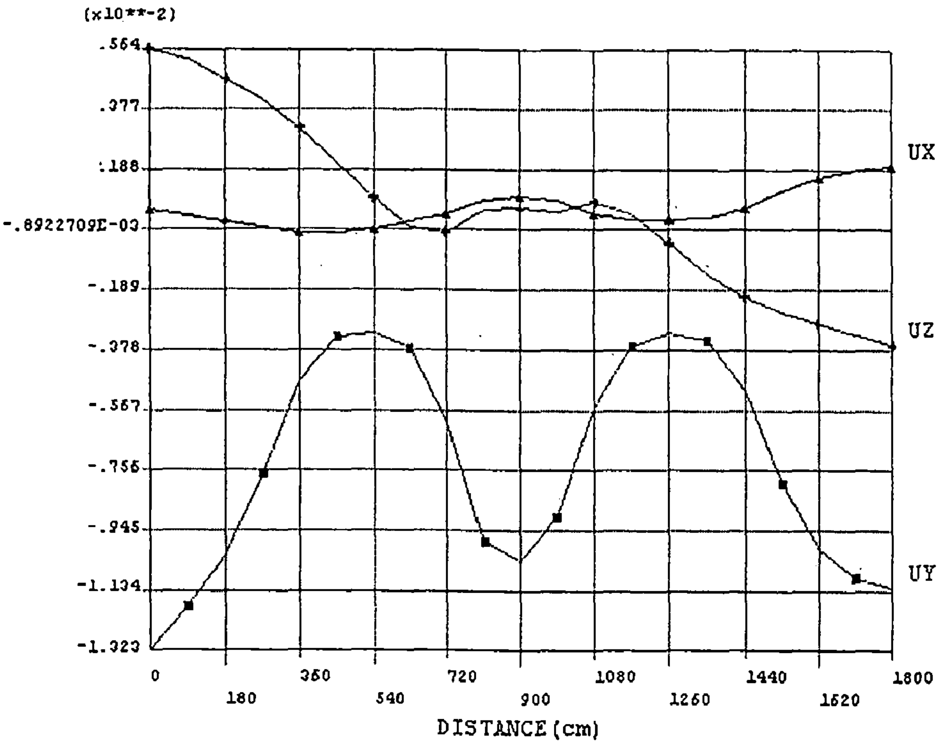


Figure 4.32 Displacement at the bottom of the draft tube openings

LOAD CASE-A
(A) D/S SECTION AT THE DRAFT TUBE (E=10*Ec)

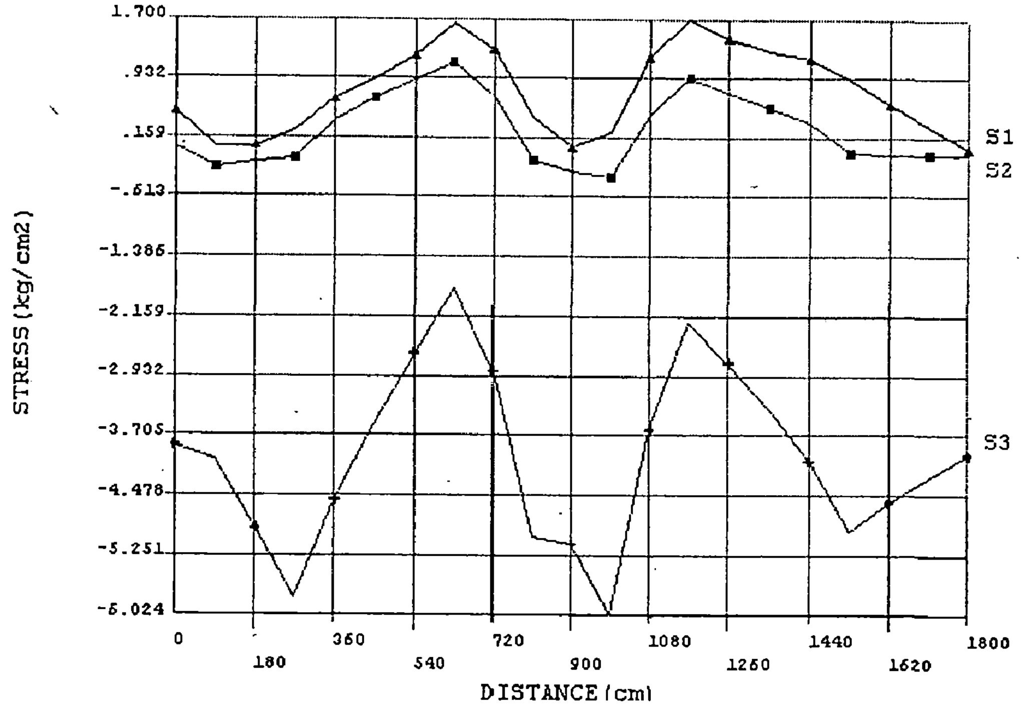


Figure 4.29 Principal Stress at the top of the draft tube openings

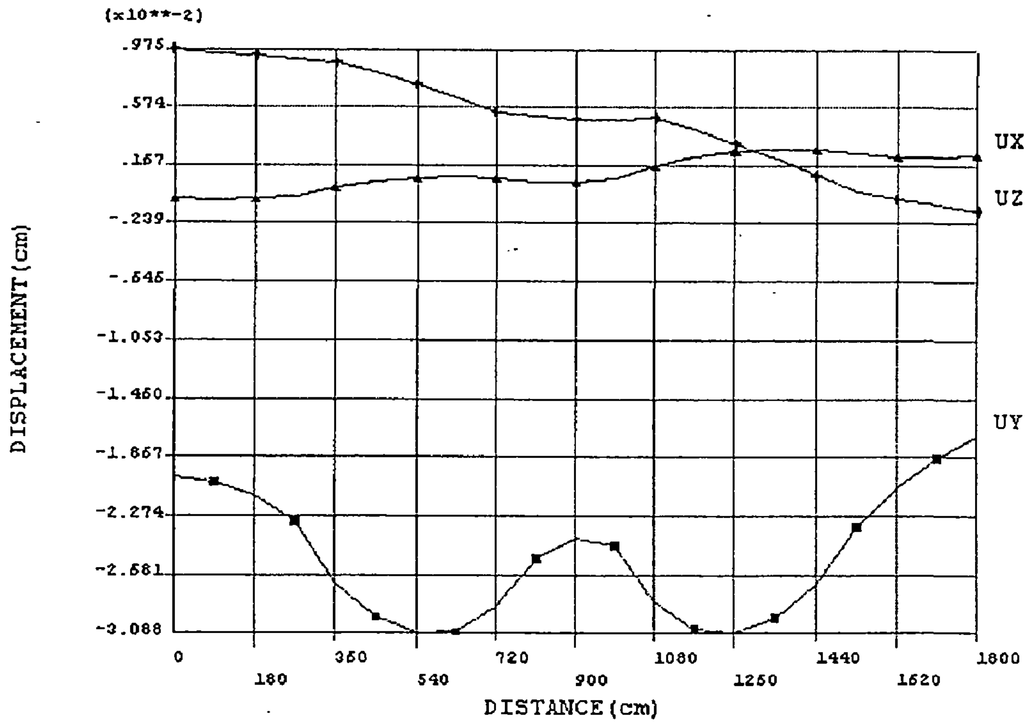


Figure 4.30 Displacement at the top of the draft tube openings

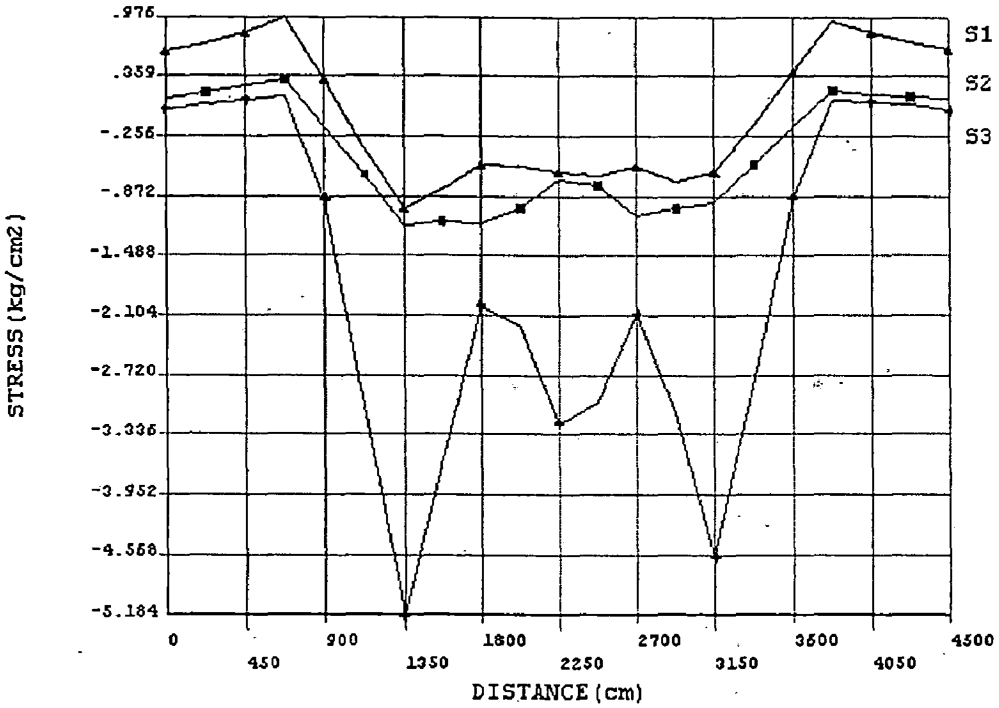


Figure 4.33 Principal Stress at the foundation contact

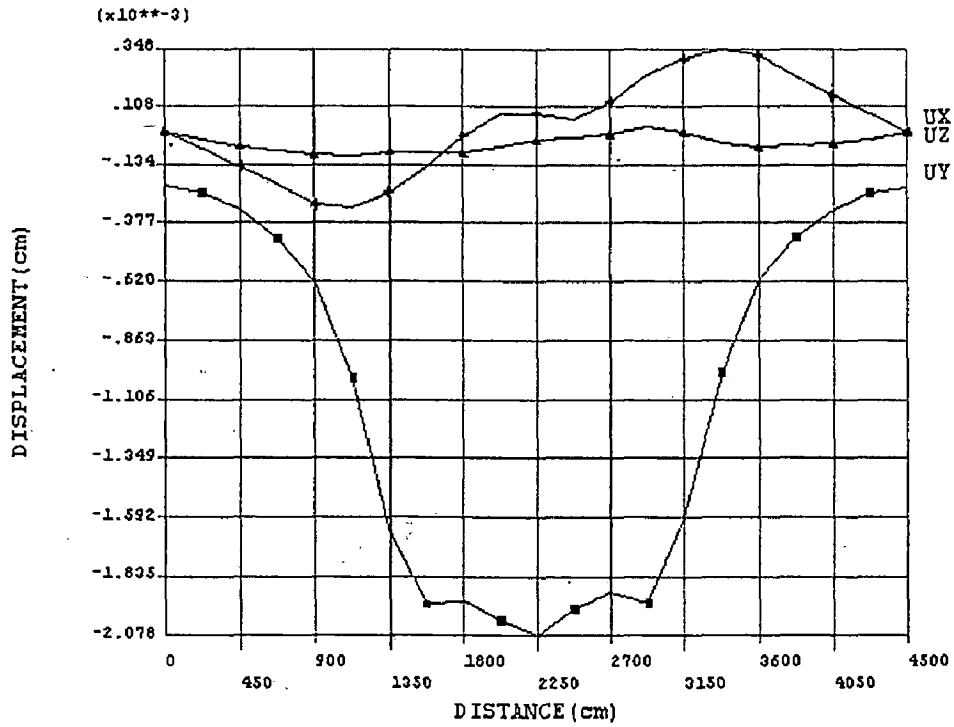


Figure 4.34 Displacement at the foundation contact

(B) D/S SECTION AT THE DRAFT TUBE (E = Ec)

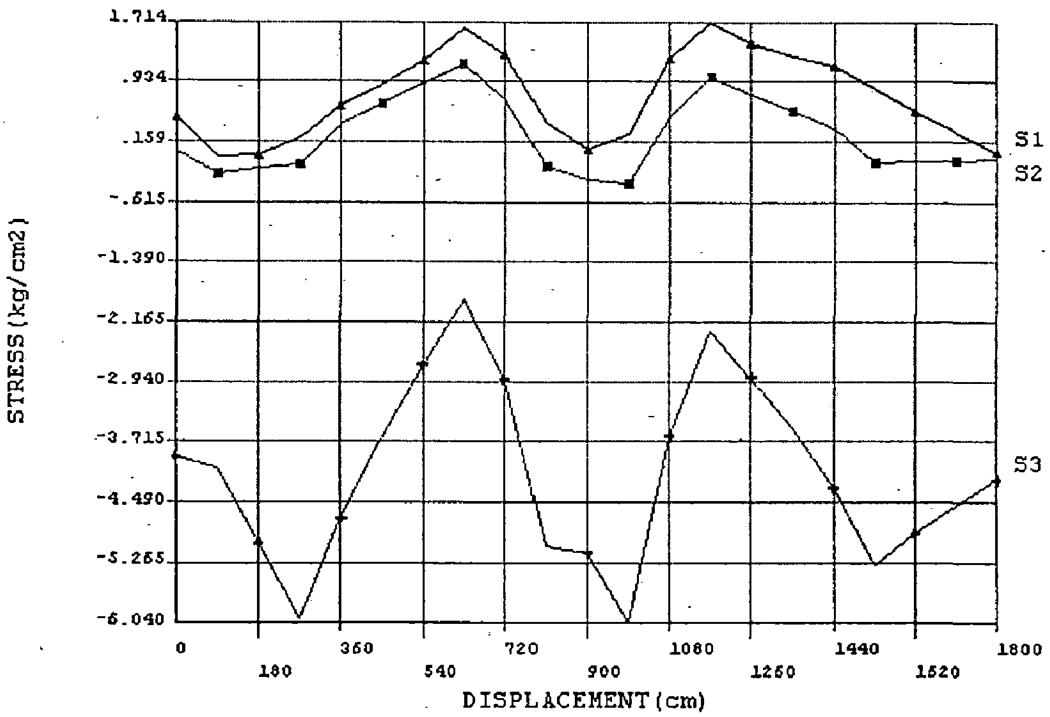


Figure 4.35 Principal Stress at the top of the draft tube openings

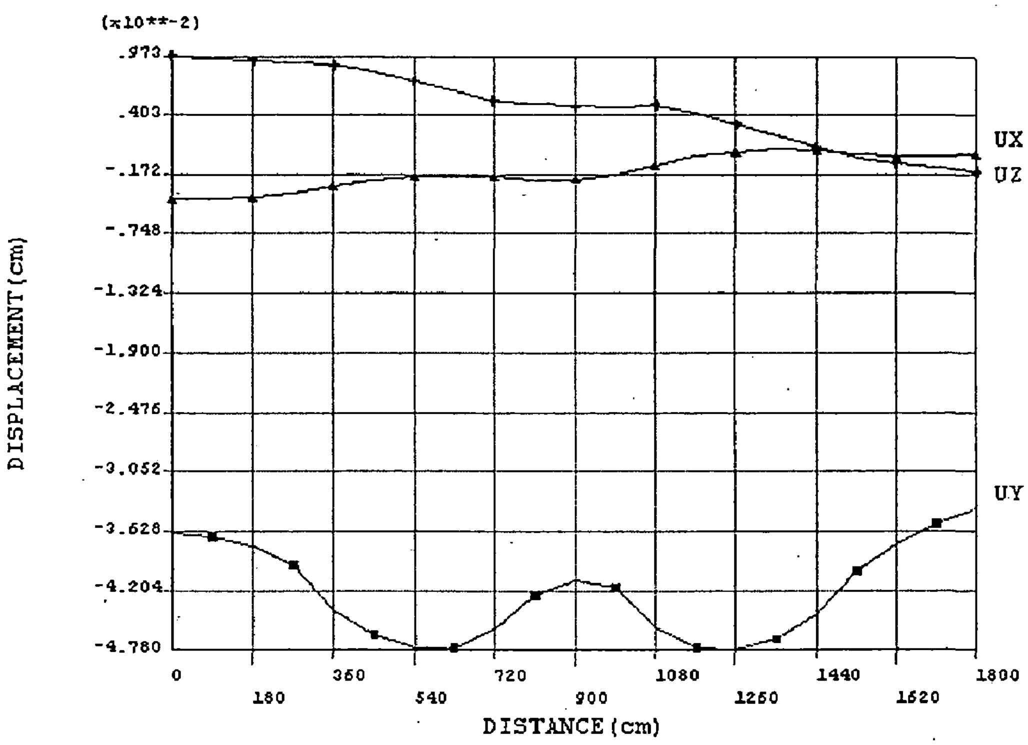


Figure 4.36 Displacement at the top of the draft tube openings

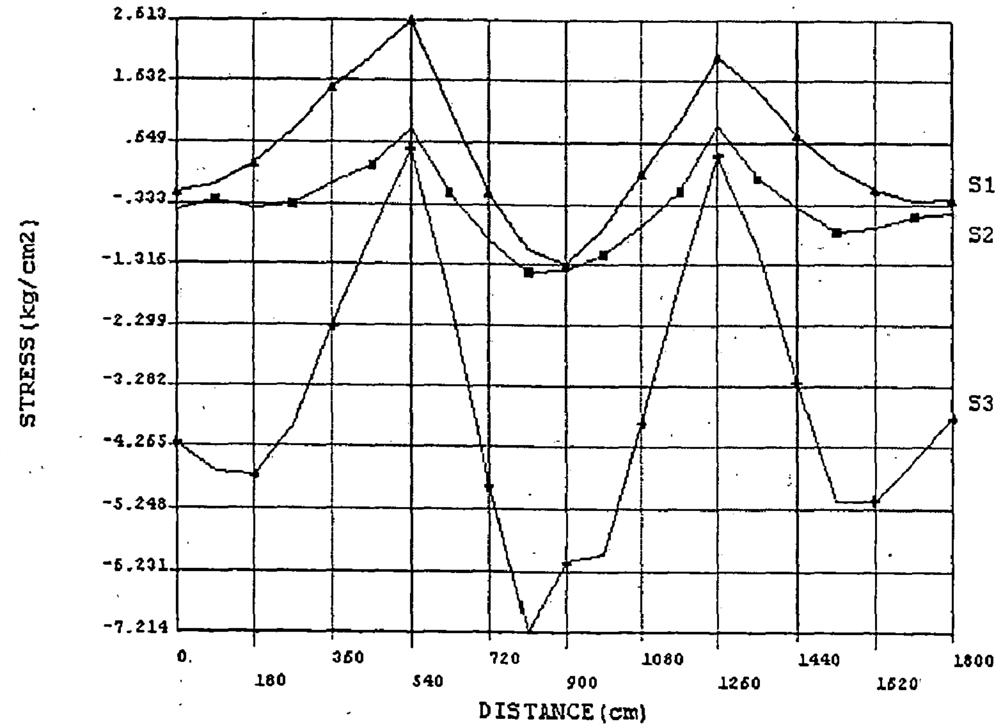


Figure 4.37 Principal Stress at the bottom of the draft tube openings

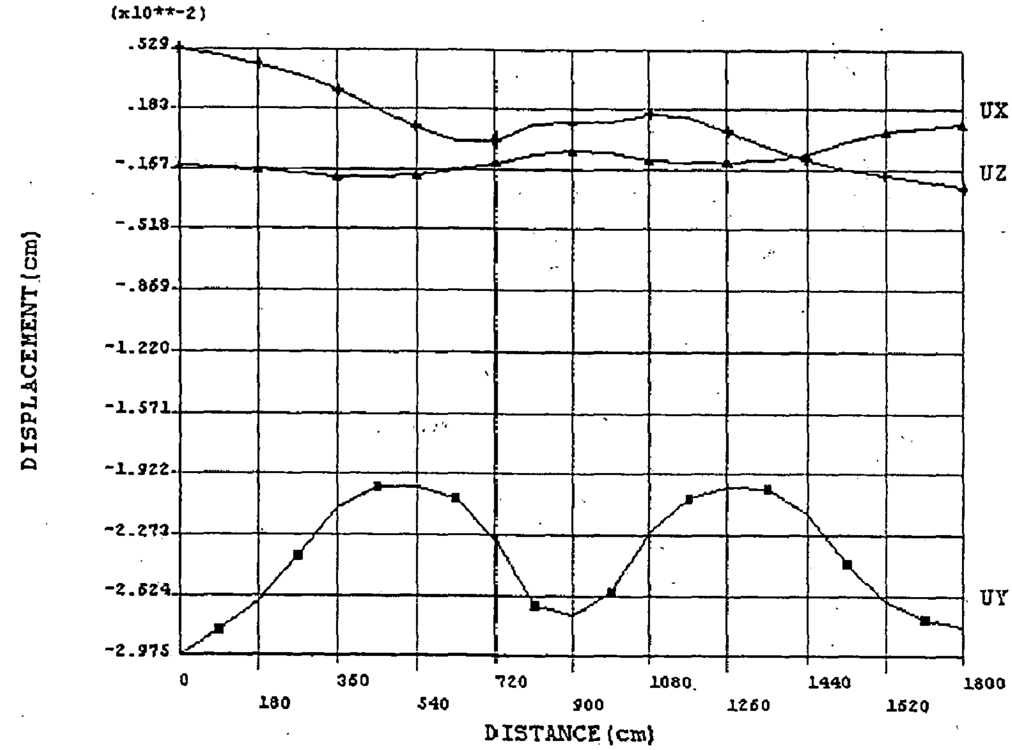


Figure 4.38 Displacement at the bottom of the draft tube openings

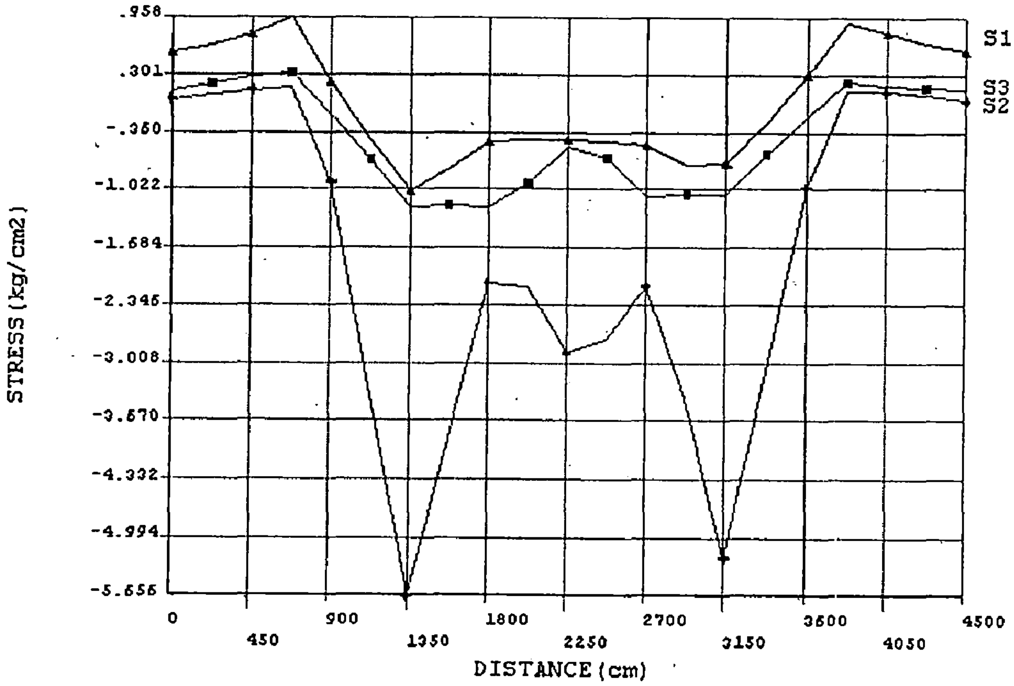


Figure 4.39 Principal Stress at the foundation contact

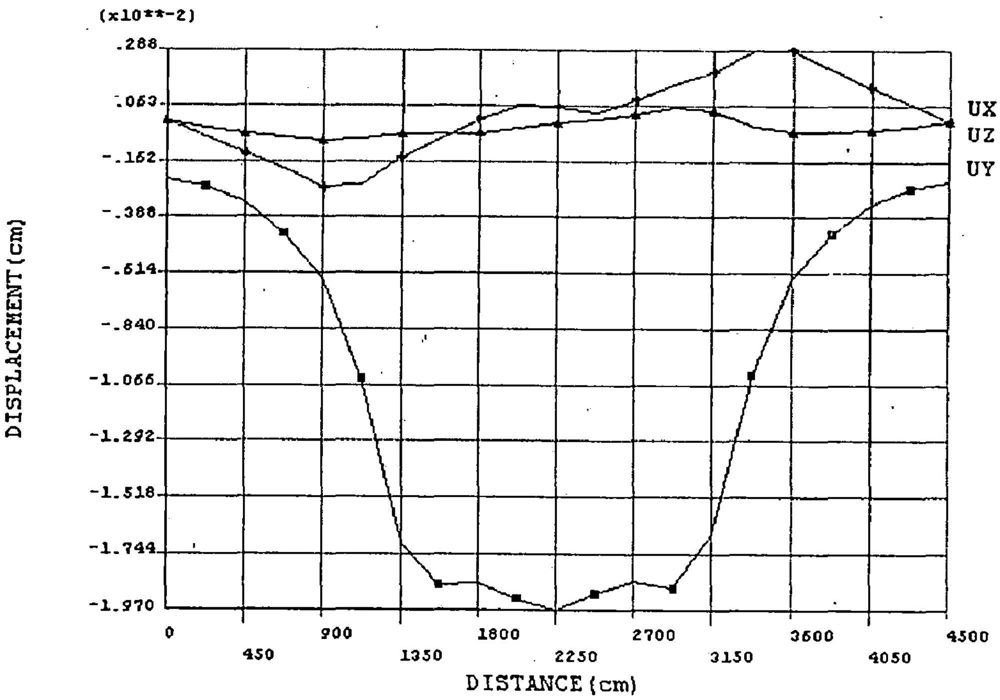


Figure 4.40 Displacement at the foundation contact

(C) D/S SECTION AT THE DRAFT TUBE (E = Ec/10)

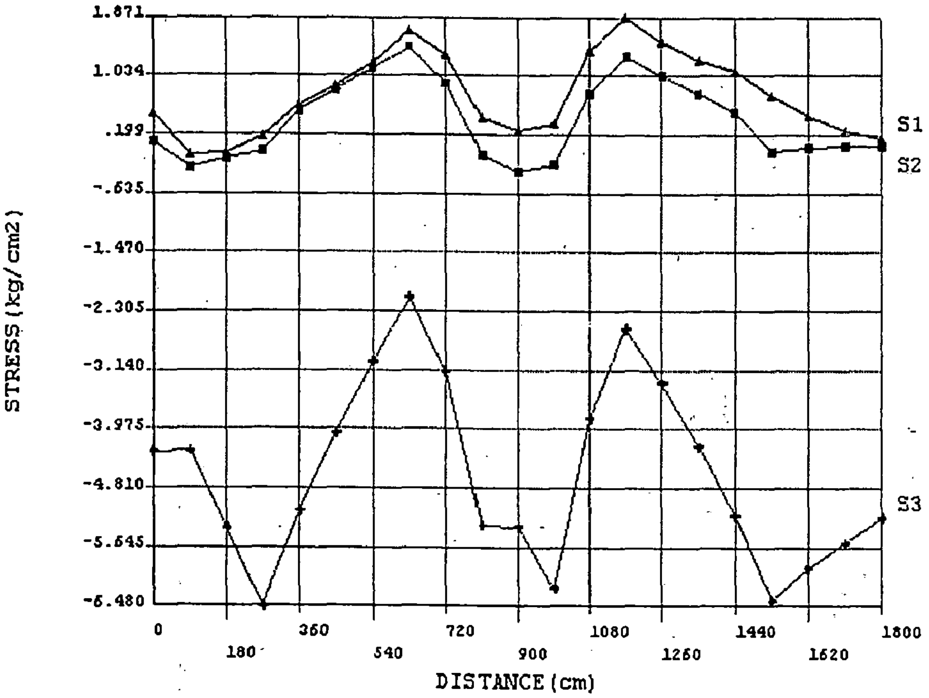


Figure 4.41 Principal Stress at the top of the draft tube openings

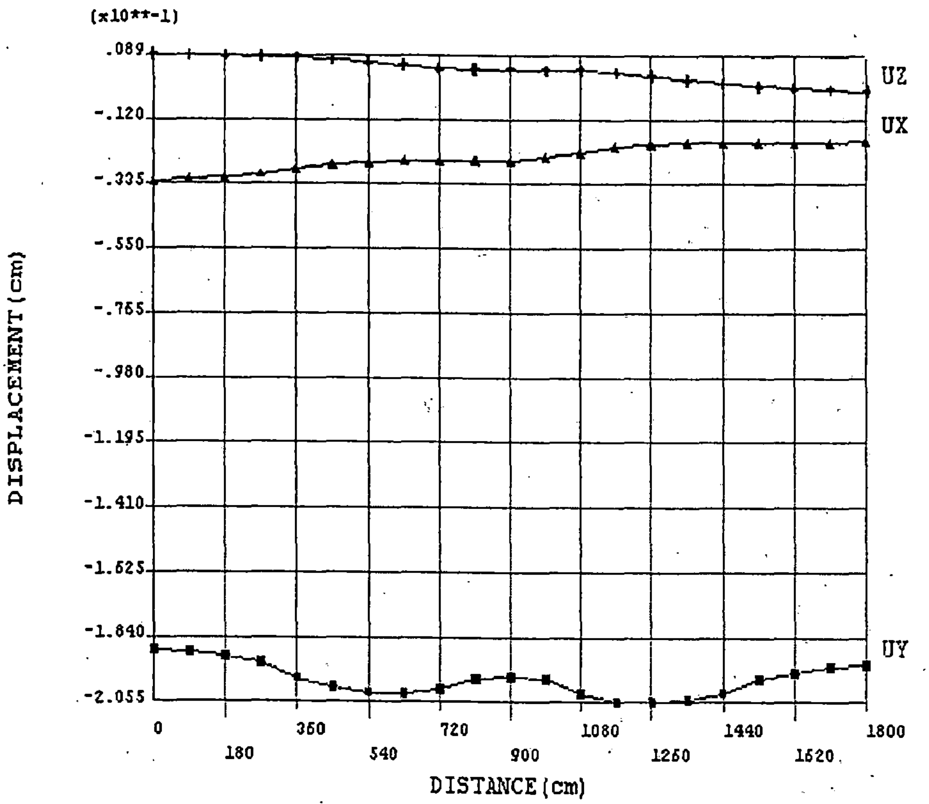


Figure 4.42 Displacement at the top of the draft tube openings

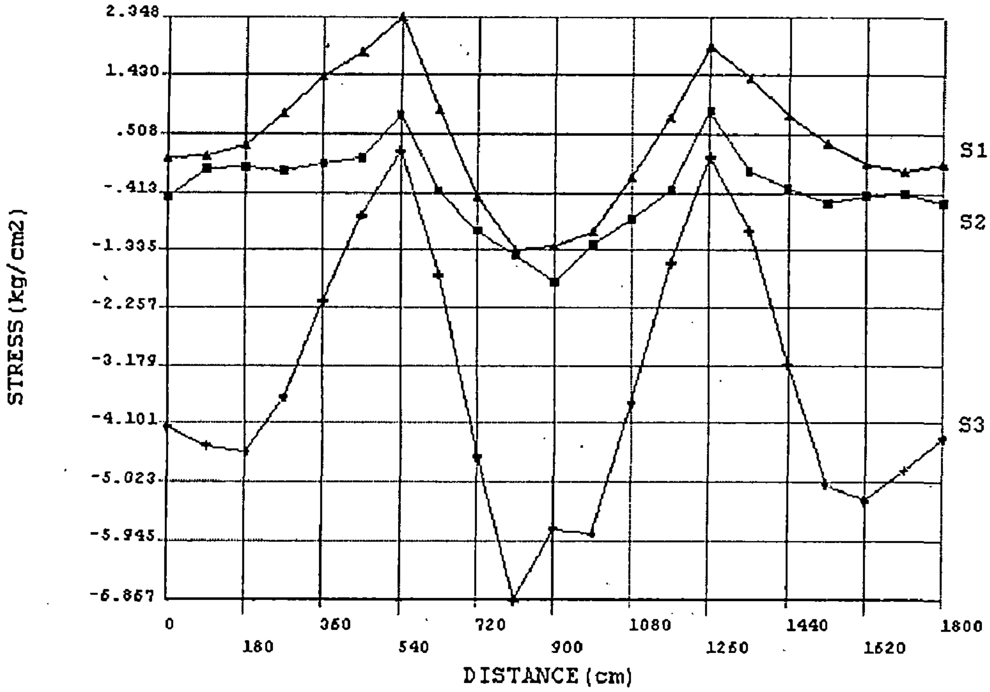


Figure 4.43 Principal Stress at the bottom of the draft tube openings

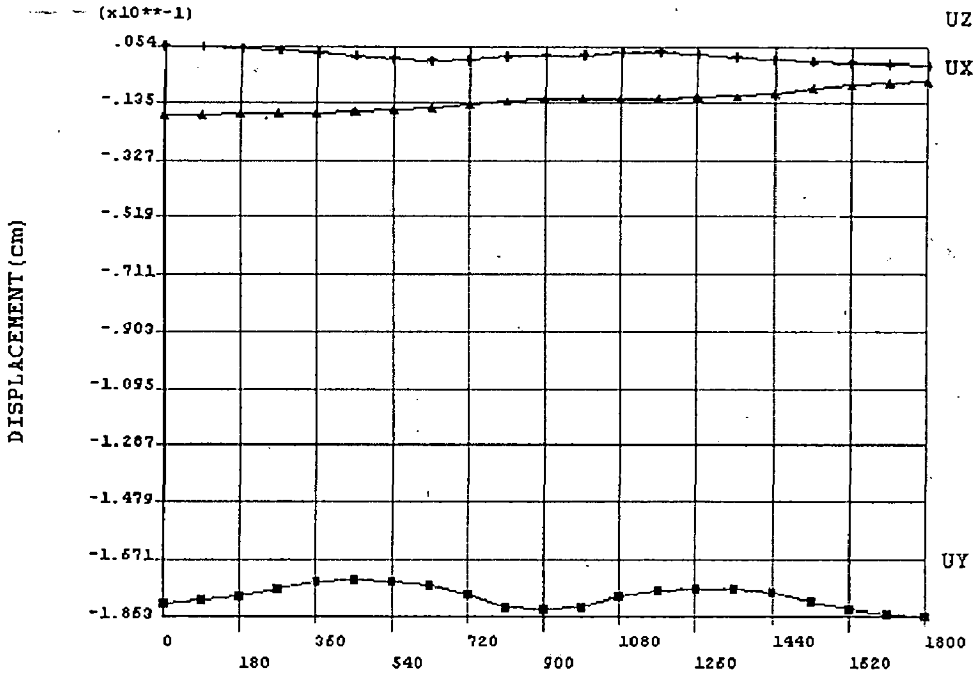


Figure 4.44 Displacement at the bottom of the draft tube openings

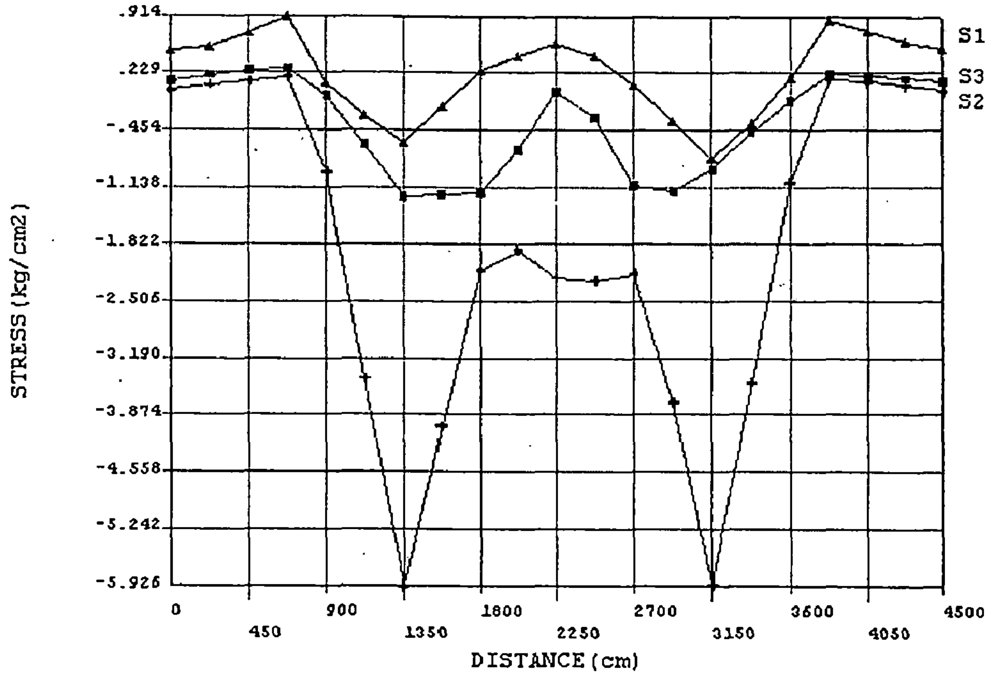


Figure 4.45 Principal Stress at the foundation contact

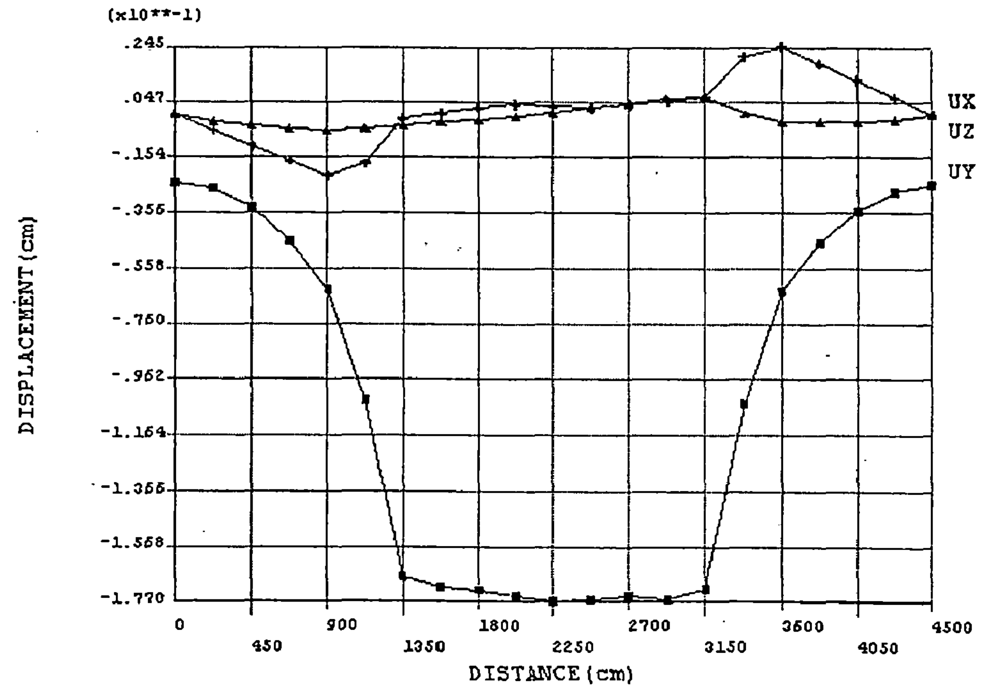


Figure 4.46 Displacement at the foundation contact

(D) D/S SECTION AT THE DRAFT TUBE (E = Ec/100)

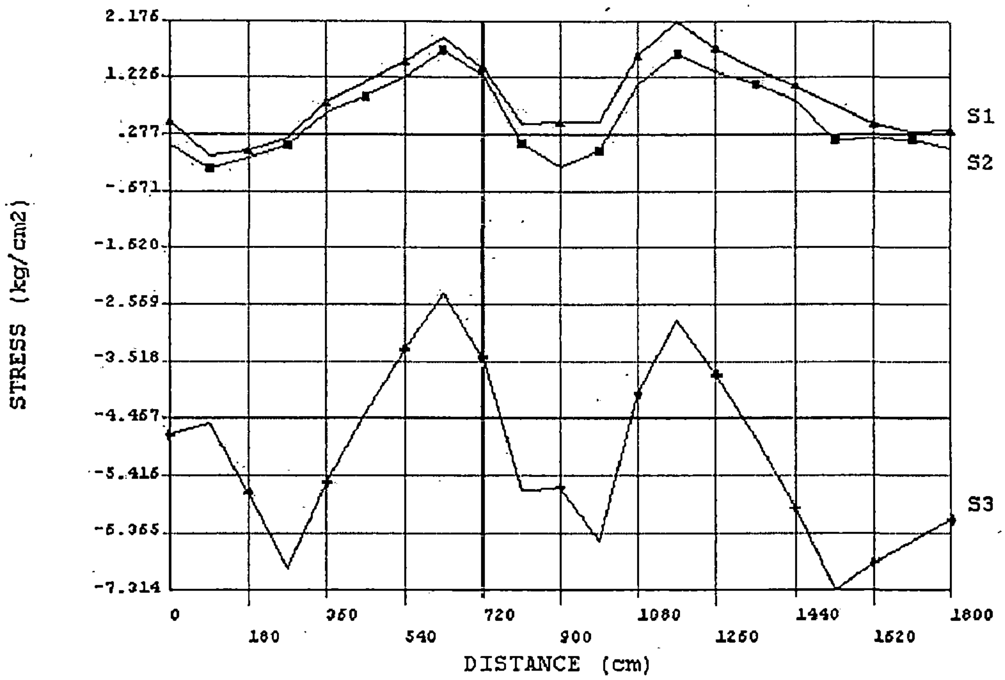


Figure 4.47 Principal Stress at the top of the draft tube openings

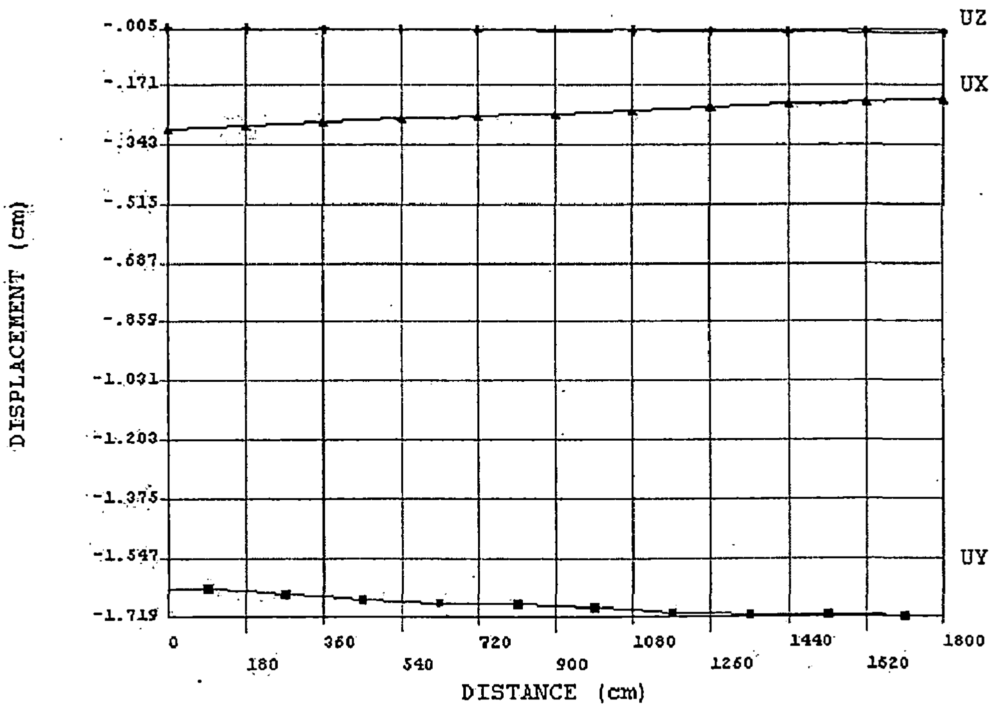


Figure 4.48 Displacement at the top of the draft tube openings

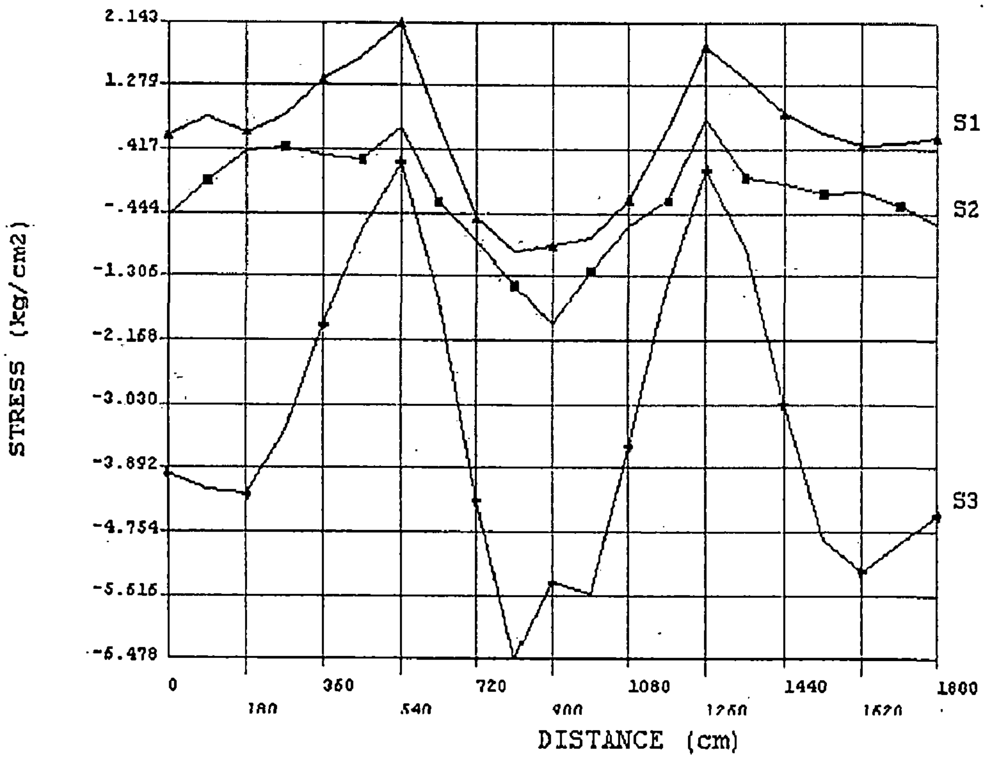


Figure 4.49 Principal Stress at the bottom of the draft tube openings

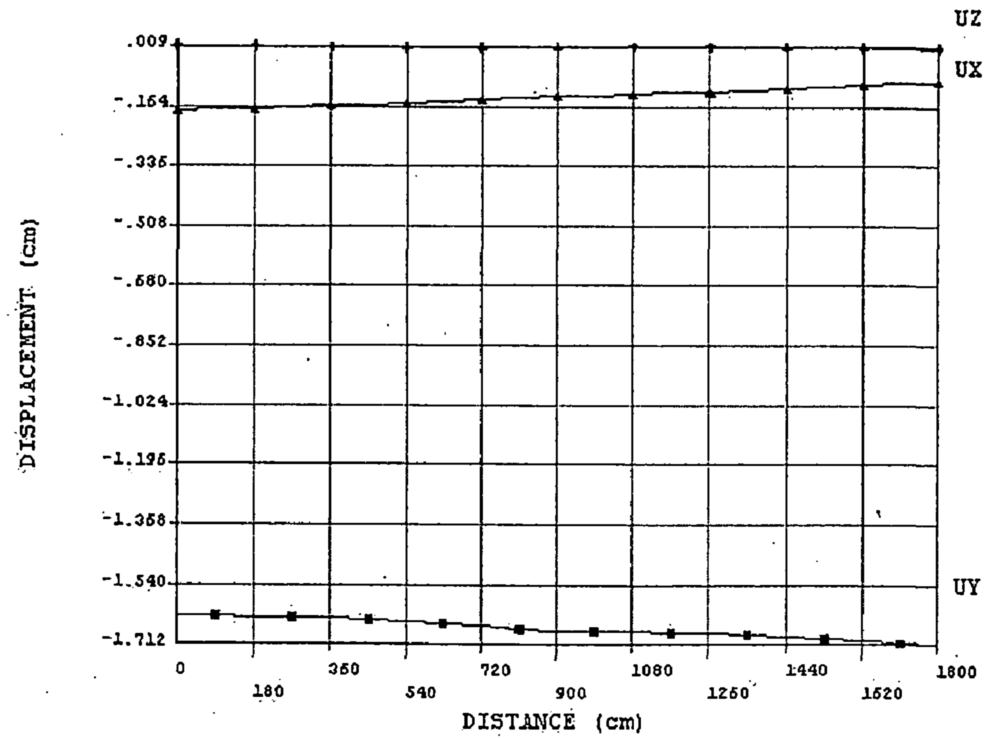


Figure 4.50 Displacement at the bottom of the draft tube openings

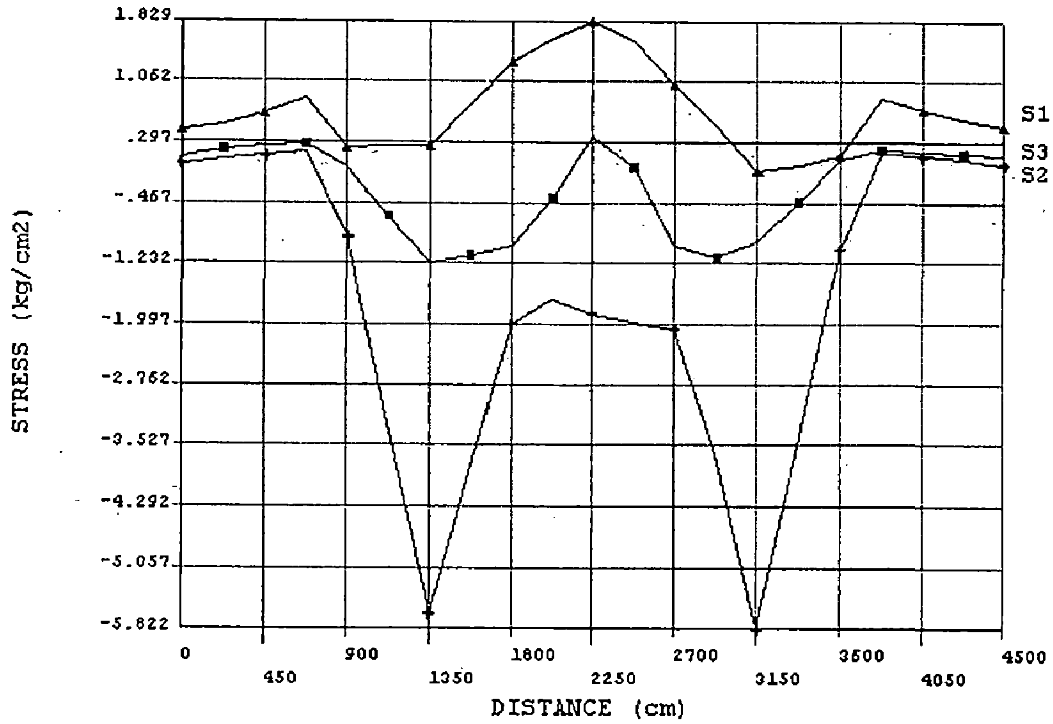


Figure 4.51 Principal Stress at the foundation contact

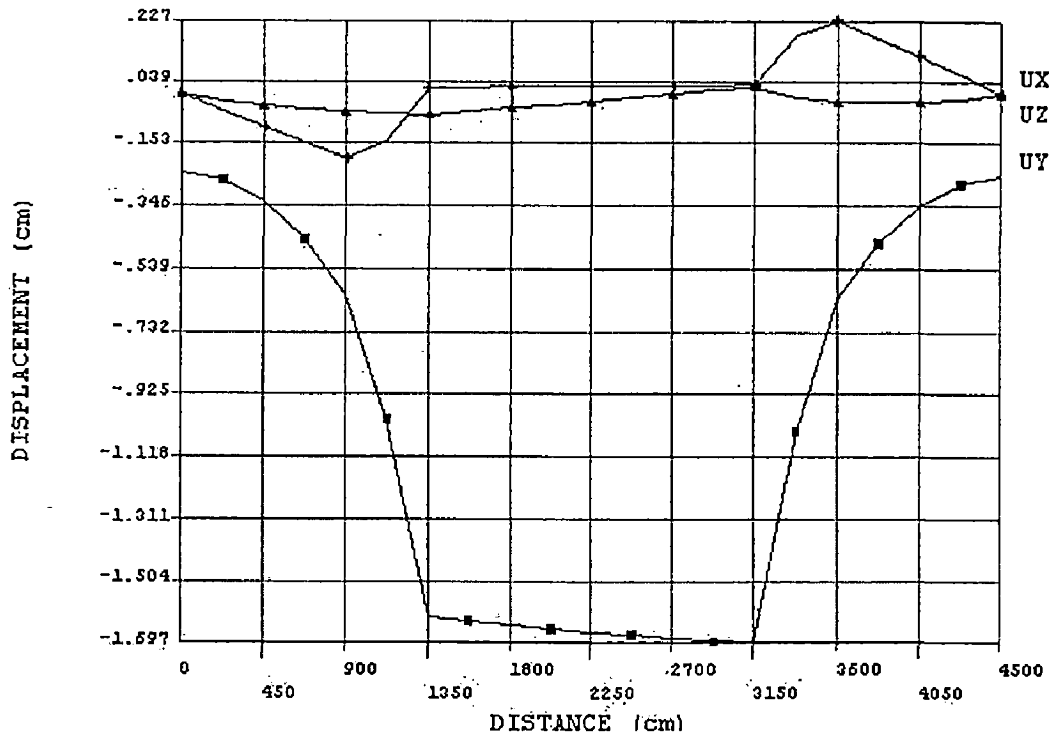


Figure 4.52 Displacement at the foundation contact

(E) SUB STRUCTURE FIXED AT THE BASE

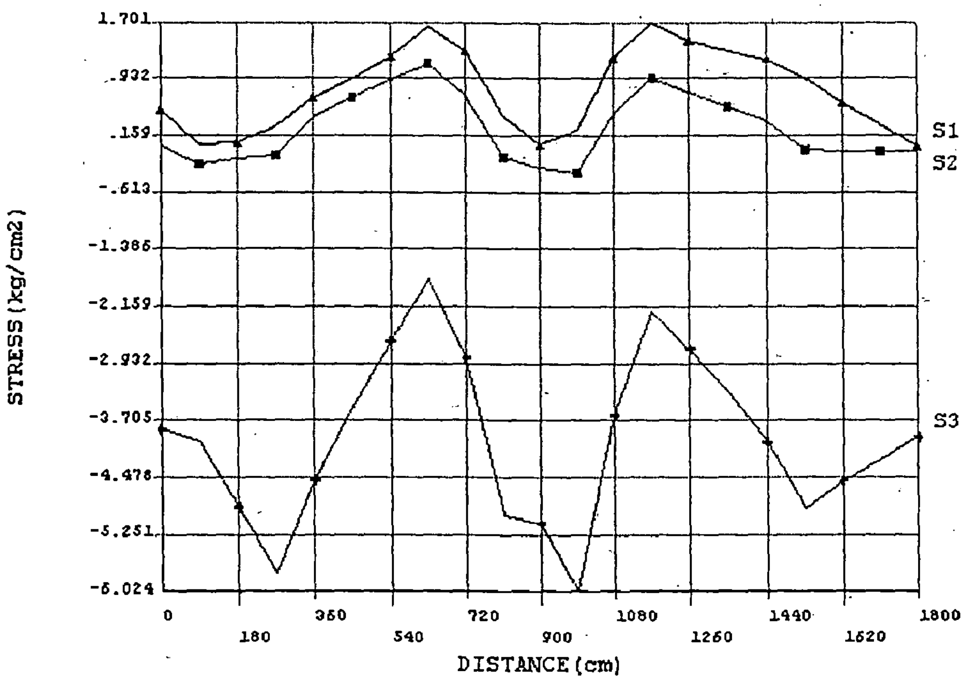


Figure 4.53 Principal Stress at the top of the draft tube openings

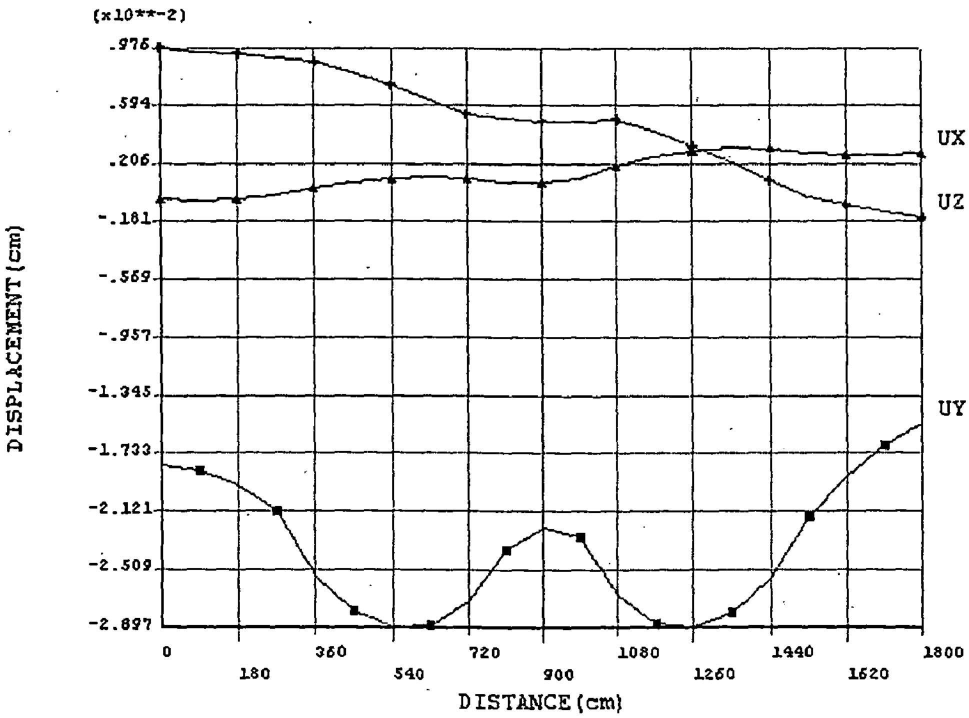


Figure 4.54 Displacement at the top of the draft tube openings

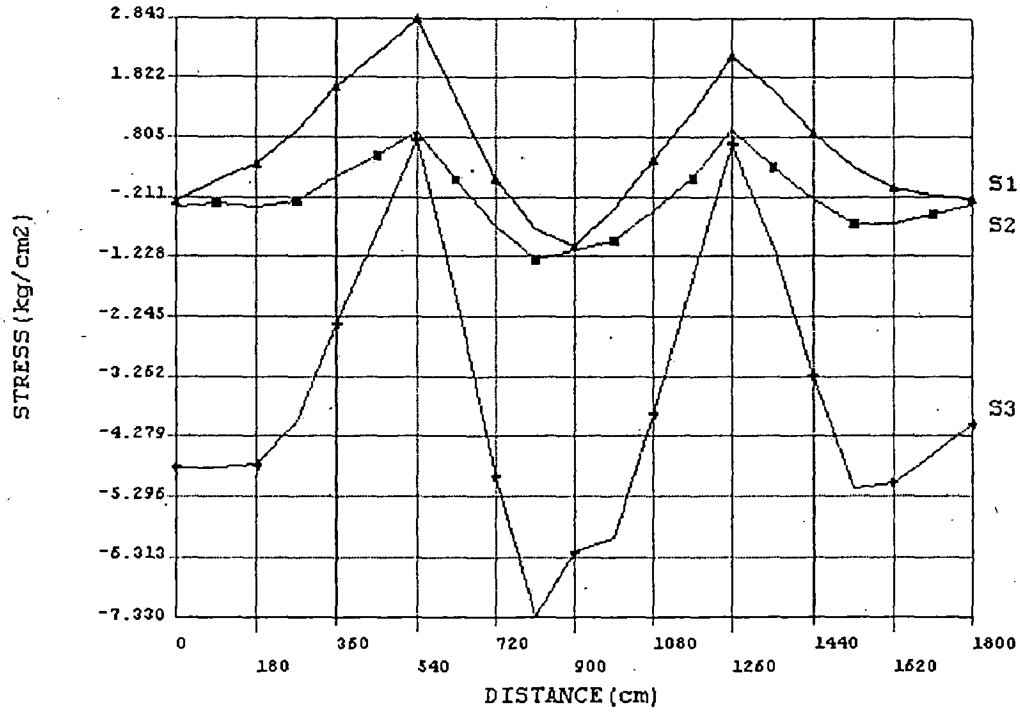


Figure 4.55 Principal Stress at the bottom of the draft tube openings

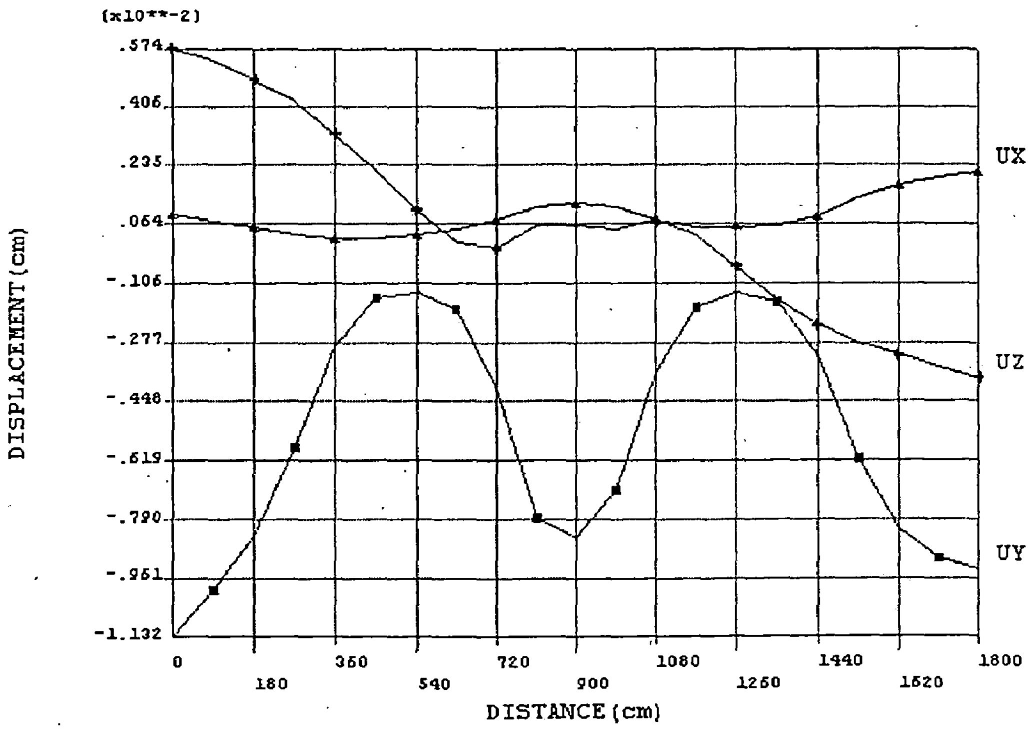


Figure 4.56 Displacement at the bottom of the draft tube openings

4.3 RESULTS OF ANALYSIS (LOAD CASE-B), E = Ec

In load case- B, water pressure in the spiral case and super structure load at the top of the intermediate structure are applied in addition to load case-A. The results of E=Ec are given here

4.3.1 U/S Section (800cm from the turbine axis)

The results of this analysis at this section are discussed below the three locations mentioned earlier.

4.3.1.1 At the top of the draft tube opening

Maximum tensile stress occurred at the left side opening of the draft tube opening with a maximum value of 6.03 kg/cm^2 which is less than the permissible stress of the concrete (7kg/cm^2). Maximum compressive stress has been observed of the order of 12.76 kg/cm2. above the draft tube pier. This is also less than permissible compressive stress of the concrete. Maximum vertical deflection has been observed of the order of 0.09 cm .Results are shown in the Figures-4.57 and 4.58.

4.3.1.2 At the bottom of the draft tube opening

Maximum tensile stress of the order of 3.32kg/cm^2 occurred at the left side opening of the draft tube. This is less than the permissible tensile stress of the concrete.

Maximum compressive stress of the order of 14.51kg/cm^2 was observed below the draft tube pier, which is well within than the compressive permissible limit of the concrete. Maximum vertical displacement of the order of 0.05 cm has been observed at the edges of the structure.Results are shown in Figures-4.59 and 4.60.

4.3.1.3 At the foundation contact

Maximum tensile and compressive stresses were observed as 2.71and 11.82 kg/cm2 respectively. Maximum tensile stress has been observed in the foundation zone while maximum compressive stress has been noticed at the left edge of the section. These stresses are also within their respective permissible limits. Maximum vertical displacement Of the order of 0.04 cm has been observed at three locations, middle and approximately at a distance of 675 cm on each side of the middle of the section as has been shown in the Figures-4.61 and 4.62.

4.3.2 D/S Section (1200cm from the turbine axis)

4.3.2.1At the top of the draft tube openings

Maximum tensile stress of the order of 2.39 kg/cm^2 has been observed at the right edge of the structure and maximum compressive stress of the order of 4.57 kg/cm2 above the draft tube pier. Maximum vertical displacement has been noticed at the right side opening having the value of 0.05 cm as observed from the Figures-4.67 and 4.68 and from the Table 4.2.

.The tensile and compressive stresses were within permissible limits.

4.3.2.2At the bottom of the draft tube openings

Maximum tensile and compressive stresses were having the value of the order of 2.10 kg/cm2 and 6.73 kg/cm2 respectively. Maximum tensile stress has been observed at the left opening of the draft tube 540 cm from the edge of the section. And maximum compressive stress was observed below the draft tube pier. The stresses were within permissible limits. Maximum displacement of the order of 0.037was observed at the left edge of the section results are given in Figure-4.69 and 4.70 and Table 4.2

4.3.2.3 At the foundation contact

Maximum tensile stress was having a value of1.80 kg/cm^2 at the right side in the foundation and maximum compressive stress was having a value of 6.25 kg/cm^2 at the left edge of the structure but both the stresses were within permissible limit. Maximum value of the vertical displacement was observed of the order of 0.03 cm at the right edge of the section. Results are shown in the Figure-4.71 and 4.72.

4.3.3 Sub Structure Fixed at the Base

At the u/s and the d/s section the stresses and vertical displacement show the same trend as discussed above for the structure with foundation rock. A slight increase in tensile stress is noticed at the opening of the structure. But the compressive stresses and vertical displacement are less when compared with the structure with foundation rock (Table-4.2).Tensile stress does not exceed the permissible stress in concrete (7 kg/cm2) at any location of the section. Results are given in Figure-4.73, 4.74 4.75and 4.76.

LOAD CASE - B

(A) U/S SECTION AT THE DRAFT TUBE (E=Ec)

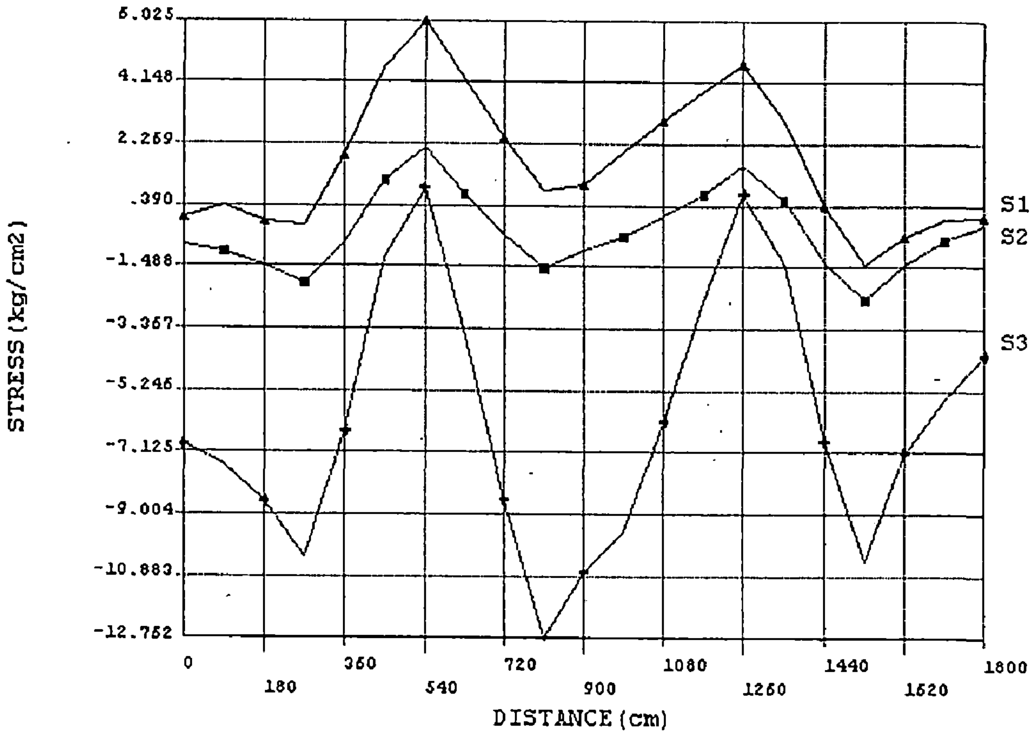


Figure 4.57 Principal Stress at the top of the draft tube openings

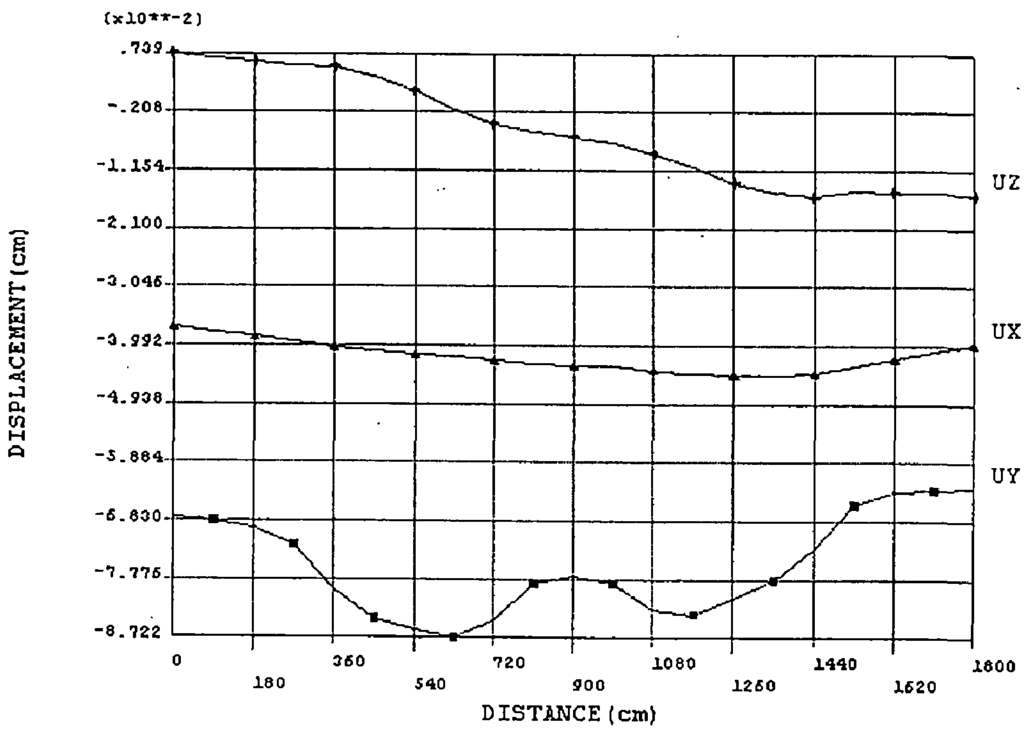


Figure 4.58 Displacement at the top of the draft tube openings

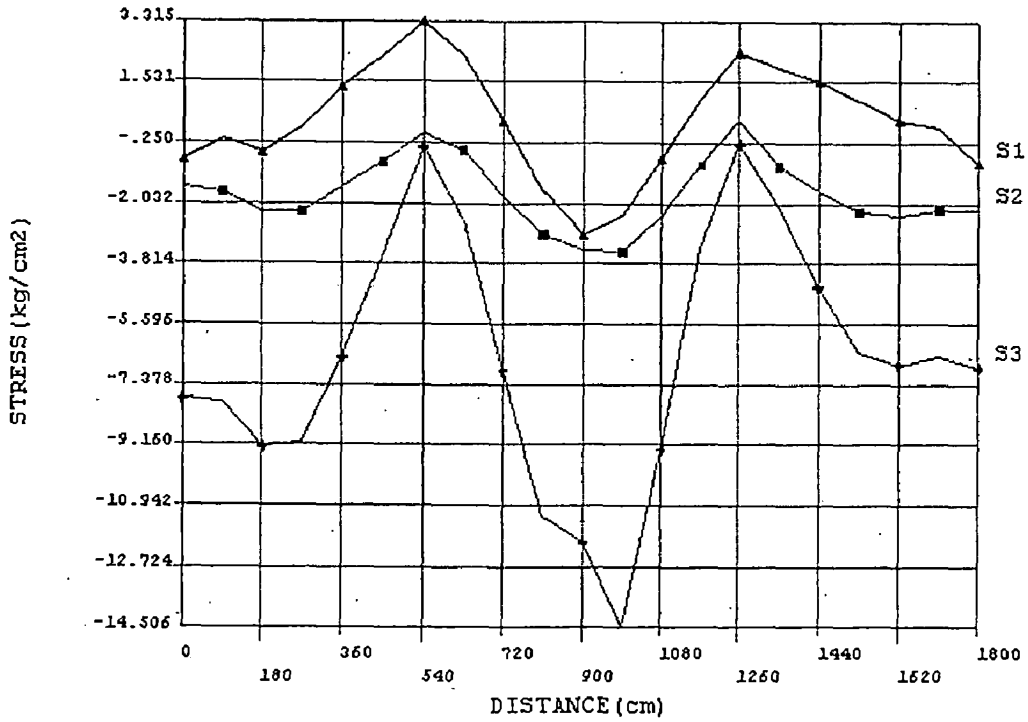


Figure 4.59 Principal Stress at the bottom of the draft tube openings

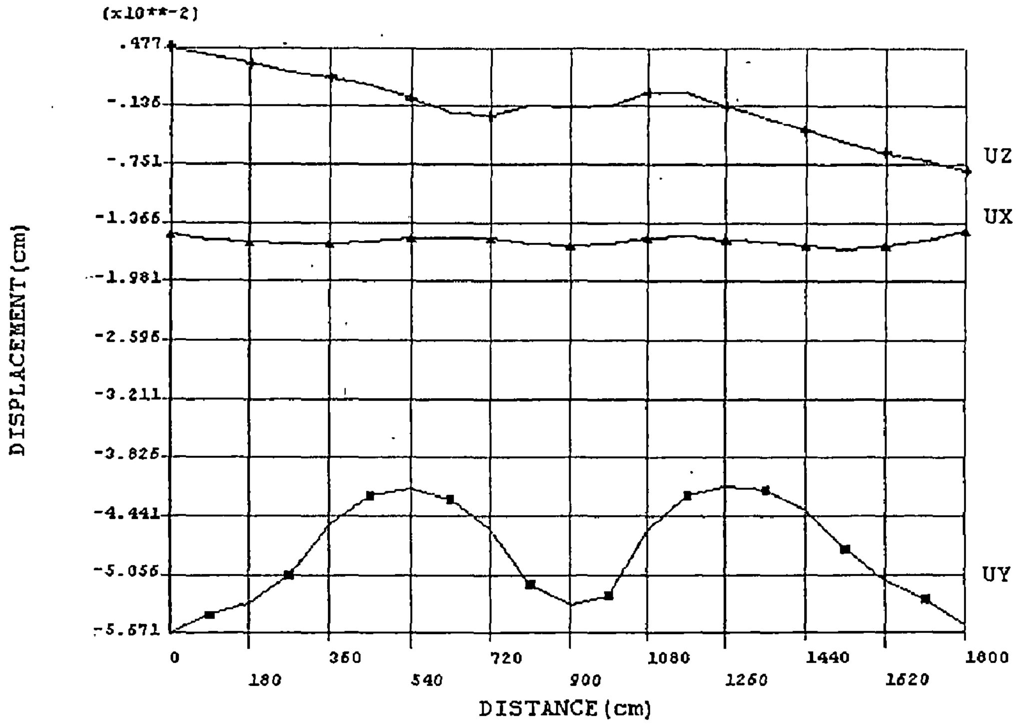


Figure 4.60 Displacement at the bottom of the draft tube openings

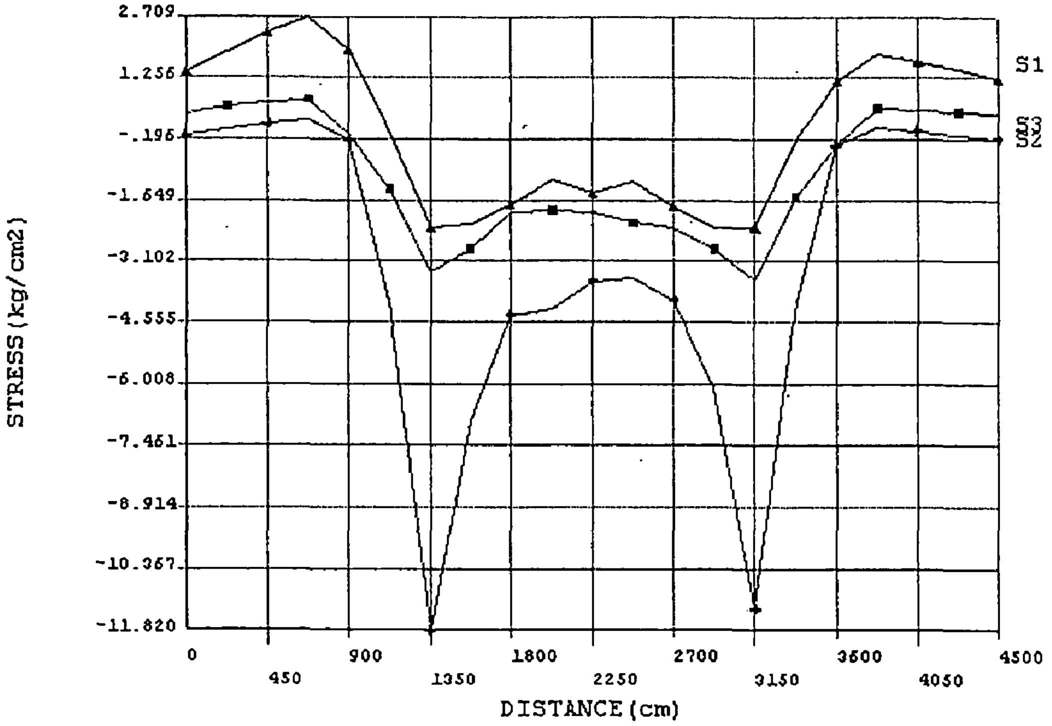


Figure 4.61 Principal Stress at the foundation contact

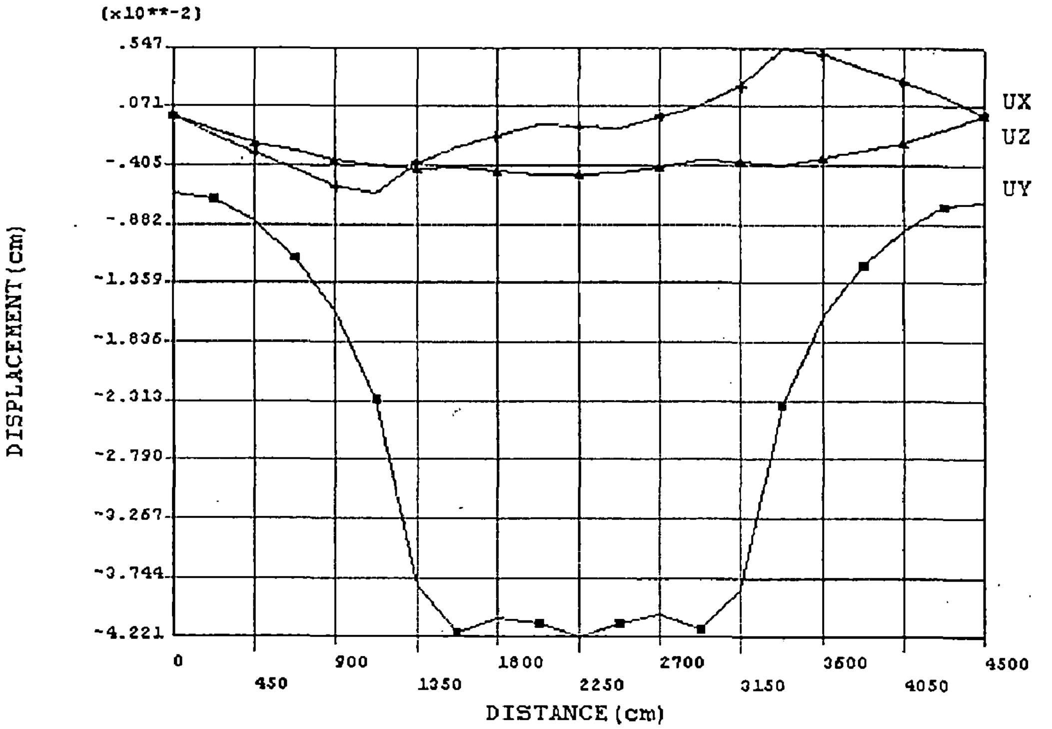


Figure 4.62 Displacement at the foundation contact

(B) SUBSTRUCTURE FIXED AT THE BASE

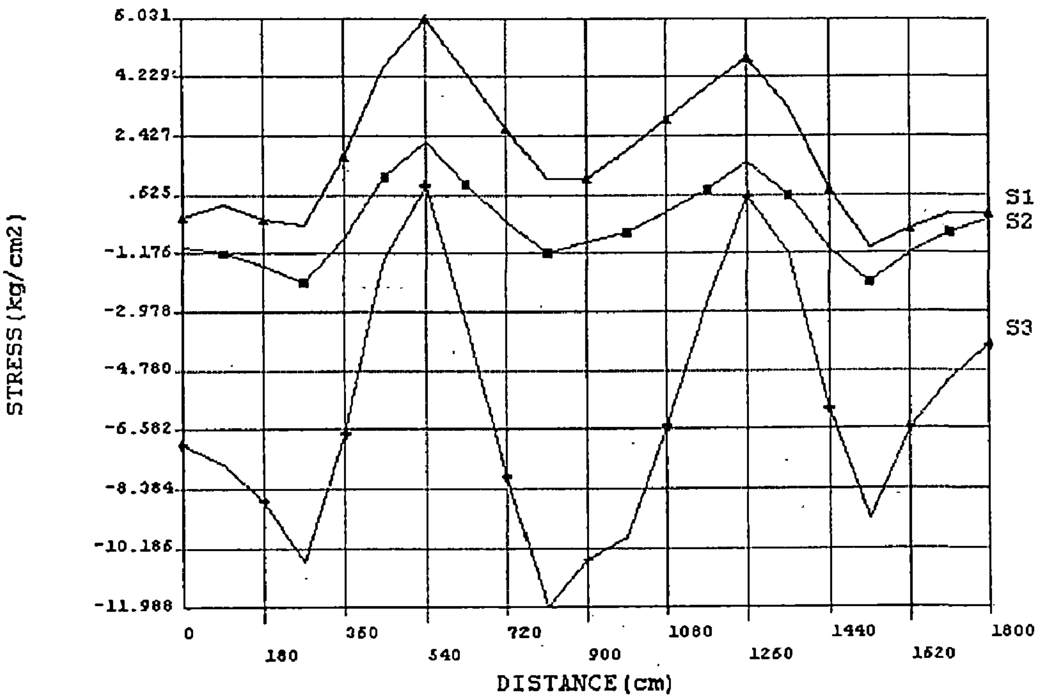


Figure 4.63 Principal Stress at the top of the draft tube openings

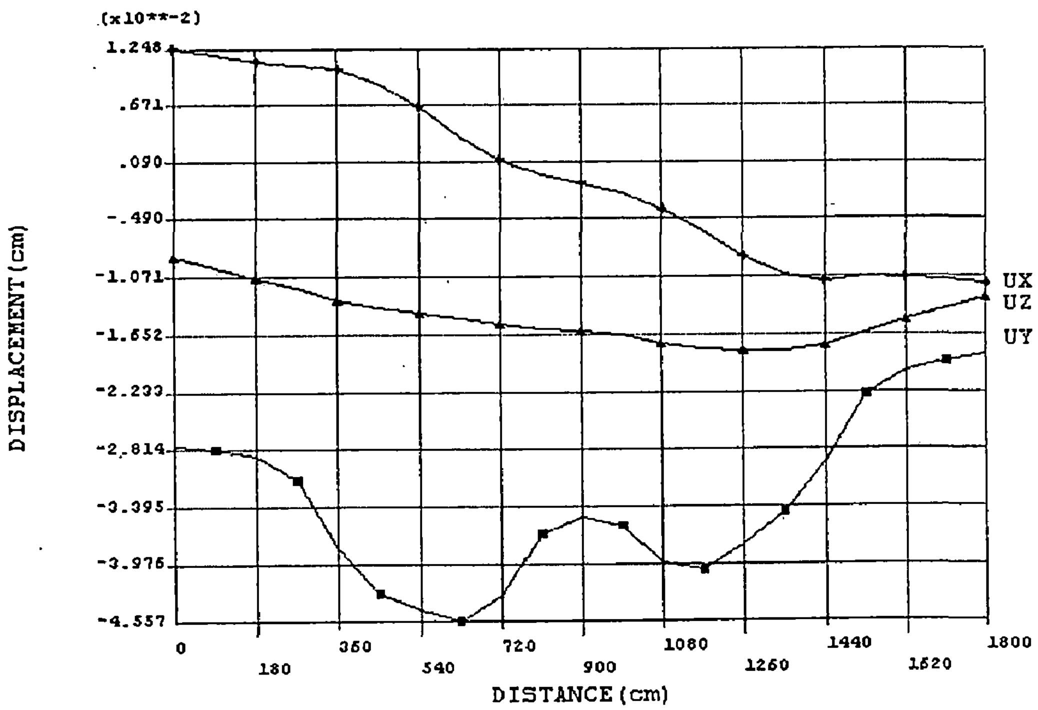


Figure 4.64 Displacement at the top of the draft tube openings

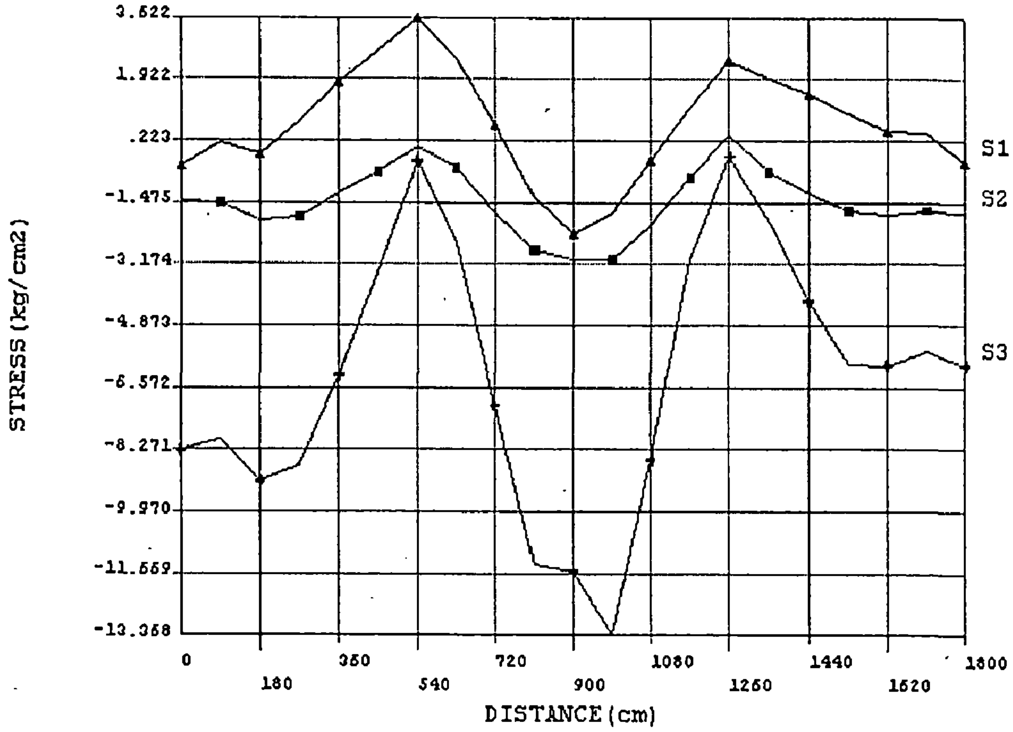


Figure 4.65 Principal Stress at the bottom of the draft tube openings

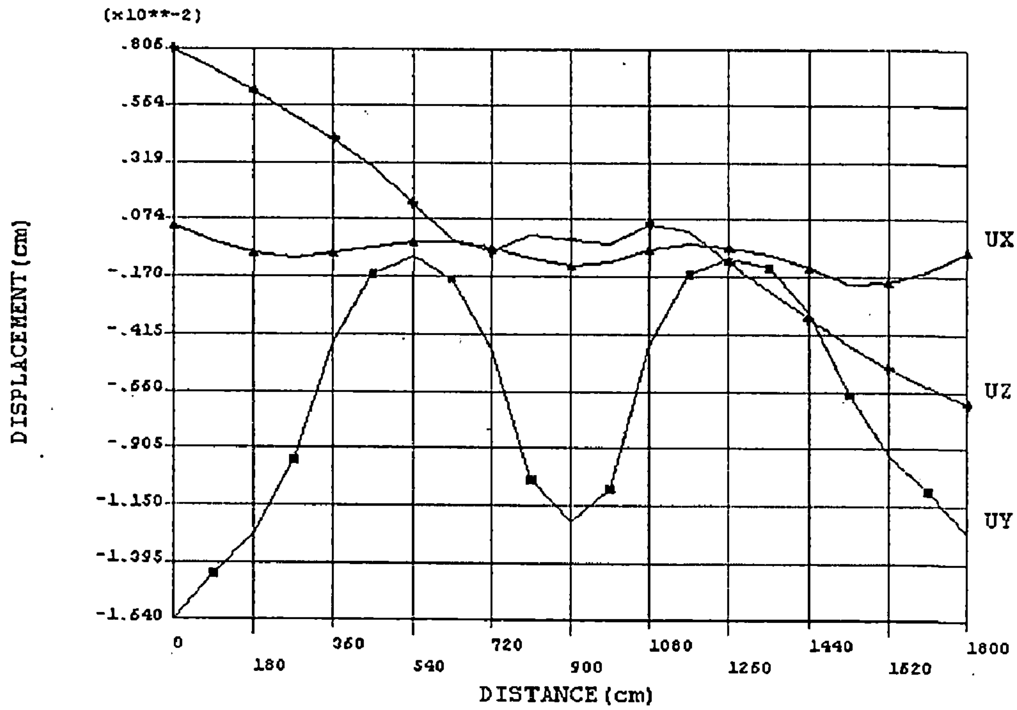


Figure 4.66 Displacement at the bottom of the draft tube openings

LOAD CASE –B

D/S SECTION AT THE DRAFT TUBE (E=Ec)

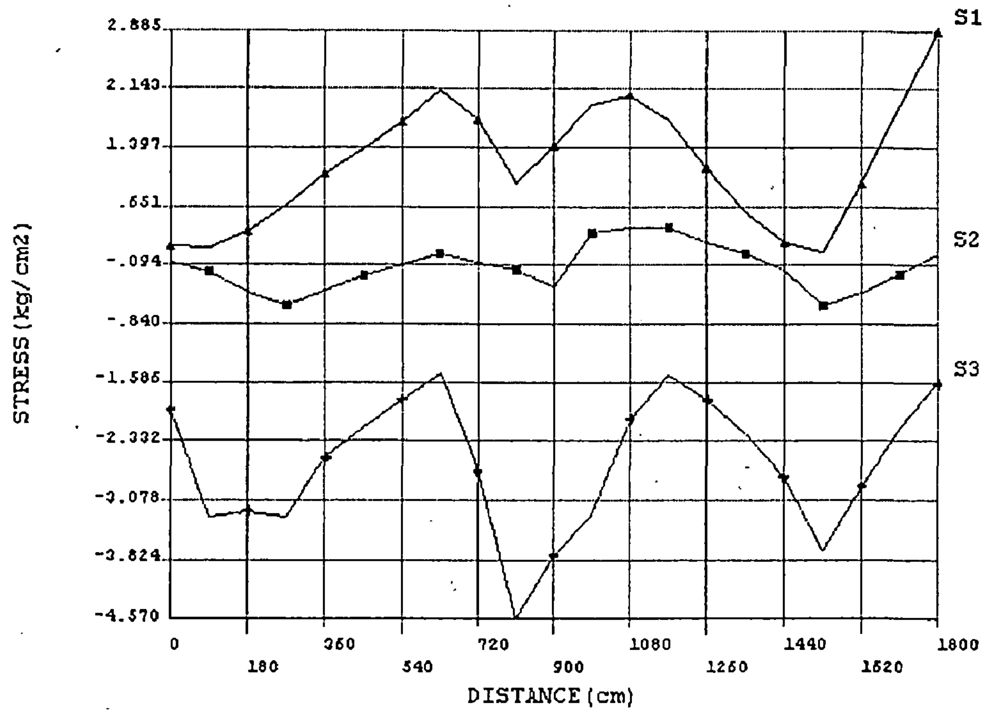


Figure 4.67 Principal Stress at the top of the draft tube openings

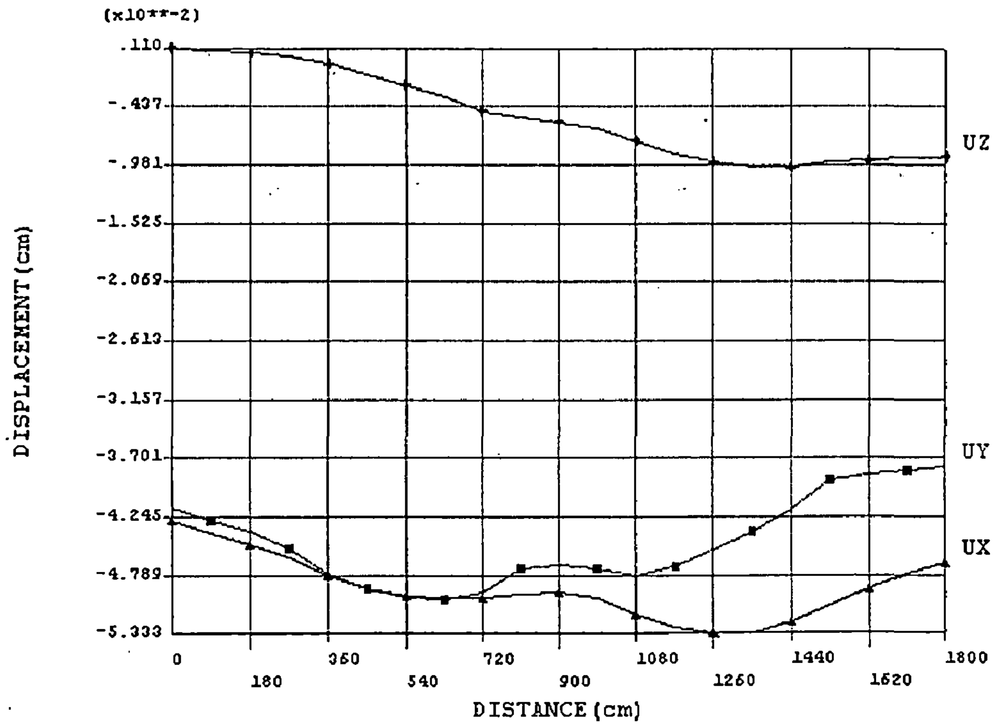


Figure-4.68 Displacement at the top of the draft tube openings

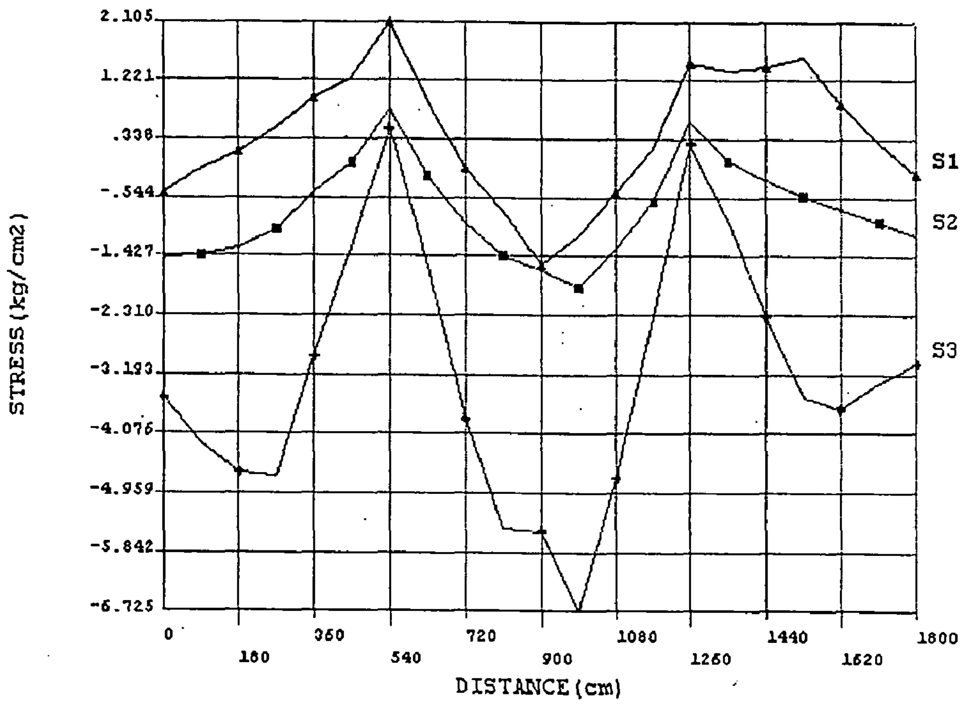


Figure 4.69 Principal Stress at the bottom of the draft tube openings

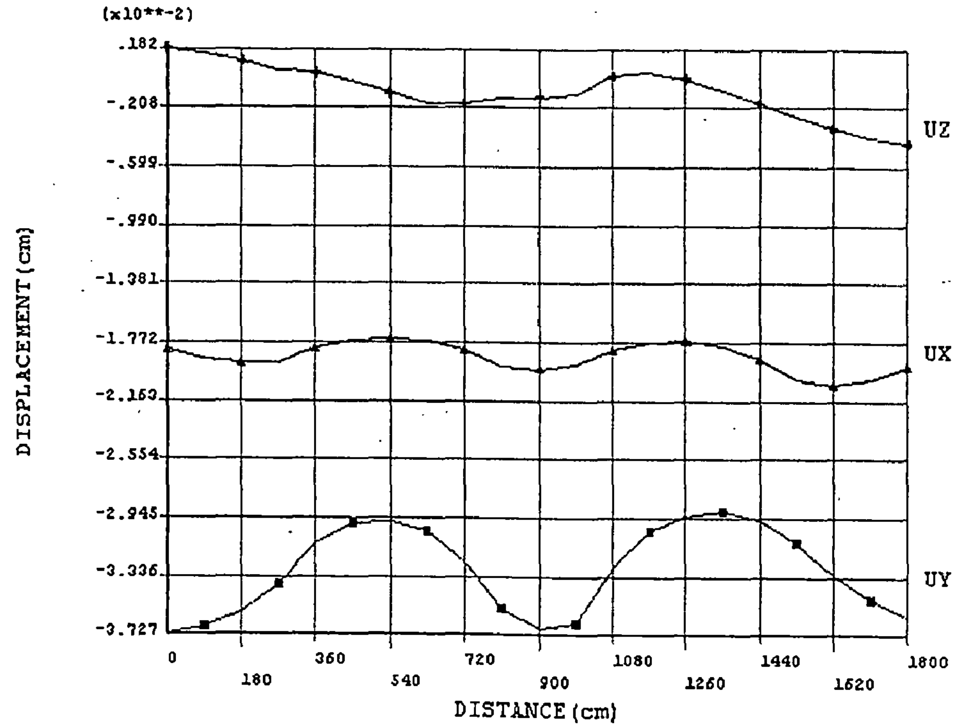


Figure 4.70 Displacement at the bottom of the draft tube openings

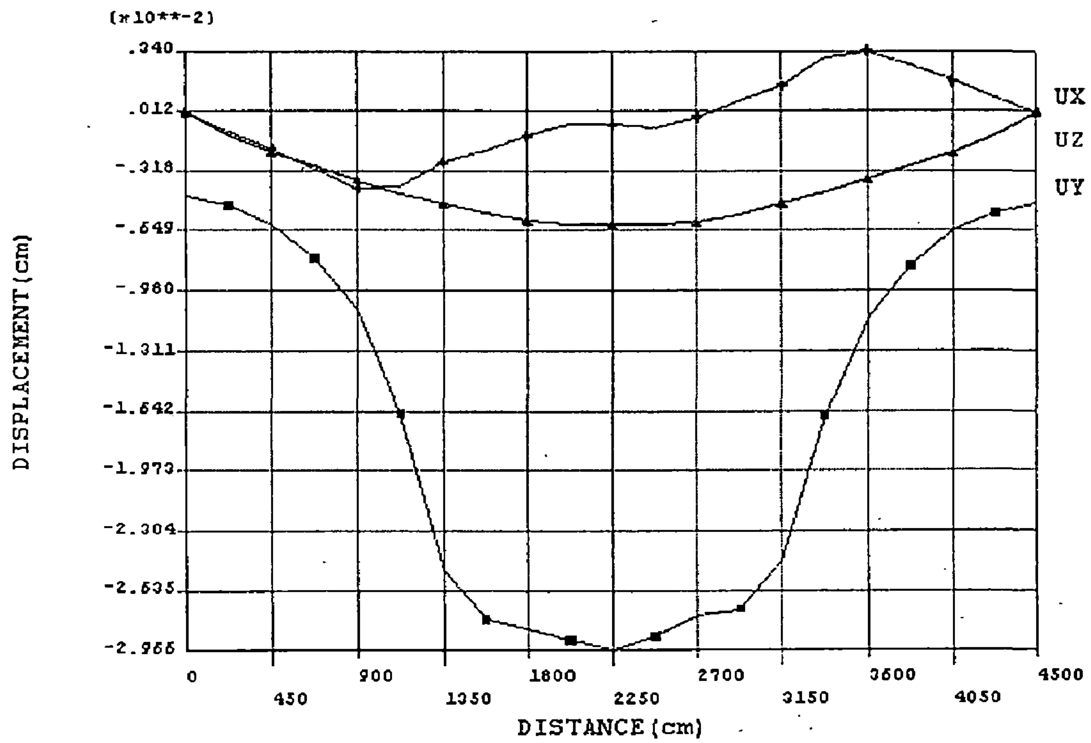


Figure 4.71 Principal Stress at the foundation contact

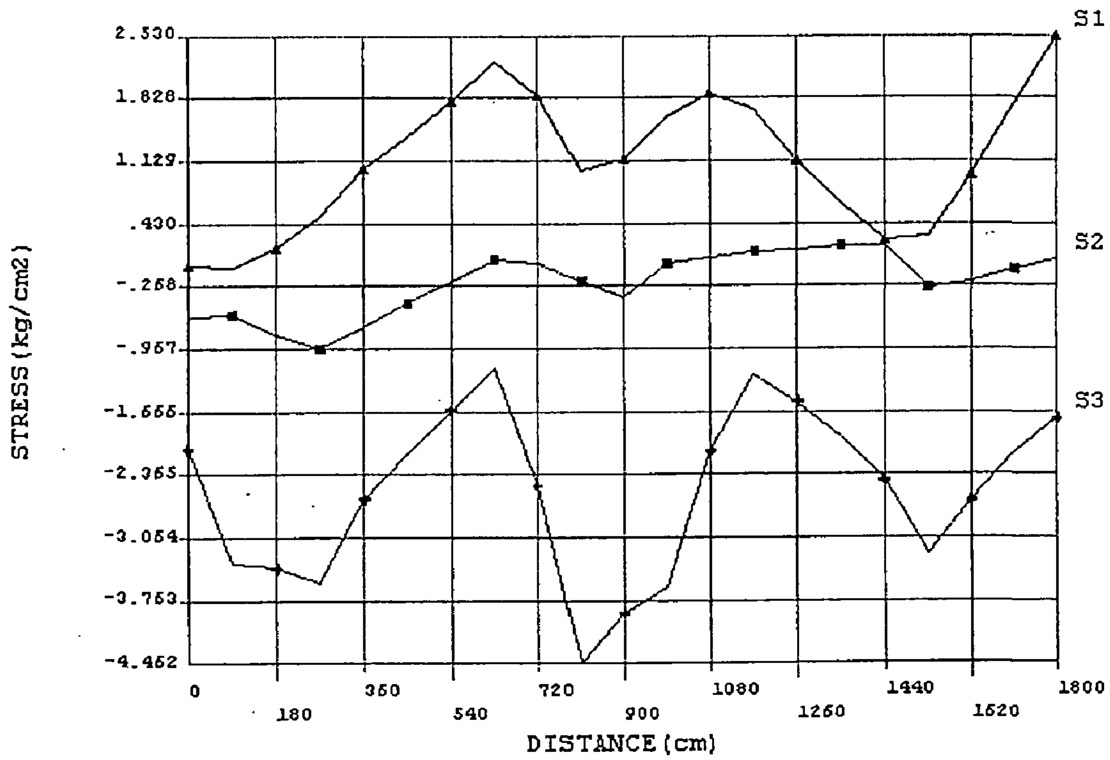


Figure 4.72 Displacement at the foundation contact

(B) D/S SECTION (SUB STRUCTURE FIXED AT THE BASE)

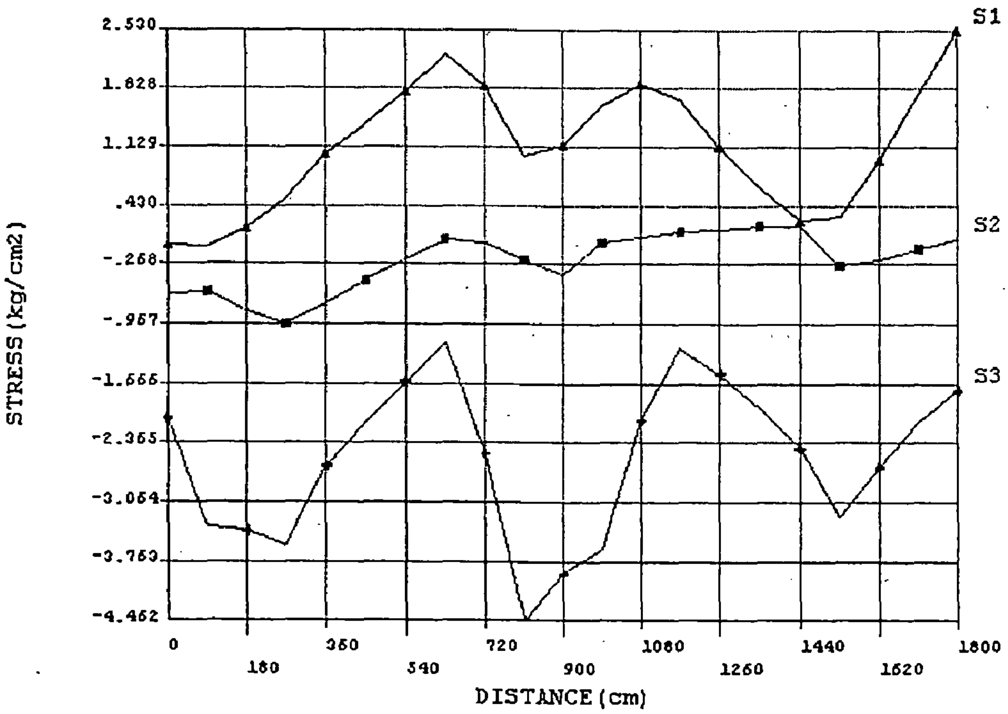


Figure 4.73 Principal Stress at the top of the draft tube openings

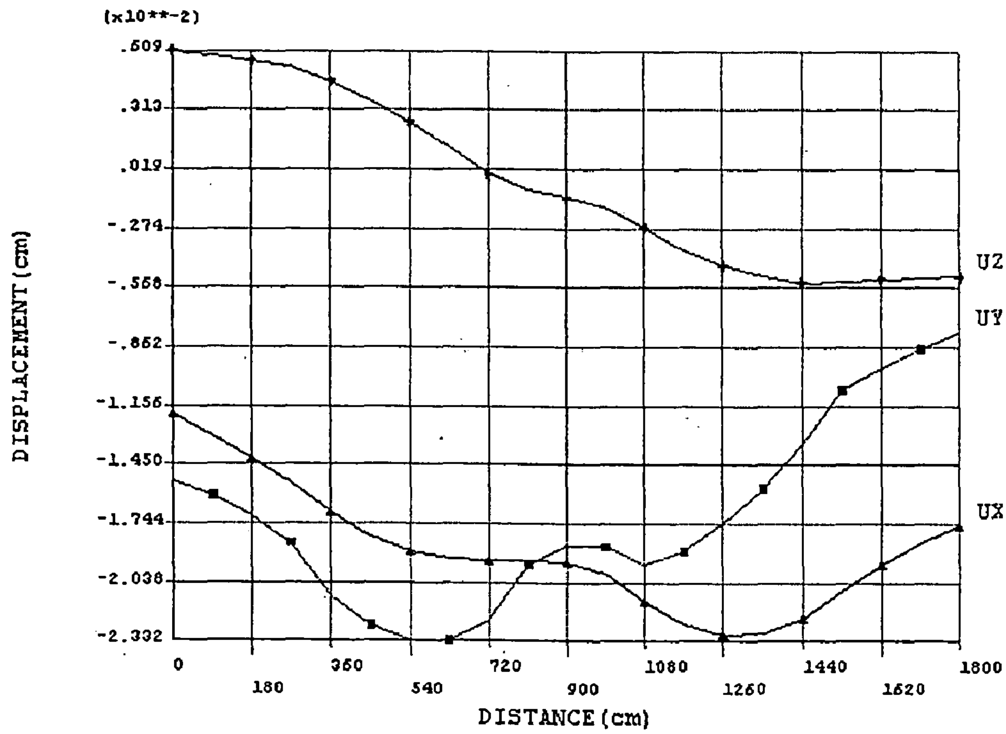


Figure 4.74 Displacement at the top of the draft tube openings

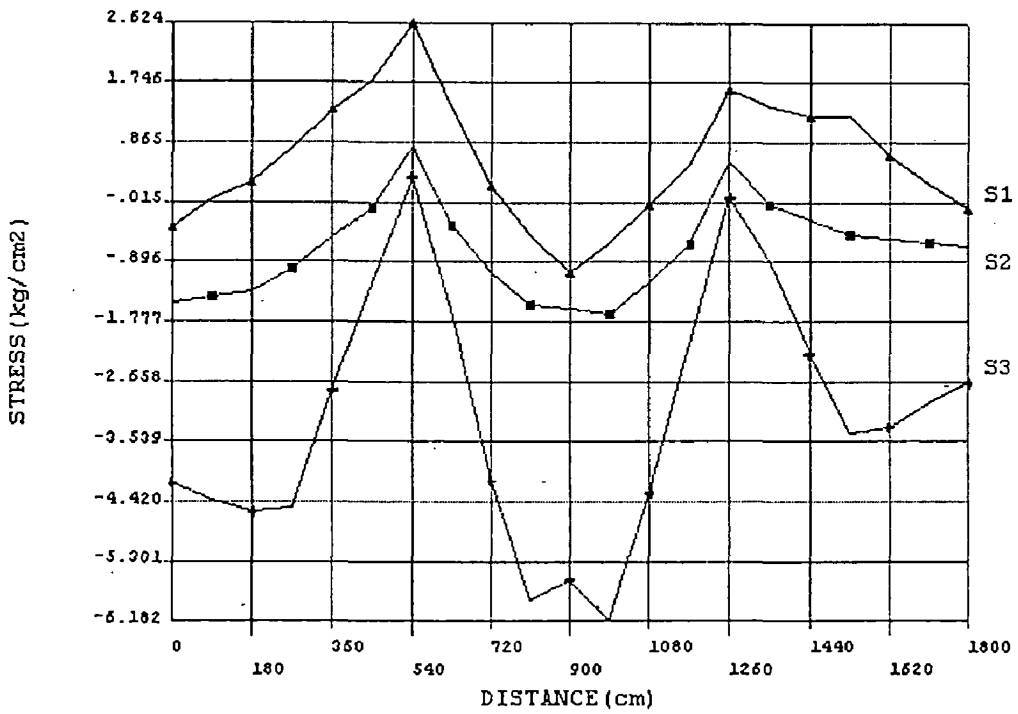


Figure 4.75 Principal Stress at the bottom of the draft tube openings

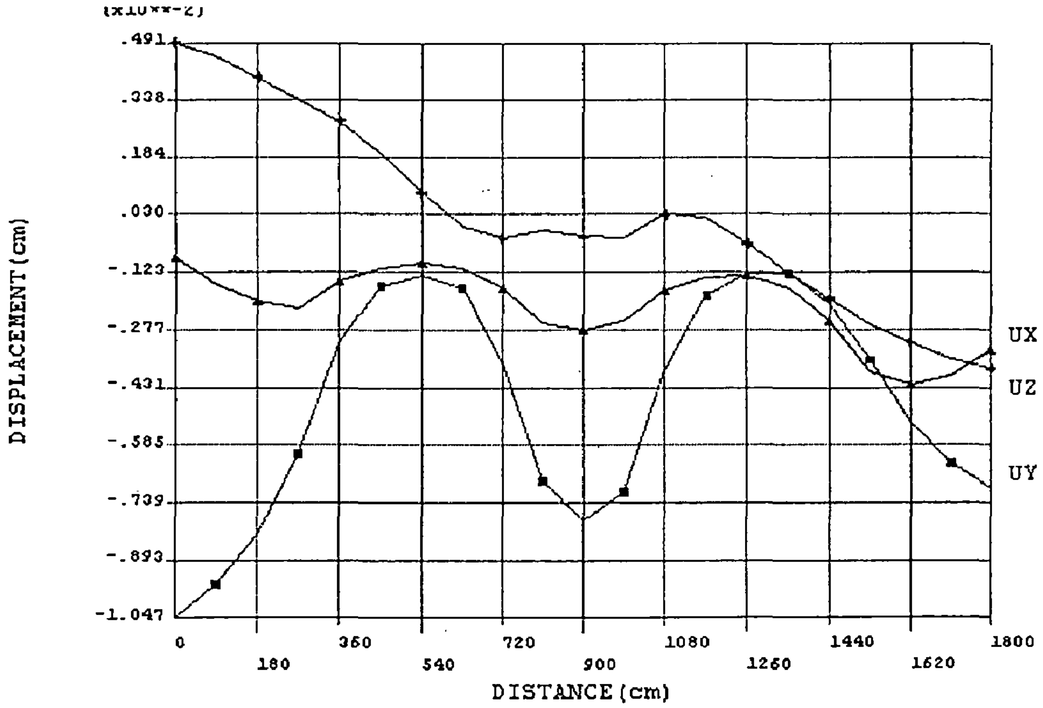


Figure 4.76 Displacement at the bottom of the draft tube openings

Table -4.1 Summary of the Results
Load case –A (Self weight, Rotor load, Stator load, Short circuit torque)

Foundation Condition	U/S section (800cm from turbine axis)									D/S section (1200 cm from turbine axis)								
	TODT			BODT			FC			TODT			BODT			FC		
	S1	S3	Uy	S1	S3	Uy	S1	S3	Uy	S1	S3	Uy	S1	S3	Uy	S1	S3	Uy
E=10*Ec	3.55	8.36	0.03	2.58	8.46	0.01	1.21	6.45	0.002	1.70	5.02	0.03	2.80	7.31	0.01	0.98	5.18	0.002
E=Ec	3.47	8.22	0.05	2.47	8.31	0.03	1.22	7.02	0.02	1.71	5.04	0.05	2.61	7.21	0.03	0.96	5.63	0.02
E=Ec/10	3.25	7.70	0.22	2.39	7.74	0.20	1.17	6.96	0.19	1.87	6.48	0.2	2.24	6.87	0.18	0.91	5.93	0.18
E=Ec/100	3.04	7.14	1.84	2.30	7.13	1.80	**1.96**	6.62	1.82	2.18	7.31	1.72	2.14	6.48	1.71	**1.83**	5.82	1.70
Substructure fixed at base	3.57	8.38	0.03	2.60	8.48	0.01	-	-	-	1.70	5.02	0.03	2.84	7.33	0.01	-	-	-

Table -4.2 Summary of the Results
Load case –B (Self weight, Rotor load, Stator load, Short circuit torque, Water pressure, Super structure)

Foundation Condition	U/S section (800cm from turbine axis)									D/S section (1200 cm from turbine axis)								
	TODT			BODT			FC			TODT			BODT			FC		
	S1	S3	Uy	S1	S3	Uy	S1	S3	Uy	S1	S3	Uy	S1	S3	Uy	S1	S3	Uy
E=Ec	6.03	12.76	0.09	3.32	14.51	.05	2.71	11.82	0.04	2.39	4.57	0.05	2.10	6.73	0.037	1.80	6.25	0.03
Substructure fixed at base	6.03	11.98	0.046	3.62	13.37	0.016	-	-	-	2.52	4.45	0.023	2.52	5.18	0.01	-	-	-

TODT- Top of the Draft Tube, **BODT**- Bottom of the Draft Tube, **FC**-Foundation contact
S1, S2, S3 in kg/cm2, Uy in cm.

4.4 ANALYSIS OF STRESS AND DISPLACEMENT CONTOURS ALONG TRASEVERSE AND LONGITUDINAL SECTION (LOAD CASE-A)

In this para the results of the analysis along the transverse and the longitudinal sections through the center line of turbine are discussed .Principal stress and vertical displacement contours as obtained are presented

4.4.1 Foundation (E=10* EC)

At the section shown in the Figure .4.77 (b), and 4.82(b) the tensile stress is observed at the elbow portion of the draft tube, above and below the spiral casing, at the top of the structure, near the point of application of the load but it does not exceed the permissible tensile stress of the concrete i.e. 7kg/cm2, except near the brackets where tensile stress has the value of the order of 15 kg/cm2 at the transverse section and 65 kg/cm2 at the longitudinal section. The stress contours of this location are grayed out to make clear the contours of other locations in the section .contours of this section are more or less similar as shown in the Figure -4.82 (a1),(a2) (Enlarged to make stresses clear)

Maximum vertical displacement is of the order of .076 cm.

4.4.2 Foundation (E=Ec)

At the transverse section first principal stress will be tensile in the elbow portion and at the top of the draft tube between 1.55kg/cm2 to 2.0 kg/cm^2.. Tension is also noticed above the spiral casing of the order of 0.87 kg/cm^2 to 1.11kg/cm^2 .Principal stress S3 shows compression of the order of 12 kg/cm^2 at the pier.

In the longitudinal direction principal stress S1 shows the tensile value at near the point of application of the load, around spiral casing and at the lower portion of the draft tube. The value of tension near point of application of load is greater than the value of tension in other portion of the structure.S3 value shows compression near the foundation contact and some tension near the spiral casing but the tensile value of S1 is governing at that location. Tension at any location under discussion does not exceed the permissible tensile value ($7 kg/cm^2$) of concrete except near the point of application of loads The maximum displacement in the section is of the order of.0.09 cm, which is negligible. Refer Figure-4.78

4.4.3 Foundation (E=Ec/10)

This foundation condition shows the same stress pattern (Figure-4.79) Tensile stress of the order of $2kg/cm^2$ has been observed above the draft tube in the elbow portion(figure-4.79(b)). The vertical displacement increases at both the sections the value of displacement is of the order of 0.27 cm (Figure 4.79) ,This indicates that as the value of E decreases to one tenth of concrete, the displacement increases three times.

4.4.4 Foundation (E=Ec/100)

With the reduction of E value, tensile as well as compressive stresses increase in the sections and the vertical displacement of the order of 2.37 cm has also increased (Figure 4.80 (e),(f)) . This indicates (Figure-4.80) that stresses in the structure increases with the decrease the stiffness of the foundation.

4.4.5 Sub Structure Fixed at the Base

Tensile stress is observed at the elbow portion, below the draft tube, above and below the spiral casing and top of the structure near the point of application of the loads. Compression is observed at the pier of the draft tube (Figure4.81

(b) of the order of 1kg/cm^2. Displacement and stress pattern resembles with the foundation condition E=10*Ec. (Figure -4.81)

4.5 ANALYSIS OF STRESS AND DISPLACEMENTCONTOURS ALONG TRASEVERSE AND LONGITUDINAL SECTION (LOAD CASE-B)

The analysis of the stress and displacement contours results are discussed under the following headings

4.5.1 Structure With Foundation (E=Ec)

In the longitudinal direction first principal stress (S1) is of the order of 60 kg/cm^2 near the load application zone (Figure 4.83 (b)). Tension prevails around the spiral casing which some times increases the permissible tensile value of the concrete (7kg/cm^2). Third principal stress S3 is compressive and of the order of 30 kg/cm^2 near the edges of foundation contact.

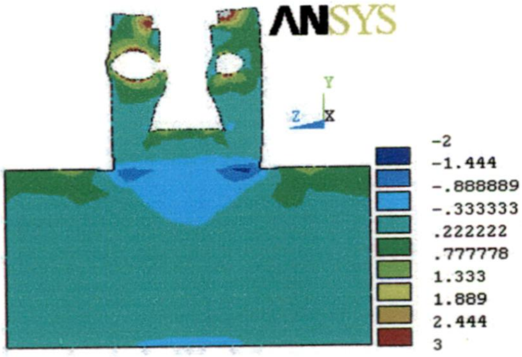
The Transverse Section in the right side portion, near the spiral casing experiences tensile stress (S1) which increases to 13.33 kg/cm^2 .The tension is also noticed at the elbow portion above the draft tube. At these locations tension in the structure is more than the permissible tensile stress (7kg/cm^2) of concrete. S3 is compressive near the foundation contact and also shows a tensile pattern at the top of the structure but the tensile stress is governed by the first principal stress (S1).At this section the compressive stress (S3) of the order of 170kg/cm^2 is noticed at the top edge (Figure 4.83 (d)) due to super structure load. Compression (S3) is also noticed at the pier.

Vertical displacement (Uy) of the order of 0.99cm is noticed near the top left of the transverse section due to super structure load at this location. At the longitudinal section, Uy is having the value of the order of.0.18 cm near the spiral casing (Figure 4.83).

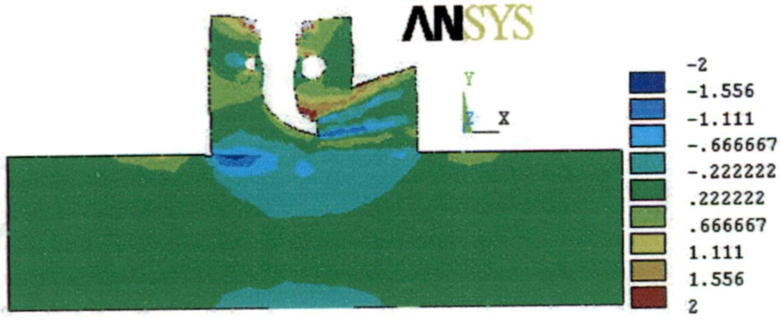
4.5.2 Sub Structure Fixed at the Base

When compared with the Sub base fixed (Figure-4.84), it is noticed that the pattern of the stresses S1 and S3 resemble with the above condition. But some change has been noticed at the foundation contact. A slight increase in the stresses is found in the case when structure is analysed with the foundation. But the displacement (Uy) values are showing the increasing trend at every point of the sections under discussion.

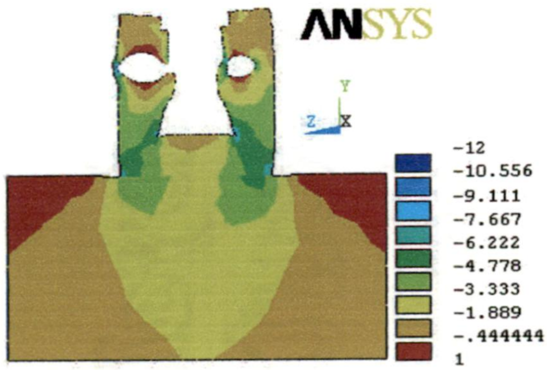
FOUNDATION (E=10*Ec)
LOADS –Self weight, Rotor, Stator, Short Circuit Torque



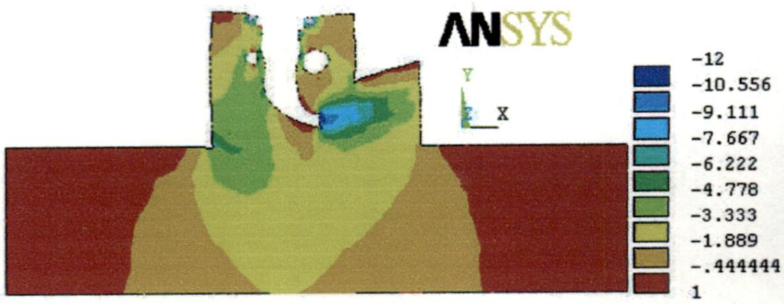
(a) Principal Stress S1



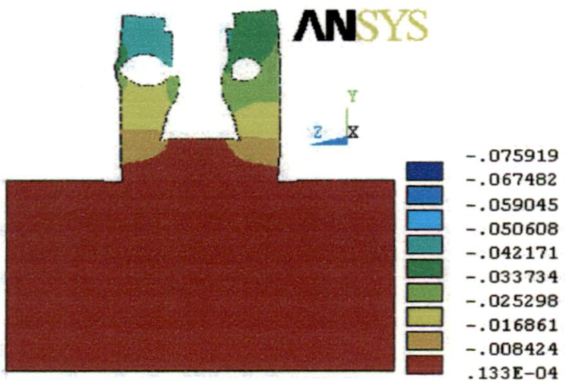
(b) Principal Stress S1



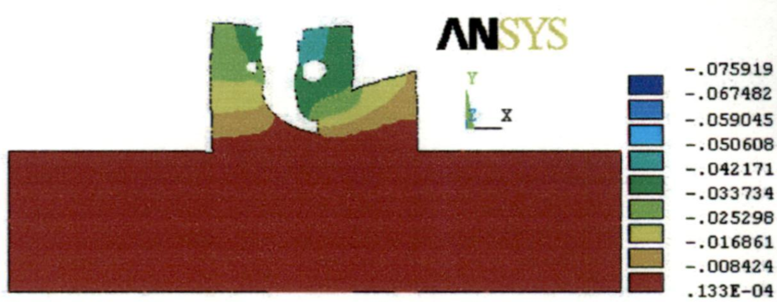
(c) Principal Stress S3



(d) Principal Stress S3



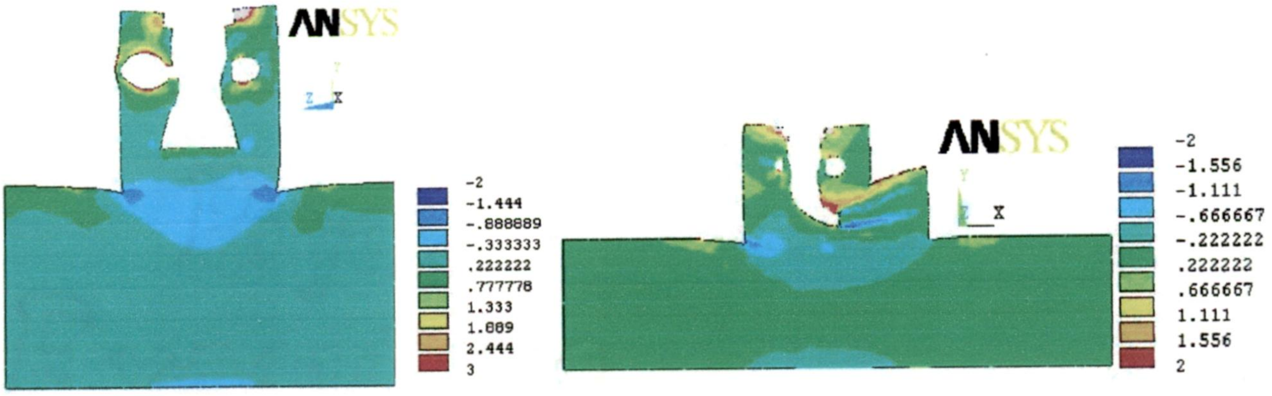
(e) Vertical Displacement Uy



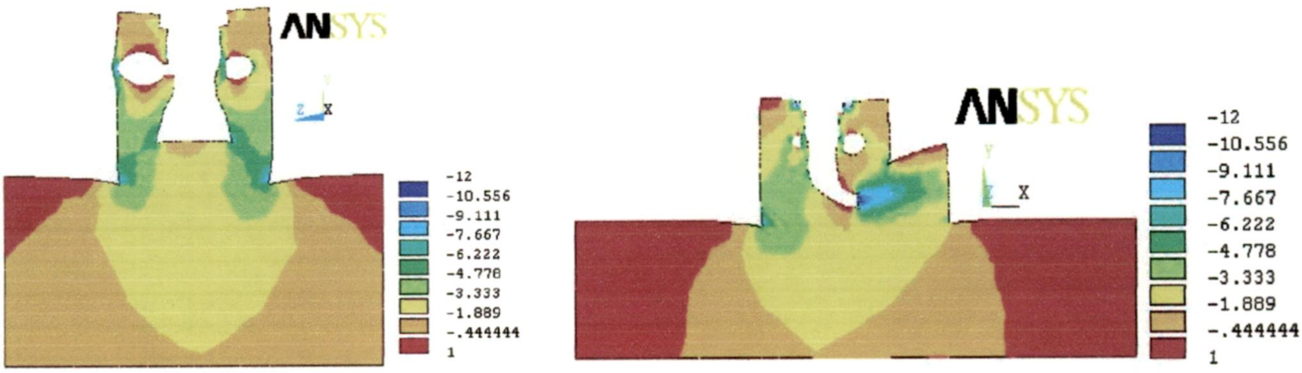
(f) Vertical Displacement Uy

Figure 4.77 Principal Stress S1, S3 and Vertical deflection U_y

FOUNDATION (E=Ec)
LOADS –Self weight, Rotor, Stator, Short Circuit Torque



(a) Principal Stress S1 **(b) Principal Stress S1**



(c) Principal Stress S3 **(d) Principal Stress S3**

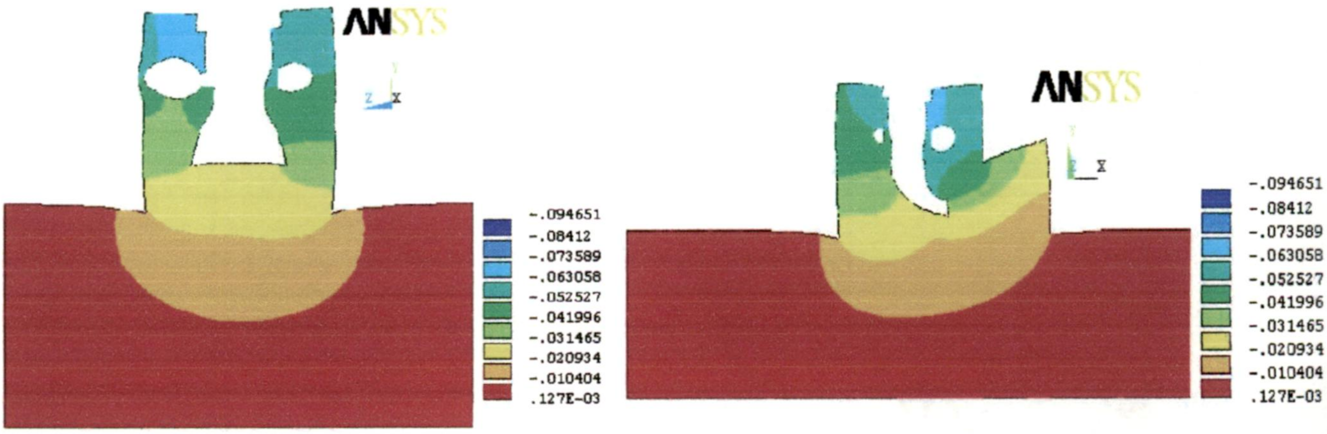
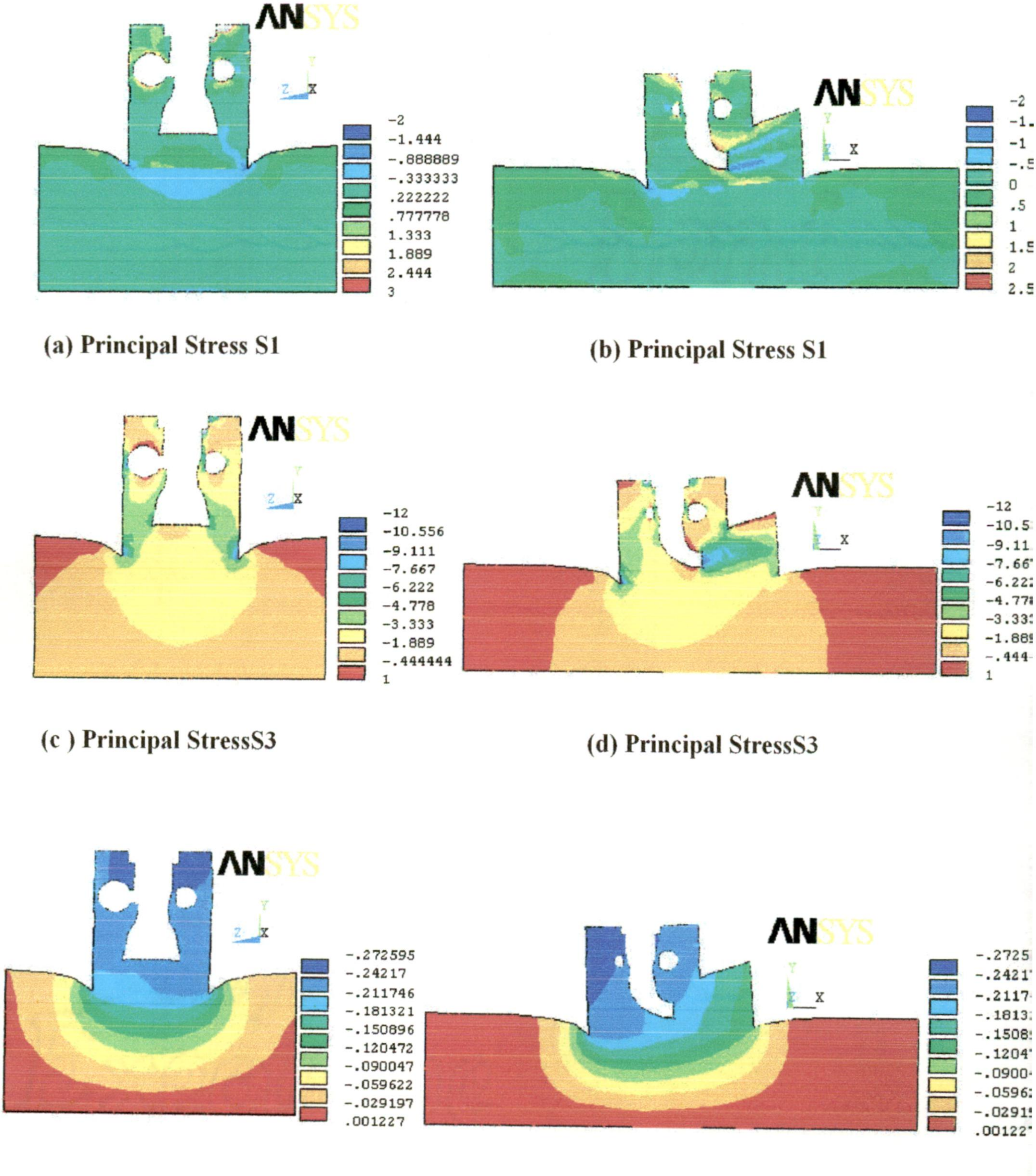


Figure 4.78 Principal Stress S1, S3 and Vertical deflection U_y

FOUNDATION (E=Ec/10)

LOADS –Self weight, Rotor, Stator, Short Circuit Torque



(a) Principal Stress S1

(b) Principal Stress S1

(c) Principal StressS3

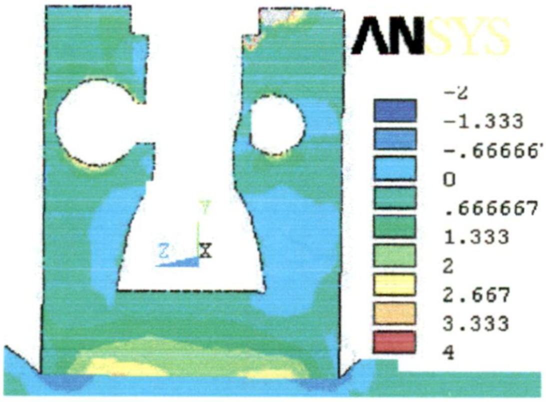
(d) Principal StressS3

(e) Vertical Displacement Uy

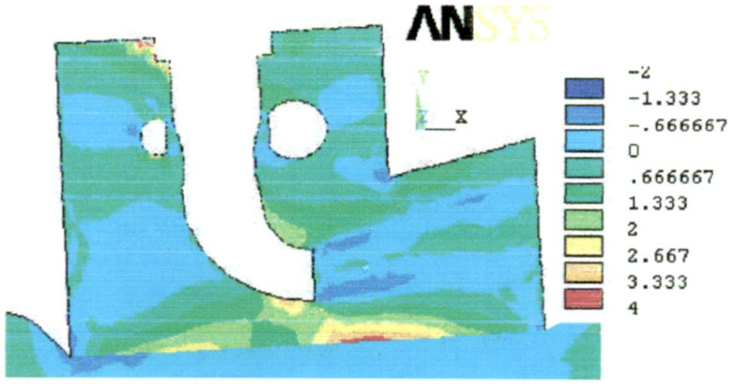
(f) Vertical Displacement Uy

Figure 4.79 Principal Stress S1, S3 and Vertical deflection U_y

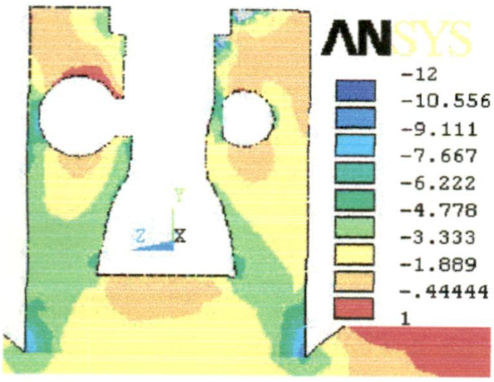
FOUNDATION (E=Ec/100)
LOADS –Self weight, Rotor, Stator, Short Circuit Torque



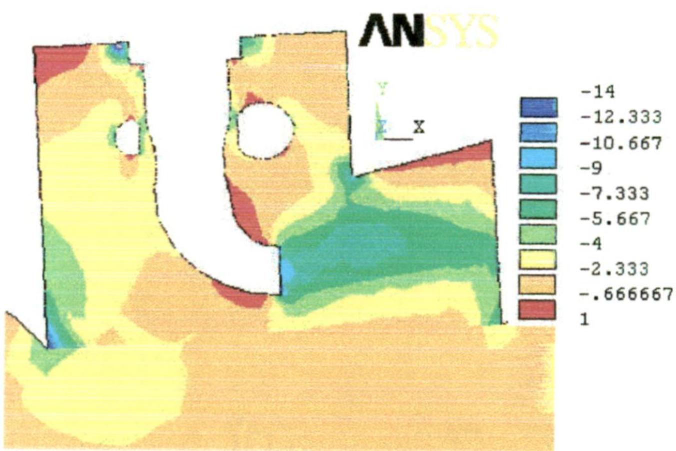
(a) Principal Stress S1



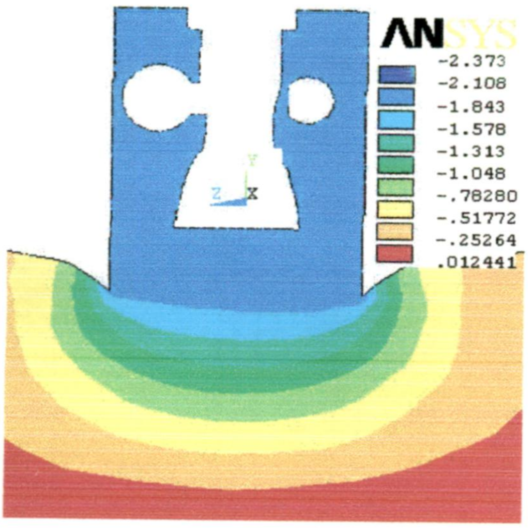
(b) Principal Stress S1



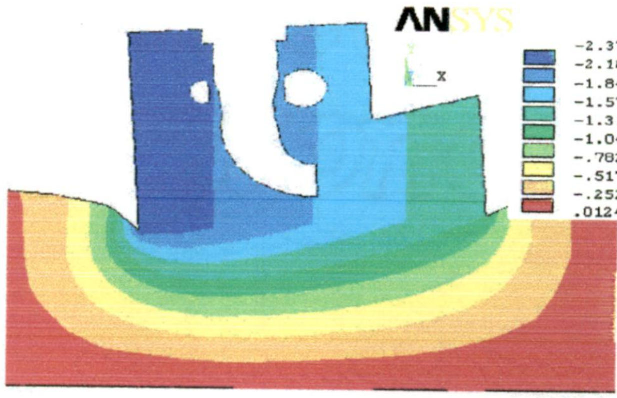
(c) Principal Stress S3



(d) Principal Stress S3



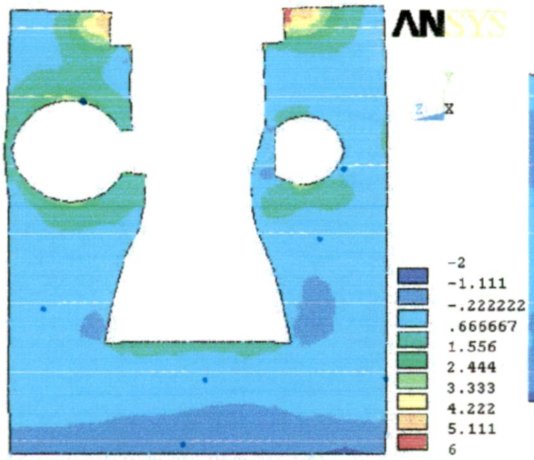
(e) Vertical Displacement Uy



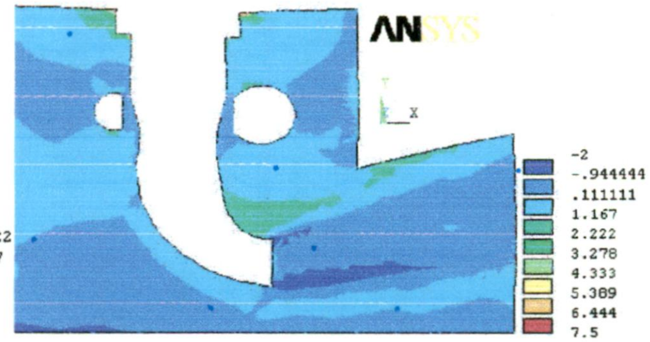
(e) Vertical Displacement Uy

Figure 4.80 Principal Stress S1, S3 and Vertical deflection U_y

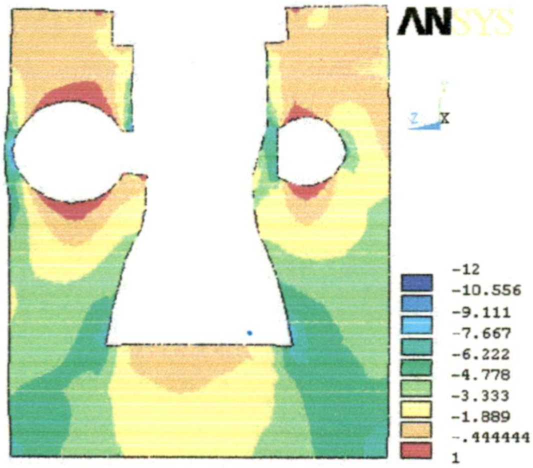
SUB STRUCTURE FIXED AT THE BASE
LOADS –Self weight, Rotor, Stator, Short Circuit



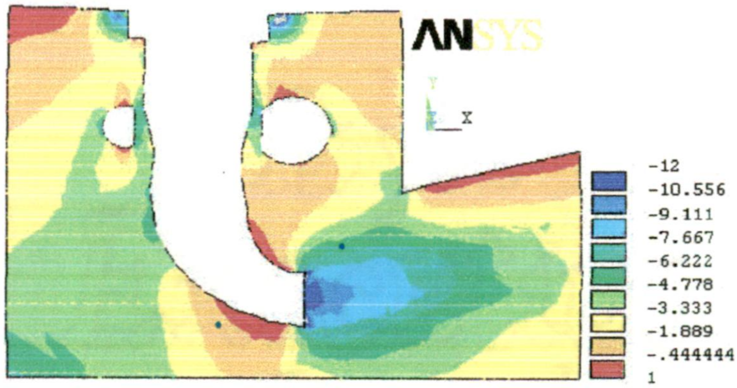
(a) Principal Stress S1



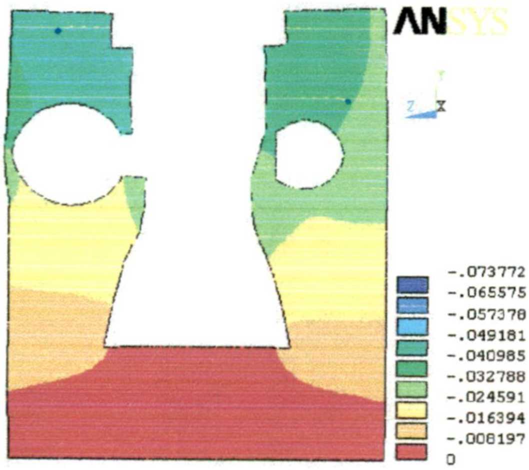
(b) Principal Stress S1



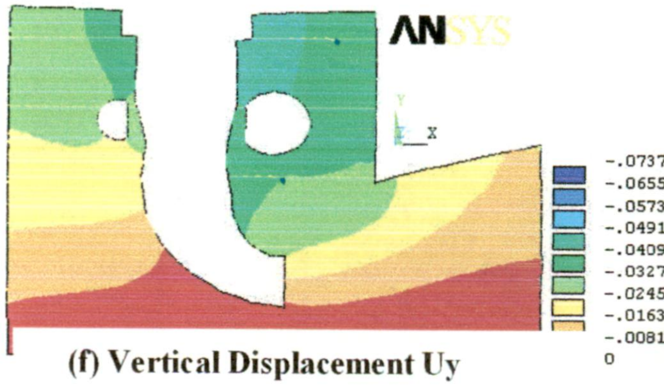
(c) Principal Stress S3



(d) Principal Stress S3



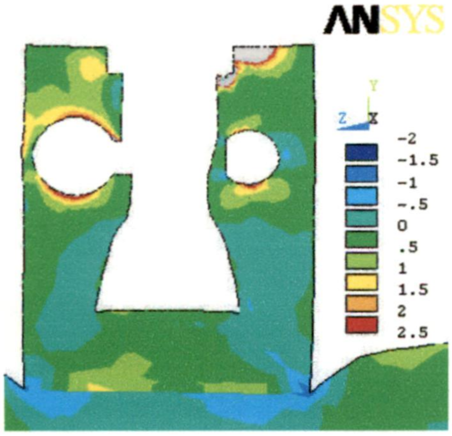
(e) Vertical Displacement Uy



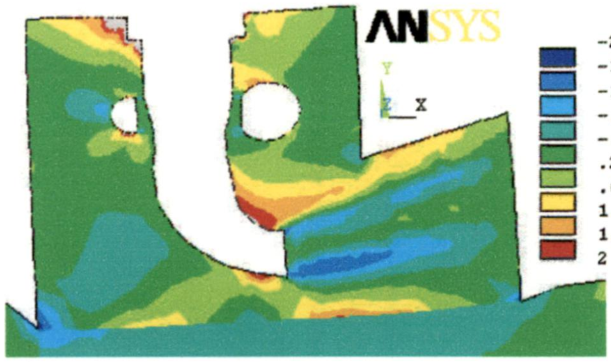
(f) Vertical Displacement Uy

Figure 4.81 Principal Stress S1, S3 and Vertical deflection U_y

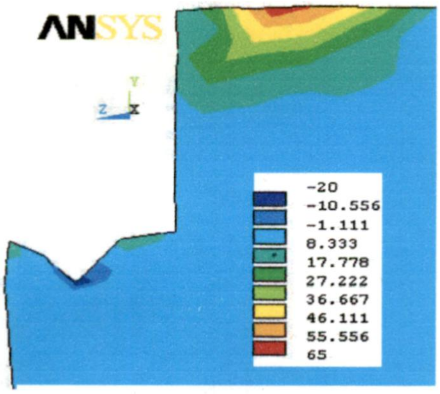
ENLARGED CONTOURS FOR STRESS,S1



(a) Principal Stress S1(Ec/10)

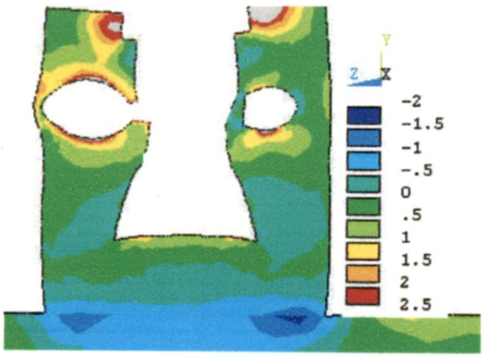


(b) Principal Stress S1 (Ec/10)

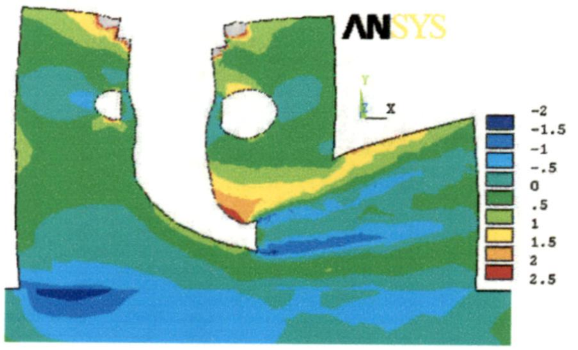


(b_1) Principal Stress S1 at brackets (E=Ec/10)

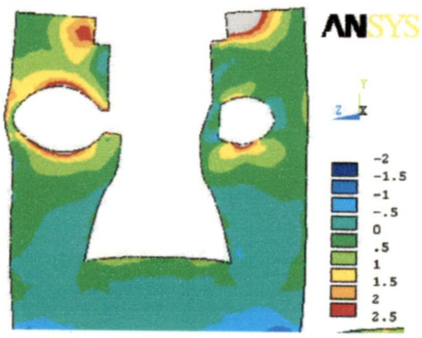
(a_1) Principal Stress S1 at left bracket (E=Ec/10)



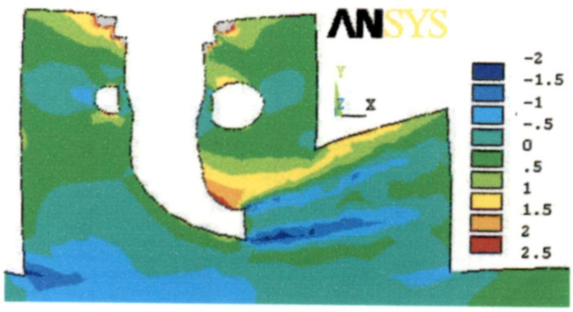
(c) Principal Stress S1 (E=Ec)



(c) Principal Stress S1 (E=Ec)



(e)Principal Stress S1 (E=10E)

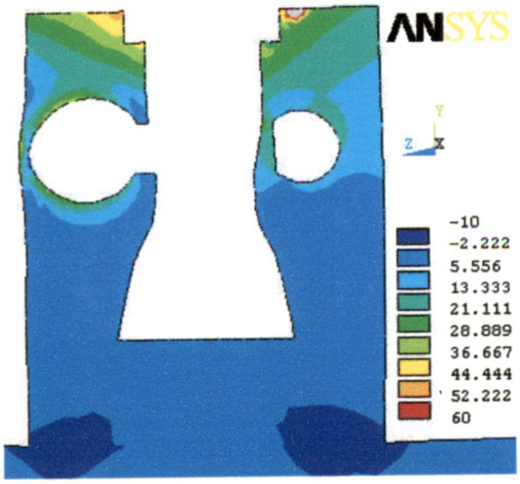


(f)Principal Stress S1 (E=10E)

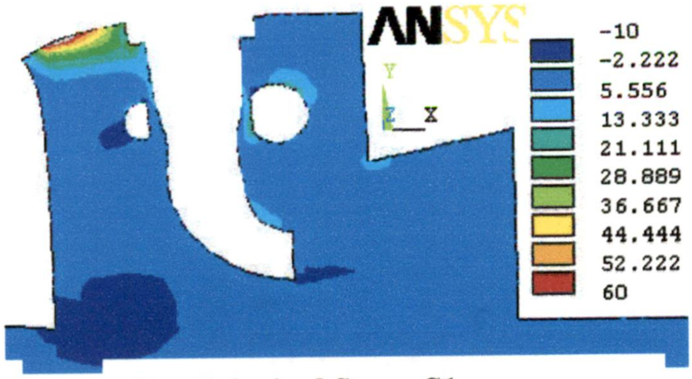
Figure 4.82 Principal Stress S1, S3 and Vertical deflection U_y

FOUNDATION (E=Ec)

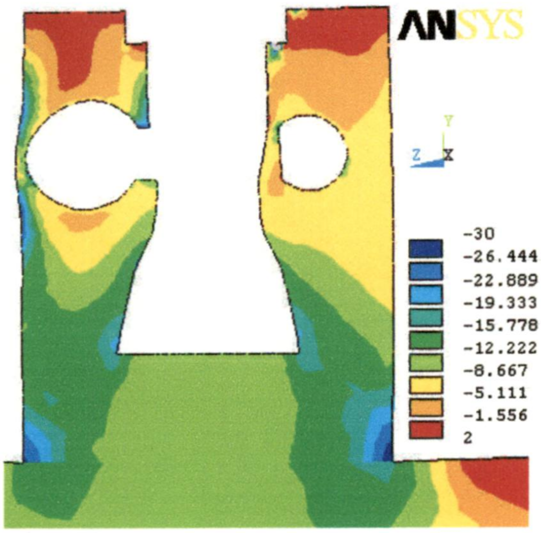
LOADS –Self weight ,Rotor, Stator, Short Circuit Torque, Water Pressure, Super structure



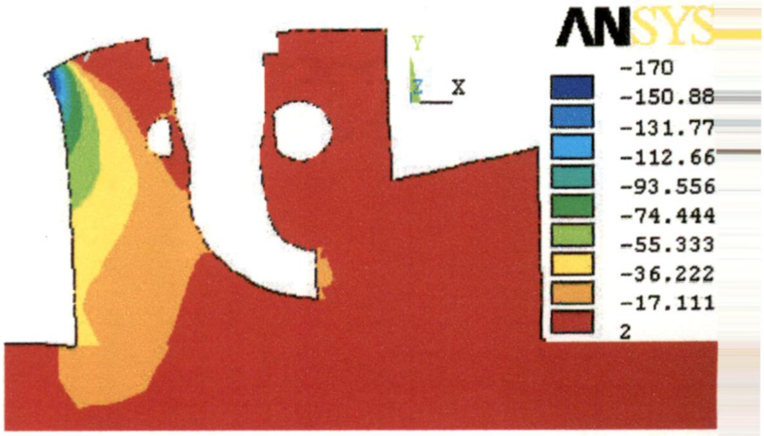
(a) Principal Stress S1



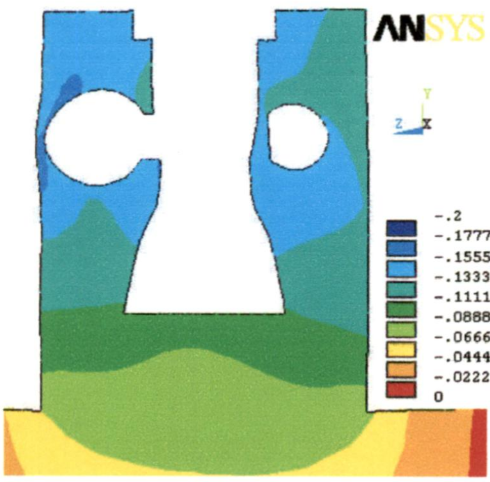
(b) Principal Stress S1



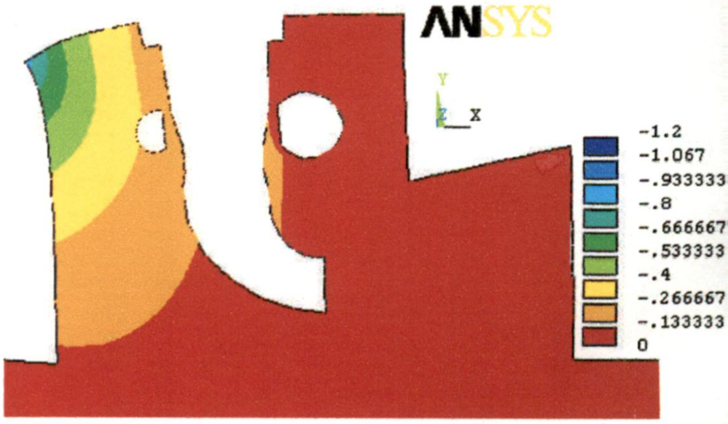
(c) Principal Stress S1



(d) Principal Stress S3



(e) Vertical Displacement Uy

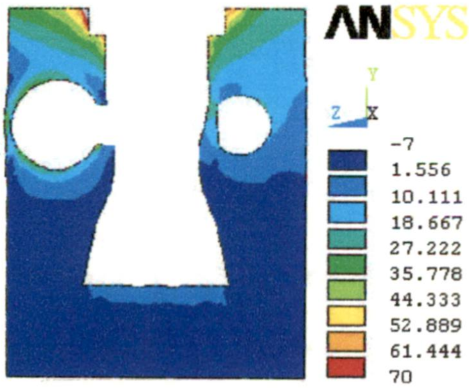


(f) Vertical Displacement Uy

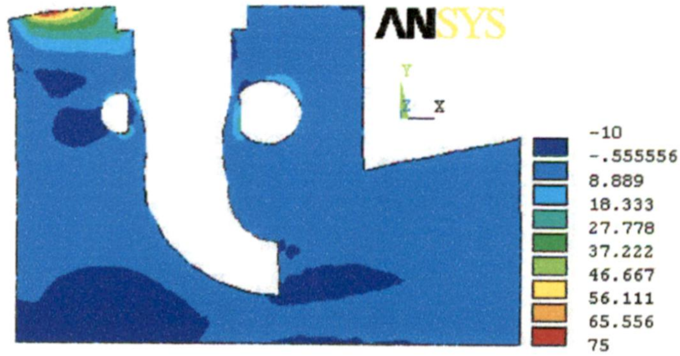
0

Figure 4.83 Principal Stress S1, S3 and Vertical deflection U_y

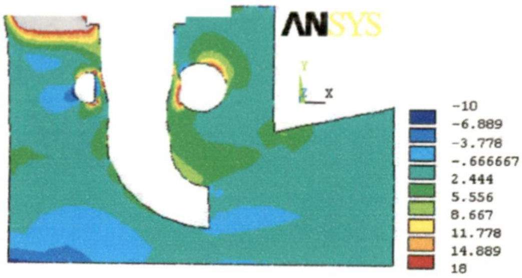
SUB STRUCTURE FIXED AT THE BASE
LOADS –Self weight, Rotor, Stator, Short Circuit Torque, Water Pressure, Super



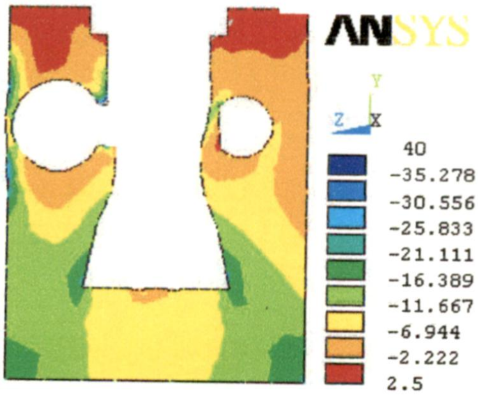
(a) Principal Stress S1



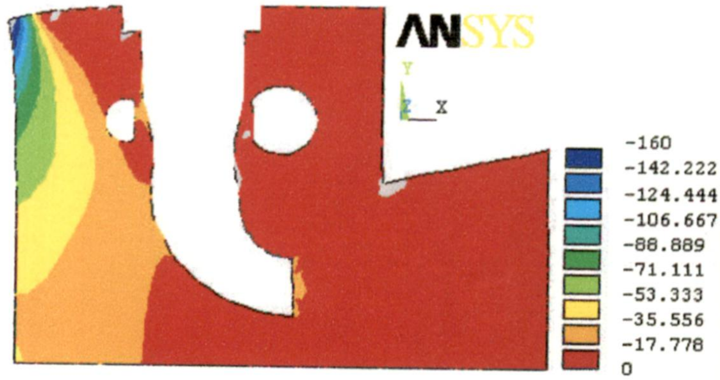
(b1) Principal Stress S1



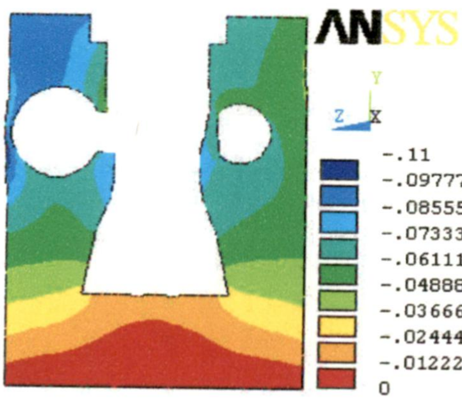
(b2)Principal Stress S1



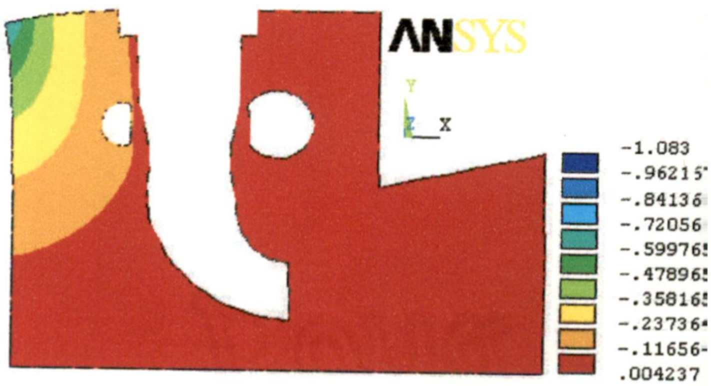
(c) Principal Stress S3



(d) Principal Stress S3



(e) Vertical Displacement Uy



(f) Vertical Displacement Uy

Figure 4.84 Principal Stress S1, S3 and Vertical deflection U_y

CONCLUSIONS AND SCOPE FOR FUTURE STUDY

5.1 GENERAL

Finite element analysis has been carried out to investigate the stress patterns and displacement of the combined substructure and intermediate structure considering these as a monolithic mass concrete of the Koteshwar power house, Uttaranchal using ANSYS software. The deflections and stress patterns obtained by FEM analysis for different loading conditions (Case A and B) and at various points of u/s and d/s sections of the draft tube have been worked out and presented in Chapter-4. Stress contours have also been prepared for the transverse and longitudinal sections passing through the turbine axis for the aforesaid loading conditions.

5.2 CONCLUSIONS

Based on the present study of power house structure, the following conclusions have emerged:

1. The study has revealed that the structure of the power house is essentially a low stressed stable structure. It was found that no where in the structure, except near the point of application of loads and around the openings of spiral case and draft tube, tensile and compressive stresses exceed the permissible tensile stress of the concrete (7 kg/cm^2) and. compressive stress (50 kg/cm^2) respectively.
2. The results (Table 4.1 and 4.2) reveal that top slab of the draft tube out side power house wall is significantly less stressed than top slab of the draft tube inside the power house. It is also observed from the

results that additional loads in case-B do not appreciably affect the results of case-A in the draft tube section out side the power house. Hence the concrete inside power house is more stressed and share loads as compared to the draft tube section out side the power house.

3. The compressive stress was observed to be more below the pier at the bottom of the draft tube openings than that of at the top of the pier at the top of the draft tube openings. This has been observed for both the u/s and d/s sections in all the foundation cases under consideration. This reveals that loads are transferred to the foundation through draft tube pier and side walls.
4. It was seen that with different values of rock modulus of foundation, structure does not show appreciable change in tensile and compressive stress. Pattern of the stresses remained practically same (Table 4.1). Above the bend of the draft tube, in the elbow portion and at some locations around the spiral casing, the value of tensile stress exceeded the permissible limit in concrete in load Case-B
5. The stresses and deformations of the structure with foundation having modulus of elasticity, $E=10\ E_C$ resembled the results of the structure fixed at the base, indicating that the structure may be assumed fixed with foundation rock when rock modulus is about ten times that of concrete.
6. Vertical deformation increased with the decrease in value of rock modulus in both the cases of loading. For the foundation condition $E=E_C/100$ the vertical displacement was found of the order of 1.82

cm. The bending of the structure about the Z-axis increased with the decrease in the value of rock modulus of the foundation. So for economical foundation design the rock modulus of foundation should be about $E_C/10$ or more.

The study has revealed that a rigorous analysis of the power house structure under static loads is not required. Nominal reinforcement is required in both directions around the openings such as spiral case and draft tube. The location of concentrated load application such as brackets supporting machine, gantry column foundation etc need special attention in design.

5.3 SCOPE FOR FUTURE STUDY

The substructure and intermediate structures with cavities and galleries need to be modelled as per actual shape, as the shape is complex. The stress analysis needs to be taken up for the study of influence of horizontal forces on the structure. Also, the power house structure need to be analyzed for dynamic loading conditions, taking earthquake forces into account.

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