3-D FINITE ELEMENT ANALYSIS OF PROPOSED ARCH CUM GRAVITY DAM AT LAKHWAR

A DISSERTATION

submitted in partial fulfilment of the
requirements for the award of the degree
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By

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CANDIDATE'S DECLARATION

I hereby certify that the dissertation entitled "3-D FINITE ELEMENT ANALYSIS OF ARCH CUM GRAVITY DAM AT LAKHWAR" in partial fulfilment of the requirements for the award of the Degree of Master of Engineering, submitted in the Department of Civil Engineering of the University, is an authentic record of my own work carried out since May 1992 under the supervision of Dr. Bhawani Singh, Professor of Civil Engineering and Dr. M.N. Viladkar, Reader in Civil Engineering, University of Roorkee, Roorkee, India.

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

Dated : November 26, 1992

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

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ABSTRACT

A 195 m high straight concrete gravity dam has been proposed at Lakhwar across river Yamuna in Distt. Dehradun of U.P. The dam foundation is very complex and consists of quartzitic slates, traps, thinly foliated slates. Besides, a 1.5 m thick shear zone exists about 70 m below the toe of the dam and traverses into the left abutment.

A 2-D FEM analysis for straight concrete gravity dam showed horizontal deflection of 33 cm at top and vertical settlement of 16.7 cm at the toe of the dam. The dam was, therefore, considered unsafe.

Alternatively, an Arch Cum Gravity concrete dam has been proposed. The present study of Arch Cum Gravity dam shows that deformation at top has now reduced to 7.85 cm. The maximum vertical settlement at toe is 5.02 cm. However, due to shear zone existing in the left abutment, deformations in the left abutment are much more as compared to the deformationsin the right. abutment.

The study shows that the deformations are within reasonable limits. But high tensile stresses along right bank and high compressive stresses along shear zone on left bank are developing. The tensile stresses can be taken care of by choosing suitable shape and curvature of the dam or alternatively, tying the right abutment by means of suitable anchors but the compressive stresses along the shear zone on left bank may force sliding along the shear zone. Thus, special treatment of left bank shall be needed.

A suitable layout alongwith the treatment of left bank will attribute to the suitability of an Arch Cum Gravity Dam at Lakhwar.

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NOTATIONS

ε	Strain vector
	Shear strain vector
σ	Stress vector
δ	Displacement vector
ξ	Natural Coordinate
η	Natural Coordinate
ζ	Natural Coordinate
A	Area
B	Matrix for partial derivatives
C_i, C_j, C_k	Gaussian weights
D	Elasticity matrix
F	Force vector
I	Identity matrix
J	Jacobian matrix
K	Stiffness matrix
N_i	Shape function
u	Displacement vector in x-direction
v	Displacement vector in y-direction
w	Displacement vector in z-direction

CHAPTER I

INTRODUCTION

1.1 GENERAL

Lakhwar Dam is proposed across river Yamuna about 100 km d/s of Yamunotri near village Lakhwar in Distt. Dehradun (Fig.1.1)It is a 195 meter high storage dam. It will impound 580 million m^3 of water and will provide irrigation to an additional area of 49,600 hectares and generate 475 million units of power at 90% water availability of the river runoff. The power house is located underground at the right bank with three units of 100 mw each with an additional provision of two units of 150 mw each.

1.2 SALIENT FEATURES

Some Salient features are as below :-

(i) Stream Flow

(a)Total catchment area	2080 Sq. km.
(b)Snow Catchment	130 Sq. km.
(c)Annual run off	2420 M. m^3
(d)Average annual run off	1882 M. m^3
(e)Maximum recorded flood	3300 cumec

(ii)Dam and Appertinent Works

(a)Top Elevation of dam	800 m
(b)Expected deepest foundation level	605 m
(c)Reservoir level (FRL)	796 m

(d)Dead Storage level	752 m
(e)Total capacity at FRL	580 M m^3
(f)Live Storage	333 M m^3
(g)Maximum design discharge of spillways	8000 cumecs
(h)Installed capacity of power house	300 mw
(i)Type of power house	Underground

1.3 ROLE OF ROCK MECHANICS

The construction of dam structure imparts pressure on the foundations resulting from the load of the structure and of the impounded water. The safety of the structure mainly depends upon :-

(i) Shear Strength of the foundation rock

(ii) Deformability of the foundation rock

(iii) Stability of the abutments

(iv) Seepage below the foundations and from reservoir.

The concrete dam is usually built on rock foundations. The rock mass behaviour is much different from that of soil. The rock mass contains joints, fissures, foliations, bedding planes, shears and faults and their dip determines the rock mass behaviour. If the foundations are not homogeneous, the different rock types having different strength and dipping of planes of weaknesses may make the dam structure unsafe. In all the circumstances, the rock mechanics solution has to be obtained. Thus, the rock mechanics plays a very important role in determining the safety of such large structures as dams.

The mechanism of rock behaviour is dependent upon its mechanical properties which determines the types of dam which will be suitable on a particular location. The types of dam may be a Gravity or an Arch, an Arch cum Gravity or an earth dam. Due to inaccessibility, a large number of features existing in the rock mass remain unknown which has caused many dam failures in the past. This further necessitates the importance of rock parameters used in the dam design .

1.4 NEED FOR NEW TYPE OF DAM

A straight gravity concrete dam at Lakhwar has been proposed and the analysis of the dam and foundation has been carried out by,

(i) analytical methods

(ii) 2-D FEM .

It has been found that although dam is safe against sliding and overturning, there are large deformations at the base and top of the dam. Consequently, there will be operational difficulties and the dam may become inoperational. Therefore, comprehensive foundation treatment is needed for the safety of the dam. Instead of straight gravity dam, possibility of other types of dam is also to be explored at the present location.

1.5 CONCEPT OF ARCH CUM GRAVITY DAM

A Gravity Dam is analysed by taking into consideration that all loads are transferred to the foundations below the dam base. Whereas, in Arch dam load transfer takes place due to arching action of the dam. Thus, substantial transfer of load takes place through the abutments and a narrow section of the dam does generally suffice. Thus, where abutments are strong enough, an Arch dam is proposed as the reduced section of dam results in substantial cost savings. In cases, where foundation rock below the dam is not strong enough and abutments as well do not have strong supports, an Arch cum Gravity dam is best suited. An Arch cum Gravity dam acts in both ways. It transfers proportionate loads to the foundations below the base and also the abutments and is, thus, a good compromise between Arch and Gravity dams.

1.6 PROBLEM DEFINITION

As referred in article 1.4 above, the 2-D FEM analysis of the Gravity Section at Lakhwar gives large deformations at the top and base of the dam which will give rise to operational difficulties and the dam may become inoperational. The left abutment at Lakhwar is also supposed to be not strong enough. Therefore, possibility of an Arch dam is also reduced. In such a situation, there may be following alternatives to the Gravity dam

(i). An Arch cum Gravity dam

(ii). A Hollow dam

(iii). An Anchored dam

In all the above alternatives an Arch cum Gravity dam appears to be the most suitable and, therefore, analysis of an Arch cum Gravity dam has been undertaken.

CHAPTER II

LITERATURE REVIEW

2.1 GENERAL

Many large dams have been built in the past in many parts of the world over complex foundations containing shear zones and faults. The problem has been tackled with great engineering skill. Even the weak foundations have to be suitably treated to withstand the loads from dam and reservoir. With the advent of computers, the technique of numerical analysis has changed the complete scenario. The minutest detail can now be analysed within no time which may be a critical component for the safety of the dam. Thus, the analysis and design for almost all dams is carried out by finite element method which gives true picture of stresses and strains in each part of the dam.

2.2 Literature Review

The rock mass in the foundation contains many discontinuities. These discontinuities can be modelled and their effect can be calculated by carrying out the finite elemnt analysis. The safety of dam depends upon the extent of stresses developed and the deformations experienced by the dam and the foundation.

Grishim, (1982)in "Hydraulic Structures Vol I has defined three limiting states for safety of dams,viz

(i)- when the structure or its foundation satisfies no longer the specified operational conditions which means that the values of loads and stresses which develop in the structure or its foundation must not exceed the load resisting capacity of the structure.

(ii)- The magnitude of deformations in the foundation and displacements occuring in the structure must not exceed the allowable value for the normal operation of the structure.

(iii) Large cracks should not appear in concrete or least their size should be such that normal operating conditions will not be disrupted.

USBR manual(1976)"Design of gravity Dams" and "Design of Arch Dams" and I.S. code (1984) have described the various load conditions and the safety factors for the dams. The dams are essentially tested against sliding , factor of safety against sliding being of prime importance.

Various authors have opined on the factor of safety. Mary (1964) noted that the factor of safety should be based on the danger of rupture arising from internal stresses in the rock. Lane (1964) thought that the determination of factor of safety would consist of finding a key zone of weaker material which always exists somewhere in the foundation rock and checking the safety of sliding of that zone. Serafim (1964) emphasized the importance of determining the geological picture before embarking on the calculation of the factor of safety.

The factor of safety against sliding of the dam, the foundation and the abutments is first and most paramount aspect of dam design. Unfortunately, this is an aspect on which finite element analysis provides little information. The analysis of stability includes simple parameters like cohesion and angle of friction which can be obtained through insitu shear tests. The main difficulty in modelling the sliding behaviour in finite element analysis is of accounting for the strain jump which takes place across the sliding plane.

Zienkiewicz (1979) use a stress transfer approach along the joint shear stresses in excess of those permitted by Mohr-Columb law.

Pande et al (1975) have useda viscoplastic approach in which stresses in the joint element can exceed those dictated by Mohr-Coulomb law instanteneously but are released by prescribing suitable strain.

Rickets (1975)uses the joint elements prescribing a yield law on the joint plane in 3-D analysis.

However, any of these models can not be stated to have met unqualified success.

Mgalobelov(1979), in his paper "Strength and Stability of Arch Dam Foundations", has given the criteria for stability as,

$$\text{factor of safety} = \frac{(G+H)\ \tan\phi + C.S}{F} \qquad (2.1)$$

Where G, H, F,C and S are net gravity load, thurst on abutments and horizontal force, cohesion and surface area respectively.

2.2.1 Gravity Dam

Gravity dams are the first ones which have successfully been constructed in the early stages. The design of these dams have been carried out analytically or using 2-D finite element methods. A 3-D analysis is usually not done, as in gravity dams, it is often assumed that vertical joints are grouted lastly and shear action of keys is not taken into account to warrant a 3-D finite element analysis which is 10 to 15 times costlier and time consuming. However, behaviour of dams has been checked by 3-D finite element analysis in few cases.

Finite Element studies for Busalletta Dam have been carried out by Haws, Furley and Zytynski.(1975). The dam located in Italy is 52 m high concrete gravity dam. The foundation rocks consist of black shales with interweaving weak layers. Also, there is 12 m wide fault zone. The sliding stability along this fault zone has been studied. The stresses were calculated by FEM and using Mohr-Coulomb relationship, its stability has been checked.

Dehousse and Diab(1975) in their paper "Analysis of the stresses due to interstitial water in Gravity dams and in their rock foundations" have shown in their finite element analysis that uplift is but a particular manifestation of the effect of pore pressures. The presence of interstitial water in rock mass influences both the state of stress and strain as well as the ultimate strength.

Bourbonnais and Morgenstern (1975) in "An analysis of the deformation of three dam foundation" have carried out finite element analysis of Krasnoyarsk, Alpa Gera and Bhakra Dams to assess the correctness of the values of modulus of deformation. The analysis predicted a settlement of 50 mm, 14 mm.and 31 mm as compared to observed value of 19.4 mm, 25 mm and 4 mm respectively. It was concluded that the behaviour of the actual foundation is better than predicted.

2.2.2 Arch Dams

The behaviour of Arch dams is different from that of Gravity dams. The design of appropriate foundations is the critical point in case of arch dams. The geomechanical stability analysis of the rock portions and wedges all round the abutments must be critically examined. This needs local distribution of forces and stresses, for which analytical methods and model testing are not sufficient. Thus, for the analysis of arch dam, a 3-D finite element analysis is essential.

Milovanovic (1987)in his paper "Appropriate foundations for Arch dams" has concluded , on the basis of finite element analysis, that change of rock deformation modulus from 5000 to 1000 MPa does not cause any considerable difference to the principal compressive stresses in the arch dam boundary. However, the principal tensile stresses, for the deformation modulus of 1000 MPa, are very much higher and the crack exists through the entire thickness.

Lida and Shibata(1987) have carried out finite element analysis for Yahagi and Kawaji Dams which have weak zone in the foundation. It has been concluded that it is important to select an arch of small angle and that the parabolic arch behaves better than the circular arch. The authors have also emphasised that it is very difficult to improve on the creation of nature such as foundation rock while it is not so difficult to improve the structures constructed artificially. Thus, the best way to design a dam is to adapt the dam body to the foundation rock.

Dungar(1985) in his analysis of Zervreila Dam has studied cracking of the grouted vertical joints on the left flank which had weak rock joints. The following elasto-plastic discontinuous rock model using empirical equation developed by Barton (1973) was used to explain the opening of joints in drawdown condition due to nonlinear foundation deformation .

$$\tau = \sigma_n \cdot \tan\left(\phi_r + JRc.\log\frac{JCs}{\sigma_n}\right) \qquad (2.2)$$

2.2.3 Arch Cum Gravity Dam

The arch cum gravity dams are most suitable where sharing of load between the foundation and the abutments is desirable.

A few Arch cum Gravity Dams have been constructed in the different regions of the world. Piccinelli et al(1987) have carried out 3-D finite element analysis for Ridracoli Dam which is adouble curvature arch gravity dam. The base width to height ratio adopted is Ø.3. The horizontal displacement at top was calculated as 3Ø mm.

Tover(1987) studied the design and construction of

Escalora Dam in Spain for three different types of dam - an embankment dam,a rollcrete dam and an arch gravity dam. In the study of arch gravity dam which had a base width to height ratio of Ø.4, 3-D finite element analysis was carried out using 344 elements with eight nodes. The results showed a tensile zone in the heel which can be marginalised in construction by means of a perimeter and longitudinal water stop joint, shaped like an arch. It was found that the structural behaviour of the arch-gravity dam is practically the same as that of a thick arch.

2.3 Comments

From the above discussion, it is apparent that an Arch cum Gravity dam is best suited where the foundations alone below the dam as well as the abutments alone are not strong enough to carry exclusively all loads due to dam construction. The finite element analysis does not provide directly any information regarding stability of the whole dam structure against sliding which is also much important. The weak zones can not be fully treated and a suitable layout and shape of the dam must be determined which will not only give safe behaviour but also economise the dam construction. The study of weak layers of rock such as shear zones or faults need special attention. If the shear stresses along the weak zone are large, there may be permanent strains and an elasto- plastic or visco-plastic analysis may be required. The material behaviour is also non linear and a non linear elastic analysis is desirable.

2.4 Justification of Problem

At Lakhwar, foundations are very complex. The rocks have different modulii of deformation and a shear zone, lying below the toe of the dam, traverses through the left flank. Thus, the foundations are not capable of taking full loads from Gravity dam section and the left abutment may not be suitable to carry thurst due to an Arch dam. In such a situaton, an Arch Cum Gravity Dam seems a viable solution. Keeping in view the present limitations, a linear elastic analysis for Arch cum Gravity Dam is desirable.

CHAPTER III

GEOLOGICAL FEATURES AND ROCK MASS CLASSIFICATION

3.1 ROCK TYPES

At the Lakhwar Dam site the river Yamuna flows through a narrow gorge in east-west direction and forms a U-shaped loop(Fig.3.1) The rock formations in the dam area comprise of phyllites, slates, quartzites and lime stones belonging to the Mandhali, Chandpur and Nagthat series(Fig.3.2). These have been folded into a major syncline named as 'Jaunsar Syncline'. The phyllites are intruded by a basic rock, petrological composition of which varies from dolerites to Hornblende granites. These have been named as "Jaunsar Traps". One of these is lenticular in shape along the trend of phyllites and cuts across Yamuna. This forms the foundations of the Lakhwar dam.

3.2 TRAPS

The traps are coarse grained and highly jointed and show some slicken-siding on joint faces. It shows that some movement has taken place within this trap even after its consolidation. The maximum width of the trap is 300 m which is just sufficient to accomodate the dam and major part of spillways(Fig.3.2). The gorge is narrow at the proposed dam site(Fig.3.2). The abutments are rising steeply above the river bed. The left abutment spur is narrower in width with steep hill slopes upto dam height and then a gentle rising profile. Besides the narrowness of the left abutment, the joints in the trap rock are dipping at low angles in the downstream and, thus, there isa possibility of the daylighting of the resultant stresses due to dam loading along the joints of the trap.

3.3 SHEAR ZONE

The foundation investigations by drill holes and drifts have indicated a 1.5 m wide shear zone at the contact of trap with d/s slates(Fig.3.2). The shear zone is filled with gougy material and, lies about 70 m below the d/s toe of the dam. After crossing the river, it traverses deep into left abutment.

3.4 SLATES

The slates, in the d/s of the shear zone(Fig.3.2), are thinly foliated and are very weak in nature. The foliations are dipping at 70 - 80 in the upstream direction.

3.5 XENOLITH

On the left abutment, an outcrop of slates and quartzites is enclosed on three sides by trap (Fig.3.1)and appears to be a caught up mass of the country rock. This has been termed as "Xenolith".

The complete geology of the area is shown in Figs.3.1,3.2 and 3.3. The geology of the area is, thus, very complex.

3.6 ROCK MASS CLASSIFICATION

3.6.1 Geomechanical Classification

Bieniawski's geomechanical classification gives Rock Mass Rating (RMR) which depends on following parameters -

(i) Uniaxial compressive strength
(ii) Rock quality designation (RQD)
(iii) Spacing of discontinuity
(iv) Condition of discontinuity
(v) Ground water condition
(vi) Orientation of discontinuity

Thus for trap rock, RMR, has been worked out as below -

	As per CSIR	As per IS code
Rock Strength	12	12
RQD	17	17
Spacing of joints	10	10
Condition of joints	20	20
Ground water condition	7	10
	66	69
Less for orientation	(-) 2	(-) 2
	64	67

Average RMR = 65

3.6.2 N G I Classification

The rock mass quality as per NGI classification has, also, been assessed.The rock mass quality, Q is given by,

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \qquad (3.1)$$

where RQD = Rock quality designation

J_n = Joint set number

J_r = Roughness of most unfavourable joint

J_a = degree of alteration

J_w = Joint water reduction factor

SRF = Stress reduction factor

The trap rock contains three prominent sets of joints. The joint walls are unaltered and joints are rough and undulating and, therefore, adopting following values of various parameters, the Q value has been found out. :

RQD = 75 % to 90 % (Avg. RQD = 82.5 %)

J_n = 12

J_r = 3

J_a = 1.0

J_w = 1.0

SRF = 2.5

Therefore, $Q = \frac{82}{12} \times \frac{3}{1} \times \frac{1}{2.5}$

$= 8.25$

3.6.3 Correlation Between RMR and Q

Corresponding to a particular value of Q, RMR can be evaluated from various correlations as detailed in Table 3.1

Table 3.1 Values of RMR for trap rock Worked out from Q

Sl. No.	Correlation for RMR	Value of		Author (year)
		Q	RMR	
1.	9 lnQ + 44	8.25	62.99	Bieniawski (1974)
2.	5.9 lnQ + 43	8.25	55.45	Rutledge and Preston (1978)
3.	5.4 lnQ + 55.2	8.25	66.59	Moreno (1980)
4.	5 lnQ + 60.80	8.25	71.35	Cameron Clark Budarari (1981)
5.	10.5 lnQ + 41.8	8.25	63.95	Abad et al (1983)

Thus a value of RMR = 65 for trap rock obtained in art. 3.6.1 seems reasonable. The RMR values for other rocks have been obtained and are as -

RMR For quartizitic rock 40

RMR For slates 25

CHAPTER IV

ROCK STRENGTH PARAMETERS AND ROCK ANISTROPY

4.1 GENERAL

As indicated in chapter II, the foundation rock consists of quartzitic slates, traps, thinly foliated slates and a shear zone. For the analysisof dam, strength parameters such as compressive , tensile, shear strengths and modulus of deformation are required to assess the rock load bearing capacity and the deformations which the rock will suffer due to dam construction.

Wide range of tests have been conducted to assertain the strength parameters and modulus of deformation for various rock types encountered in the foundations. The data regarding the tests conducted for obtaining the value of deformation modulus are presented in-

Appendix-I	for Trap rock
Appendix-II	for Thinly foliated slates and
Appendix-III	for Quartzitic slates

The data regarding the shear parameters for tests conducted are presented in-

Appendix-IV	for Trap rock-Concrete,

Appendix-V	for Trap rock alone,
Appendix-VI	for Xenolith rock- Concrete
Appendix-VII	for Xenolith alone,
Appendix-VIII	for Slate rock - Concrete
Appendix-IX	for Quartzitic Slate - Concrete,
Appendix-X	for the Slate alone
Appendix-XI	for tests on Quartzitic Slate alone
Appendix-XII	gives the triaxial compression test data for tests conducted on Slates.

All this data (Appendix -I to XI) have been obtained by the U.P. Irrigation Design Organisation, Roorkee on the basis of field tests conducted at the dam site and presented here due to its courtesy.

4.2 STRENGTH PARAMETERS

4.2.1 Trap Rock

On the basis of above tests, following strength values for trap have been found :

(i) Average Compressive strength — 980 kg/cm^2

(ii) Average tensile strength — 110 kg/cm^2

(iii) Average Shear Strength

(a) Friction Angle

Rock to Rock — 48°

Rock to concrete — 48.5°

(b) Cohesion

Rock to Rock — 5.70 kg/cm^2

Rock to concrete 2.0 kg/cm^2

(v) Poissons Ratio 0.235

4.2.2 Thinly Foliated Slates

The triaxial tests on thinly foliated slates carried out in the Geotechnical Engineering Laboratory of UOR, Roorkee (Appendix XII) shows that slates are unable to bear high compressive stresses. They slates softens at a deviatoric stress of 128 kg/cm^2.

4.2.3 Gouge Material

Lab tests on gouge material indicate that gouge material is ML-CL (Silty clay). The strength can be described by coulomb's law. Ladyani and Archambault (1977) carried out large number of tests and reached the conclusion that the stiffness and shear strength of a filled discontinuity decrease with increasing infilling thickness but always remain higher than those of the filling alone.

4.3 MODULUS OF DEFORMATION

The load deformation property of the rockmass is governed by modulus of deformation. Since the rock mass contains micro cracks and other discontinuities such as bedding planes, a laboratory tests carried out on a small sample can not be the true representative of the in situ value. Therefore, large number of insitu cyclic tests are carried out to determine the value of modulus of deformation. The simplest tests are plate jacking test and the flat jack tests with bore hole extensometer.

Various authors have used the cyclic tests data in different ways. Some consider that it is the elastic modulus which is important. Some consider total deformation modulus as important. Whichever may be the criteria, it is generally accepted that the ratio of elastic to total modulus is important as it determines how the foundations can behave.

A large number of plate jacking, flat jack tests with bore hole extensometers have been carried out to determine the value of modulus of deformation which is to be considered for design purposes. The deformation modulus has been calculated for the second cycle and based on total deformation. The results indicate that the values for the second cycle vary as follows :

(i) For trap, E_d = 4,688 to 3,06,250 kg/cm^2

(ii) For quartzitic slates, E_d = 6,730 to 40,920 kg/cm^2

(iii) For thinly foliated slates E_d = 1,200 to 14,600 kg/cm^2

Thus there is large scatter in the value of modulus of deformation and it is very difficult to adopt a reasonable value. Bieniawski(1978) in his paper "Determining Rock Mass Deformability, Experience From Case History" has suggested that the value should be correlated with that of RMR. The various correlations for RMR and modulus of deformation (E_d) give following value of E_d as listed in Table-4.1.

Table-4.1 Deformation Modulous (E_d) as obtained from various correletations with RMR

Sl. No.	Author (year)	Correlation E_d (in GPA)	Rock Type	RMR	E_d (Kg/ Sq cm)x10^5
1.	Bieniawski (1978)	RMR-100	Trap	65	3.0
2.	Serafim and Pereira (1983)	$10^{\frac{(RMR-10)}{40}}$	Trap	65	2.37
			Quartzitic Slates	40	0.562
			Slates	20	0.287
3.	Mehrotra * (1992)	$10^{\frac{(RMR-25)}{40}}$	Trap	65	1.0
			Quartzitic Slates	40	0.237
			Slates	20	0.10

* yet to be published

Bieniawski's(1978) relationship is linear and is not justifiable for lower Himalayas where the rocks are of recent origin. Relationship by Serafim and Pereira(1983) is generally applicable for RMR less than 50. Mehrotra (1992) after a large number of field observations has obtained the relationship especially for lower Himalayas. It is, therefore, more practical and appropriate.

Based on above discussion, it is reasonable to adopt following values of modulus of deformation (E_d) -

(i) For trap, E_d = 1,00,000 kg/cm^2

(ii) For quartzitic slates E_d = 25,000 kg/cm^2

(iii) For Slates $E_d = 10,000 \ kg/cm^2$

The laboratory tests on gouge material of shear zone indicate that the minimum value of modulus of deformation for gouge material lies in between 125 to 450 kg/cm^2. It is the minimum value which has been calculated from initial void ratio and compressibility of the material. A state of high insitu stress exists below the foundations. This alongwith the fact that the stiffness of the filled discontinuity is always higher than the fill material , a higher value of 2000 kg/cm^2 is adopted for design calculations for the shear zone.

4.4 ROCK ANISTROPY

The slates are thinly foliated. The foliations are dipping at 70° - 80° in the u/s direction. The behaviour of the slates is, therefore, perfectly anistropic. The peak strength of anistropic rocks varies with the orientation of the plane of isotropy or foliation plane with respect to the principal stress direction.

The shear strength across the discontinuity is high and approaches that of intact rock but shear strength along discontinuity is low and approaches the shear strength of the material which fills the discontinuity.

Jaegar and Cook (1979) developed a theory to predict the strength of rock containing a single plane of weakness. It assumes failure by sliding along the plane of weakness and is given by -

$$(\sigma_1 - \sigma_3) = \frac{2C + 2\sigma_3 \tan\phi}{(1 - \tan\phi \cot\phi) \sin 2\beta} \qquad (4.1)$$

where, c = cohesion

ϕ = angle of friction

β = angle of inclination of the plane of weakness with the major stress direction.

Minimum strength is given , when

$\tan 2\beta = - \cot\phi$

Since c and ϕ vary with the inclination of the plane of weakness, it is, therefore, difficult to use the above expression.

Ramamurthy et al (1980) have suggested the following strength criteria

$$\sigma_{ij} = A - D [\cos 2(\beta m - \beta)] \qquad (4.2)$$

where A & D are constants

and βm is the orientation angle corresponding to minimum value of σ_c.

By carrying out three uniaxial compression tests at values of β equal to 0°, 30° and 90° and two triaxial tests at β=90°, values of A, D and β_m can be evaluated.

On the basis of expression given by Jaegar & Cook,(1979) the slate strength is lowest at orientation angle of 45° + $\frac{\phi}{2}$. It reaches a peak value when the plane of weakness is oriented at 90° or lies between zero to ϕ° to the direction of major principal stress (Fig 4.1). The failure may also occur due to shear fracture through the rock material depending upon the extent of the stresses transferred to the shear zone.

CHAPTER V

THEORETICAL ANALYSIS

5.1 FINITE ELEMENT METHOD

Various techniques are available for designing of the dam and its foundations. The techniques usually adopted are -

(i) Analytical approach

(ii) Numerical approach

The analytical approach does not provide true picture of the foundation behaviour whereas in the numerical approach, Finite Element Analysis gives true picture of the behaviour but it has its limitations too. The FE Analysis is preferred due to following reasons :

(i) It provides the information regarding deformations at various points in the dam body and foundations which are essentially required for the design of dam.

(ii) The analytical approach assumes linear variation of normal stress at the base of the dam and therefore does not distinguish if the foundation rocks are stiff or soft. There is more concentration of stresses at the heel and toe in case of soft rock which is depicted in the finite element analysis.

5.2 ELEMENT FORMULATION

The formulation involves following steps -

(i) Discretization of the continuum - The continuum is discretised into elements which may belong to any of the families - Simplex, Langrangian or Serendipity and may be linear, quadratic, cubic in form etc. Besides, isoparametric elements are generally adopted which have the same parameters and same order for the geometry and the displacement. The isoparametric elements are most efficient and even the curved and arbitrarily shaped elements provide efficient differentiation and integration.

(ii) Description of geometry of the element -

The geometry of the element is defined in terms of shape functions, N_i which are chosen in such a way that it guarantees continuity of the function between elements and is able to produce a constant strain condition throughout the element. The geometry is, thus, defined as

$$x = \Sigma N_i x_i$$
$$y = \Sigma N_i y_i$$
$$z = \Sigma N_i z_i$$
$$\text{and volume } v = \Sigma N_i v_i \tag{5.1}$$

The shape functions are represented in terms of natural coordinates, ξ, η, ζ and are given for a quadritic element as below (Fig. 5.1),-

(a) <u>At corner Nodes</u>

$$N_i = \frac{1}{8}(1 + \xi\xi_i)(1 + \eta\eta_i)(1 + \zeta\zeta_i)(\xi\xi_i + \eta\eta_i + \zeta\zeta_i - 2) \tag{5.2}$$

(b) <u>At mid side nodes</u>

For N_2, N_6, N_8, N_{14}

$$N_i = \frac{1}{4} (1 - \eta^2) (1 \pm \xi\xi_i) (1 \pm \zeta\zeta_i) \qquad (5.3)$$

For N_4, N_8, N_{16}, N_{20}

$$N_i = \frac{1}{4} (1 - \xi^2) (1 \pm \eta\eta_i) (1 \pm \zeta\zeta_i) \qquad (5.4)$$

For N_9, N_{10}, N_{11}, N_{12}

$$N_i = \frac{1}{4} (1 - \zeta^2) (1 \pm \xi\xi_i) (1 \pm \eta\eta_i) \qquad (5.5)$$

(iii) Variation of unknown displacement function - This is also given in terms of the shape functions as below :

$$\begin{Bmatrix} u \\ v \\ w \end{Bmatrix} = \left[I_1N_1 + I_2N_2 + I_3N_3 \right] \begin{Bmatrix} u_1 \\ v_1 \\ w_1 \\ u_2 \\ v_2 \\ w_2 \\ u_3 \\ v_3 \\ w_3 \end{Bmatrix} \qquad (5.6)$$

(iv) Strain-Displacement Relationship -

The strain vector in 3-D is defined as

$$\{\epsilon\} = \begin{Bmatrix} \epsilon_x \\ \epsilon_y \\ \epsilon_z \\ \gamma_{xy} \\ \gamma_{yz} \\ \gamma_{zx} \end{Bmatrix} = \begin{Bmatrix} \dfrac{\partial u}{} \\ \dfrac{\partial v}{\partial y} \\ \dfrac{\partial w}{\partial z} \\ \dfrac{\partial u}{\partial y} + \dfrac{\partial v}{\partial x} \\ \dfrac{\partial v}{\partial z} + \dfrac{\partial w}{\partial y} \\ \dfrac{\partial w}{\partial x} + \dfrac{\partial u}{\partial z} \end{Bmatrix} \qquad (5.7)$$

The strain - displacement relationship is defined as,

$$\{\epsilon\}^e = [B]\ \{\delta\}^e \tag{5.8}$$

where

$$[B\ ] = \begin{bmatrix} \frac{\partial N_i}{\partial x} & 0 & 0 \\ 0 & \frac{\partial N_i}{\partial y} & 0 \\ 0 & 0 & \frac{\partial N_i}{\partial z} \\ \frac{\partial N_i}{\partial y} & \frac{\partial N_i}{\partial x} & 0 \\ 0 & \frac{\partial N_i}{\partial z} & \frac{\partial N_i}{\partial y} \\ \frac{\partial N_i}{\partial z} & 0 & \frac{\partial N_i}{\partial x} \end{bmatrix} \tag{5.8a}$$

(v) Stress - Strain Relationship

For linear variation of stress - strain relationship

$$\sigma = E\ (\epsilon - \epsilon_0) + \sigma_0 \tag{5.9}$$

or $$\{\sigma\}^e = [D]\ \{\epsilon\}^e \tag{5.9a}$$

where [D] is elasticity matrix

For plane strain condition, matrix [D] is given as,

$$[D] = \begin{bmatrix} 1-\nu & \nu & \nu & 0 & 0 & 0 \\ \nu & 1-\nu & \nu & 0 & 0 & 0 \\ 0 & \nu & 1-\nu & 0 & 0 & \\ 0 & 0 & & (1-2\nu)/2 & 0 & 0 \\ 0 & 0 & 0 & 0 & (1-2\nu)/2 & 0 \\ 0 & 0 & 0 & 0 & 0 & (1-2\nu)/2 \end{bmatrix} \tag{5.9b}$$

(vi) Equilibrium Equation

Principle of Virtual Work is applied to obtain the equilibrium equation. Thus we have,

External virtual work = Internal virtual work or strain energy

Applying the principle, we get

$$[\Delta\delta^T]\ \{F\} = \sum_{e=1}^{n} \int_e \Delta\epsilon.\sigma.\ A\ ds$$

$$\text{or } F = \Sigma_{e=1}^{n}\ \int_V [B]^T\ [D]\ [B]\ dv.\ \{\delta\} \qquad (5.10)$$

Thus, for an element, we get

$$\{F\}^e = \int_V [B]^T [D]\ [B]\ dv\ \{\delta\}^e \qquad (5.10a)$$

$$\text{or} \quad \{F\}^e = [K]^e \{\delta\}^e \qquad (5.10b)$$

$$\text{where, } [K]^e = \int_V [B]^T [D]\ [B]\ dv \qquad (5.10c)$$

Where $[K]^e$ is stiffness matrix.

5.3 TRANSFORMATIONS

The shape and displacement of the elements are defined in terms of shape functions which are given in natural coordinates. Thus, for assemblege of entire elements load, transformation of shape functions in global coordinates is desired. The equilibrium equation at the element level is given by,

$$\{F\}^e = [K]^e \{\delta\}^e \tag{5.11}$$

$$= \int_V [B]^T [D] [B] \, dv. \{\delta\}^e \tag{5.11a}$$

The strain-displacement matrix, [B] is defined in terms of derivatives of Ni with respect to global coordinates. The shape functions, Ni are written in terms of the natural coordinates ξ, η, ζ. Thus partial derivatives of Ni with respect to the natural coordinates can be defined as,

$$\frac{\partial N_i}{\partial \xi} = \frac{\partial N_i}{\partial x}\frac{\partial x}{\partial \xi} + \frac{\partial N_i}{\partial y}\frac{\partial y}{\partial \xi} + \frac{\partial N_i}{\partial z}\frac{\partial z}{\partial \xi} \tag{5.12}$$

Similarly, other derivatives $\frac{\partial N_i}{\partial \eta}$ and $\frac{\partial N_i}{\partial \zeta}$ can be expressed.

Thus, we have the relationship,

$$\begin{Bmatrix} \frac{\partial N_i}{\partial \xi} \\ \frac{\partial N_i}{\partial \eta} \\ \frac{\partial N_i}{\partial \zeta} \end{Bmatrix} = [J] \begin{Bmatrix} \frac{\partial N_i}{\partial x} \\ \frac{\partial N_i}{\partial y} \\ \frac{\partial N_i}{\partial z} \end{Bmatrix} \tag{5.12}$$

where [J] is given by

$$[J] = \begin{bmatrix} \frac{\partial x}{\partial \xi} & \frac{\partial y}{\partial \xi} & \frac{\partial z}{\partial \xi} \\ \frac{\partial x}{\partial \eta} & \frac{\partial y}{\partial \eta} & \frac{\partial z}{\partial \eta} \\ \frac{\partial x}{\partial \zeta} & \frac{\partial y}{\partial \zeta} & \frac{\partial z}{\partial \zeta} \end{bmatrix} \tag{5.12a}$$

where J denotes the Jacobian matrix. Thus,

$$\begin{Bmatrix} \dfrac{\partial N_i}{\partial x} \\ \dfrac{\partial N_i}{\partial y} \\ \dfrac{\partial N_i}{\partial z} \end{Bmatrix} = [J]^{-1} \begin{Bmatrix} \dfrac{\partial N_i}{\partial \xi} \\ \dfrac{\partial N_i}{\partial \eta} \\ \dfrac{\partial N_i}{\partial \zeta} \end{Bmatrix} \tag{5.12b}$$

$[J]^{-1}$ is given as,

$$[J]^{-1} = \frac{1}{\det[J]} \cdot \mathrm{Adj}\ [J] \tag{5.12c}$$

Using these relationships, the volume is calculated as

$$\int_V dv = \int dx\ dy\ dz = \int_{-1}^{1}\int_{-1}^{1}\int_{-1}^{1} \det[J]\ d\xi\ d\eta\ d\zeta$$

$$= \sum_{i=1}^{n}\sum_{j=1}^{n}\sum_{k=1}^{n} |j|\ \ C_i \cdot C_j \cdot C_k \tag{5.13}$$

Thus the stiffness matrix

$$[K^e] = \int_V [B]^T\ [D]\ [B]\ dv$$

$$= \sum_{i=1}^{n}\sum_{j=1}^{n}\sum_{k=1}^{n} [B]^T[D][B]\ \ |j| C_i C_j C_k \tag{5.14}$$

where C_i, C_j, C_k are the weighting coefficients of the Gaussian Integration scheme.

5.4 ASSEMBLEDGE

The stiffness matrices are evaluated for all the elements of the mesh and are assembled to get the global equilibrium equation-

$$\{F\} = [K]\ \{\delta\} \tag{5.15}$$

The solution of this equation system yields displacements, $\{\delta\}$ from which the strains and stresses are calculated for all the elements as-

$$\{\epsilon\}^e = [B]^e\{\delta\}^e \tag{5.16a}$$

and

$$\{\sigma\}^e = [D]\ \{\epsilon\}^e \tag{5.16b}$$

CHAPTER VI

DESIGN CONSIDERATIONS AND IDEALISATION

6.1 DESIGN CONSIDERATIONS

6.1.1 Layout of Dam

Various authors and codes of practices have described Arch Dam Layout as per U.S.B.R,

(a) Radius of Arch, $R_{axis} = 0.6\ L_1$
where L_1 is the crest length.

(b) Central angle for top arch = 90° to 110°

Keeping above in view, the radius of top arch and central angle have been kept as 300 m and 86° respectively for the present analysis of Arch cum Gravity dam.

6.1.2 Section of Dam

For simplicity of calculations, the dam section has been taken with u/s side vertical and d/s sloping so as to get base width of 95 m which is about 50% of the dam height.

6.2 DESIGN CRITERIA

The design criteria for solid straight gravity dam has been specified in Indian Standard Code I.S.:6512-1984 which is usually adopted. However, references made in "Design of Arch Dams" by U.S.B.R. are taken note of.

IS:6512-1984 and USBR Specified the following loads in design of gravity of dams -

(i) Dead load of the dam and appertinant works.

(ii) Reservoir and tail water loads

(iii) Uplift pressure on the body of the dam

(iv) Pressure due to silt which is assumed to be deposited upto dead storage level

(v) Wind pressure

(vi) Wave Pressure

(vii) Earth quake forces

(viii) Ice load where applicable

6.3 LOAD COMBINATIONS

(a) IS:6512-1984 specified the following load combinations -

(i) Load combination A (construction condition) - Dam completed but no water in the reservoir and no tail water.

(ii) Load combination B (normal operating condition) - Full reservoir level, normal dry weather, tail water, normal uplift and silt pressure.

(iii) Load combination C (flood discharge condition) - Reservoir at maximum flood level, all gates open, tail water at d/s flood level, normal uplift and silt.

(iv) Load combination D - combination A with earthquake

(v) Load combination E - combination B with earthquake but no ice load.

(vi) Load combination F - combination C but with extreme uplift (i.e. drains inoperative)

(vii) Load combination G - combination E but with extreme uplift

(i.e. drains inoperative)

(b) As per U.S.B.R., the following are the load combinations-

(i) Usual loading condition - Normal design reservoir elevation with dead loads, uplift, silt, ice load and tail water. If temperature loads are applicable, use minimum usual temperature.

(ii) Unusual loading condition - maximum design reservoir elevation with dead loads, uplift, silt, minimum temperature if applicable and tail water.

(iii) Extreme loading condition - Normal reservoir elevation with dead load, uplift, silt, ice, usual minimum temperature if applicable, tail water plus the effect of earthquake.

6.4 LOADS CONSIDERED FOR ANALYSIS

For the present analysis, load combination B of IS code (or usual loading conditions of USBR) has been adopted.

6.4.1 Water Pressure

The water level in full reservoir condition is at Elevation of. 796 m and is assumed as acting on the upstream face. The downstream water level is at an elevation of 630 m . However its magnitudes being small(head of 25m. only),the effect of downstream water pressure has not been considered in the analysis.

6.4.2 Silt Pressure

The silt has been supossed to be deposited upto dead storage level(752m) and will exert pressure on the dam face. Bouyant weight of the silt has been considered.

6.4.3 Uplift Pressure

Uplift pressure has been assumed as acting at the base and at the upper layers(i.e. 95 m above the base in this case). The magnitude of the uplift has been taken as varying from u/s head to d/s head of water. The effect of seepage gallery in reducing uplift has not been considered due to large sized elements in the dam body.

6.4.4 Initial Stresses and Temperature Effect

The observations made in the exploratory shaft at 45 m below the river bed indicate large insitu stresses. The vertical stresses are far in excess of overburden load. The relationship has been obtained as -

$$S_v = 0.0602 \text{ Mpa/m of depth}$$

Insitu stresses govern the excavation of any opening in the rock mass as direction of opening is governed by the direction of major principal insitu stress. For dam foundations, these can be added to determine total stress acting at a point in the rock mass. Similarly the effect of temperature is to create initial stresses in the dam body which can be superimposed, if desired.

Thus, for the present analysis, effects of initial insitu stresses and temperature stresses are not being considered.

6.5 MATERIAL CHARACTERISTICS OF DAM AND FOUNDATIONS

The dam and foundations have been considered totally integrated at the interface and following characteristic of dam and foundations have been taken in the analysis -

Elasticity of Concrete, E_c	= 2,00,000 kg/cm^2
Poissons Ratio, ν	= 0.2
Specific gravity,	= 2.4
Modulus of Deformation for trap	=1,00,000 kg/cm^2
Modulus of Deformation for Quartzite Slates	= 25,000 kg/cm^2
Modulus of Deformation for slates	= 10,000 kg/cm^2
Modulus of Deformation for gouge	= 2,000 kg/cm^2
Poisson's ratio for above	= 0.235

The rock mass has been considered as weightless.

The slates have been assumed to be isotropic due to limitations of the program used.

6.6 DISCRETIZATION OF CONTINUUM

The loads and stresses vary over the extent from 1 H to 4 H depth, the dam height, H being 195m.. However, due to limitations of computer capacity, the whole continuum has been considered as indicated in Fig.6.1.

In X-direction (u/s to d/s), a length of 345 m of rock mass has been considered giving atleast Ø.5H dimension in the u/s and d/s direction beyond dam body (Fig.6.1).

In Y-direction (across the river) a maximum width of 66Ø m has been considered giving at least Ø.5 H dimensions on each side of the dam abutment and extending beyond shear zone in the left abutment(Fig.6.1).

In Z-direction (vertical), 1 H depth has been considered below dam(Fig.6.2 to 6.7).

The whole continuum has been discretised into large size 2Ø noded brick elements. The dam body contains 6 elements and foundation and the abutments together contain 9Ø elements. These large size elements should not affect the analysis. In fact, Zienkiewicz(1979) has shown that quite remarkable accuracy can be achieved even with a single element .

The total nodal points in the mesh are, thus, 687. The entire discretisation has been done manually due to nonavailability of 3-D mesh generation program. However, these have been checked by computer on 2-D plots. The plots at desired sections are shown in (Fig.6.2 to 6.8). The entire analysis has been done using the available 3-D package.

6.7 BOUNDARY CONDITIONS

The boundary at the bottom of continuum has been restrained in all the three x, y, z directions.

The extreme left and right ends of the continuum in x-z plane have been restrained in y direction. The extreme u/s and d/s ends y-z plane of the continuum have been left free.

However, analysis has also been carried out by restraining the extreme u/s and d/s ends so as to compare the results with earlier 2-D FEM analysis.

CHAPTER VII

DISCUSSION OF RESULTS

7.1 DEFORMATIONS

7.1.1 Vertical Deformations

Contours of vertical deformations in the dam and foundations have been plotted for the central section, B-B (Fig.6.1), in the valley in Fig. 6.8 and for the section A-A along the upstream face of dam (Fig. 6.1) in Fig. 6.9. The results indicate that a maximum deformation of 6.29 cm occur at the top of the dam while in the foundations, a deformation of 5.02 cm occurs below the toe of the dam (Fig. 6.8).

7.1.2 Horizontal (Lateral) Deformations

The horizontal (Lateral) deformation of the upstream and the downstream faces of the dam are plotted in Fig. 6.10. It is seen that the base of the dam at the upstream face undergoes a lateral deformation of 4.0 cm whereas the downstream face at the base experiences a deformation of 4.2 cm. The crest of the dam experiences a lateral displacement of 7.85 cm whereas maximum lateral displacement of 7.99 cm has been obtained at the height of about 155 m.

Figure 6.11 shows the contours of horizontal displacement along section A-A,i.e. along the upstream face, in the abutments, dam body and the foundation rock. Figure 6.12 shows the contours of horizontal displacements plotted for the cross-section along section B-B in the centre of the valley. The contours are plotted in the dam body and in the foundation portion. It shows a horizontal displacement of about 4.5 cm. along the base of the dam.

7.1.3 Displacements of the Shear Zone

The normal deformations across the shear zone are plotted in Fig. 6.13. The plot shows that shear zone has undergone large normal deformations. A maximum compression of 7.14 cm has been obtained at an elevation of 675.Ø m i.e. 7Ø m above the base of the dam (Fig. 6.5) which has resulted in a resultant settlement of the left abutment by 6.63 cm.

Figure 6.14 shows the values of shear displacement plotted along the oblique plane representing the shear zone. The shear displacement was obtained by summing up the components of normal displacements u, v and w along the direction of the shear zone. A maximum shear displacement of 3.75 cm has been found at an elèvation of 675.Ø m in the left flank at a height of 7Ø.Ø m above the base of dam.

The vertical and the lateral deformations in the dam body are within reasonable limits. However, the shear displacements, particularly in the left bank abutment, warrant a detail analysis of this abutment alone.

7.2 STRESSES

7.2.1 Major Principal Stresses

Major principal stresses along the upstream face (section A-A) and across the valley(section B-B) have been plotted and shown in Figs. 6.15 and 6.16 respectively. The maximum compressive stress of 446 T/sq m occurs in the dambody at a height of 60.0 m above the dam base. The foundations develop a maximum compressive stress of 360 T/sq m below the dam base near the toe.

The high compressive stresses concentrate between the left abutment of dam and the shear zone. Fig. 6.16 shows the high compressive stress of 80 t/m^2 in the major portion of the shear zone. This further necessitates strengthening of the weak shear zone.

7.2.2 Minor Principal Stresses

Minor principal stresses have been plotted and shown in Figs. 6.17 and 6.18 for the same sections as for the major principal stresses. Figure 6.17 indicates that tensile stresses as large as 200 T/Sq m have developed along the right abutment. The right abutment of the dam, thus, encounters high tensile stresses. The suitable shape and curvature of the dam alongwith suitable layout of the dam may help in reducing the high tensile stresses or alternatively, the right abutment may require suitable anchoring.. There are no tensile stresses at the heel of the dam. However, small tensile stresses have been found on the downstream face of dam which will not cause any damage to the dam.

The foundations show development of high tensile stresses of the order of 16 kg/cm^2 on the right flank (Fig. 6.17). The high insitu stresses present in the foundations are of the order of .0607 MPa per metre depth (art. 6.4.4). At a depth of about 40.0 m below the top of the right abutment, the in situ stresses work out to be of the order of 24 kg/cm^2. These in situ stresses will therefore balance the tensile stresses of the order of 16 kg/cm^2. However, an extensive analysis is required if the orientation of dam axis and suitable shape and curvature of the dam do not reduce these stresses.

Shear zone also faces development of tensile stresses which may further open it up. Since these are very small and high in situ stresses exist in the foundation, these may not create any problem.

7.3 SHEAR MOBILISATION ALONG CONTACT PLANES

The shear friction factor along abutment and base of dam has been calculated and plotted in Fig. 6.19. This indicates that leaving a part of right abutment which is undergoing state of high tensile stresses, the shear friction factor along the contact of the dam with the foundations is very high, meaning thereby, that the dam base width can be reduced.

7.4 STABILITY AGAINST SLIDING

The Finite Element analysis does not give directly the factor of safety against sliding. However, it can be assessed from the development of tensile stresses at the heel. If the tensile stresse at the heel are high, the dam may be unsafe due to cracking.An assessment has been made to find factor of safety against sliding by calculating the thurst from the stresses in the abutments and using the equation (eqn 2.1) given by Mgalobelov (1979).

The factor of safety has been found to be 2.19

7.5 COMPARISON OF RESULTS OF TWO TYPES OF DAMS.

A comparison has been made with the 2-D FEM analysis carried out in year 1988 in UOR , Roorkee for the gravity section and the present 3-D FEM analysis for Arch Cum Gravity Dam. The transverse deflections at the top and at the base have been plotted in Fig. 6.20. It may be noted that there is a substantial reduction in the values of deformations at the top and at the base of the dam. The deformations have reduced from the earlier 24 cm to 7.85 cm at the crest and from 8.00 cm to 4.21 cm respectively at the base of the dam.

It is also interesting to note that when the upstream and downstream extreme boundaries are also restrained, the deformations have considerably reduced . These are now 4.67 cm at top and 2.37 at the heel (Fig. 6.20).

For exact comparison, a 3-D FEM analysis has been carried out with the modulus values as adopted in earlier 2-D FEM analysis but with upstream and downstream extreme boundaries restrained. The results are very encouraging (Fig. 6.20) and show that the deformations are only 5.94 cm at the top and 3.31 cm at heel as compared to 24 cm and 8 cm respectively in the 2-D finite element analysis. This adds to the knowledge that the value as low as 56,000 kg/cm^2 of the modulus of deformation for trap rock will not be a constraint for the proposed Arch cum Gravity dam at Lakhwar.

CHAPTER VIII

CONCLUSION AND SCOPE FOR FUTURE WORK

8.1 CONCLUSION

From the 3-D FEM analysis, it is observed that the foundations for Arch cum Gravity dam will experience much lesser deformations as required for the safety of dam. The stresses in the foundations as well as in the body of dam are not very excessive and optimization of the section and layout of the dam will certainly improve the behaviour of the foundations. The proximity of shear zone in the left flank is a cause of concern. However, it can be suitably treated and the natural cure would be to find that the resultant of forces in the left abutment do not cross the shear zone diagonally.

Thus, it can be safely concluded that the Arch cum Gravity dam at Lakhwar is a viable solution for the existing complex foundation conditions. Even a value of modulus of deformation lower than the value cosidered in the present analysis will not be a cause of concern.

8.2 SCOPE FOR FUTURE WORK

The present analysis has been carried out by adopting a coarse mesh of elements. With the finer mesh, the rock properties for every change of location of rock can very well be included in the analysis. Moreover, the analysis has been done with a plane section whereas the optimized section would be a polynomial of 3rd or 4th degree. A single arch has been considered in the analysis whereas multiple curvature arches or parabolic arches would provide an economical as well as safe layout. The stability of left bank is to be tested and the shear zone needs to be analysed as a joint following Mohr-Coulomb or any other law.

Lastly, since most rocks behave non linearly and the traps are highly jointed, a mathematical model for such jointed rock should be prepared and a non linear behaviour should be studied.

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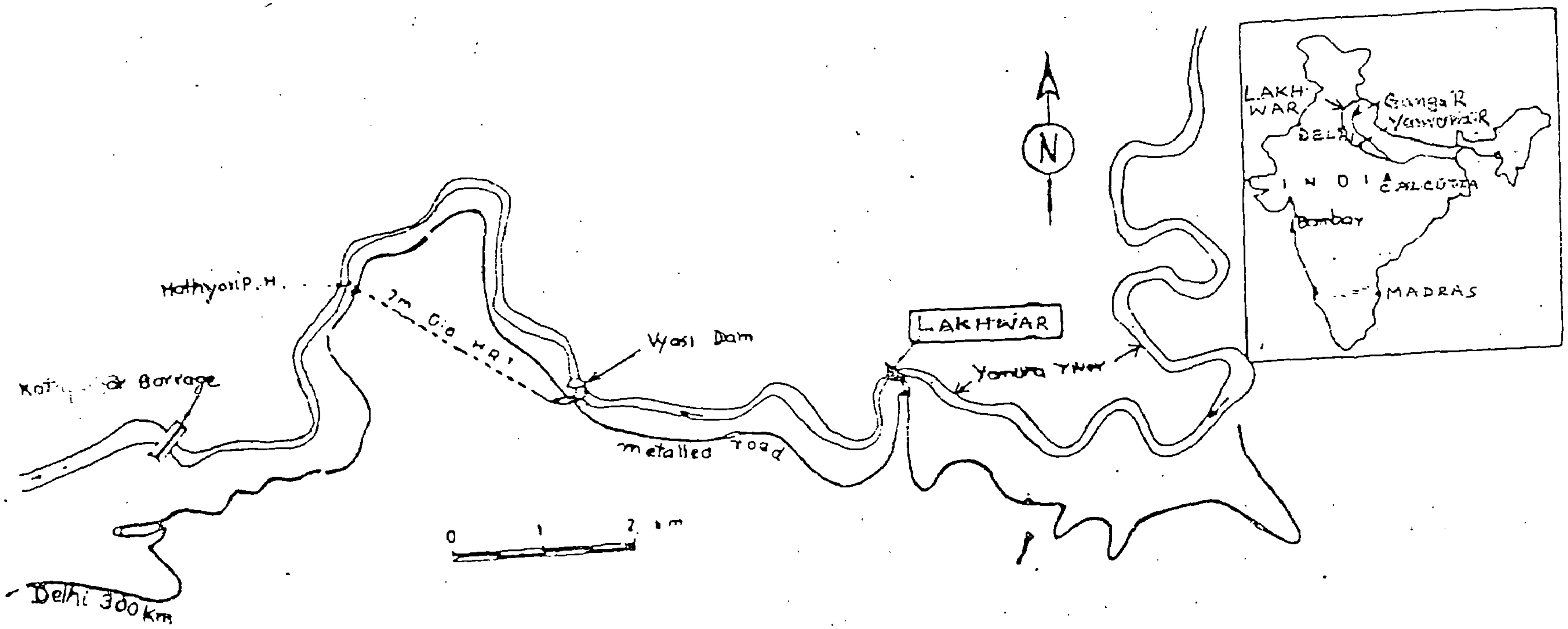


FIG 1.1. LAYOUT OF LAKHWAR-VYASI HYDRO ELECTRIC SCHEME

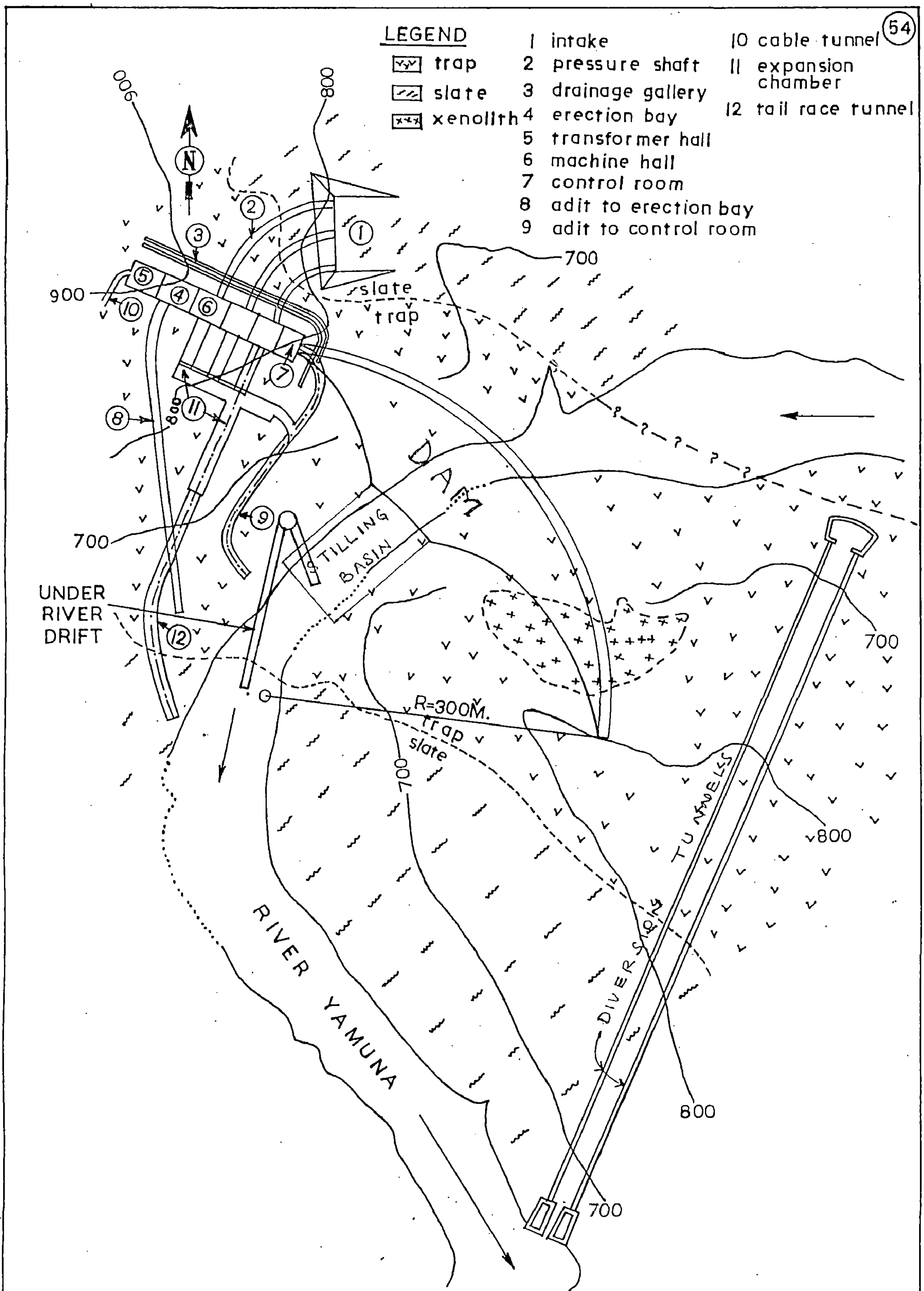


FIG 3·1. GEOLOGICAL PLAN SHOWING LAKHWAR DAM

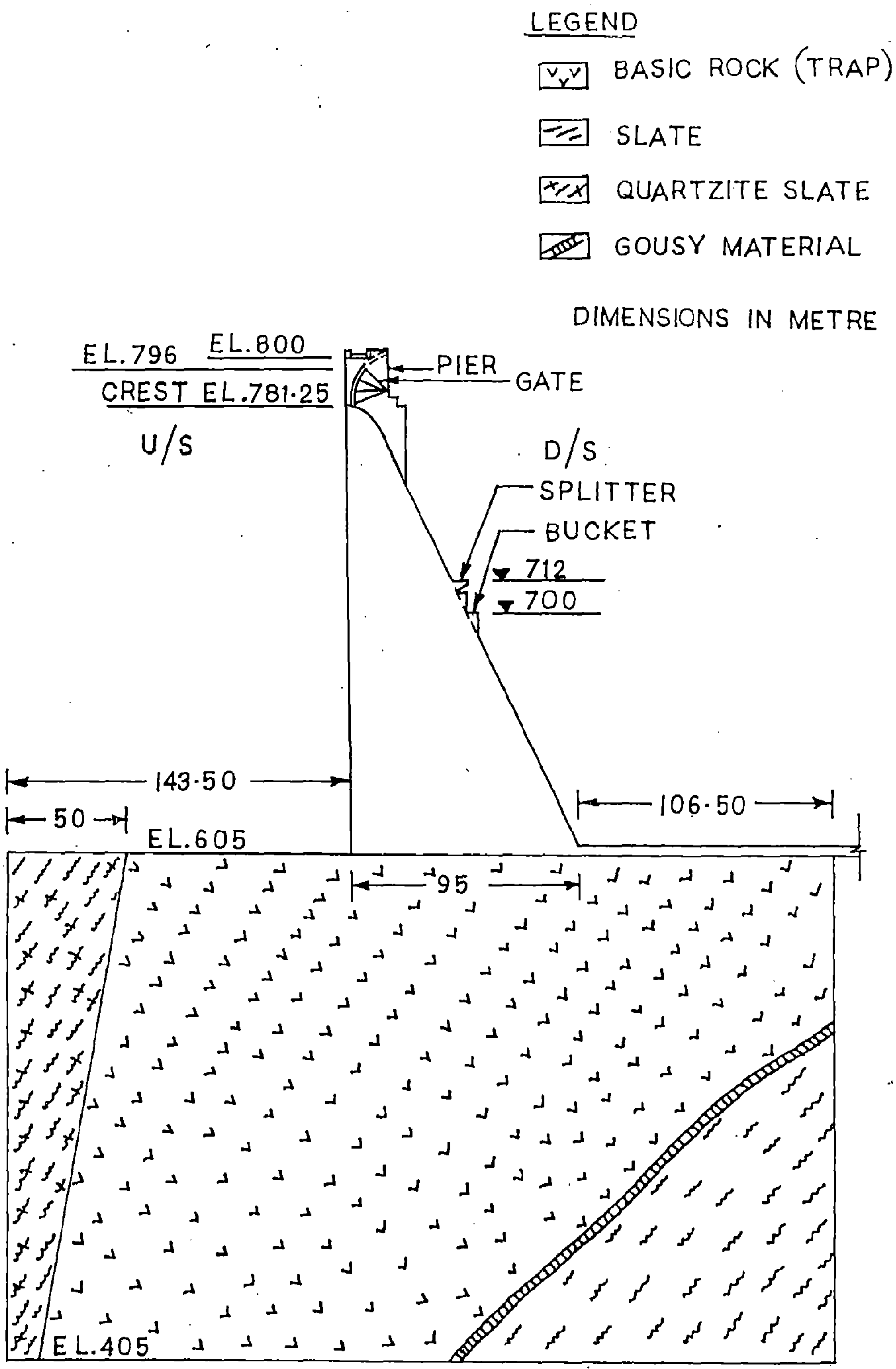
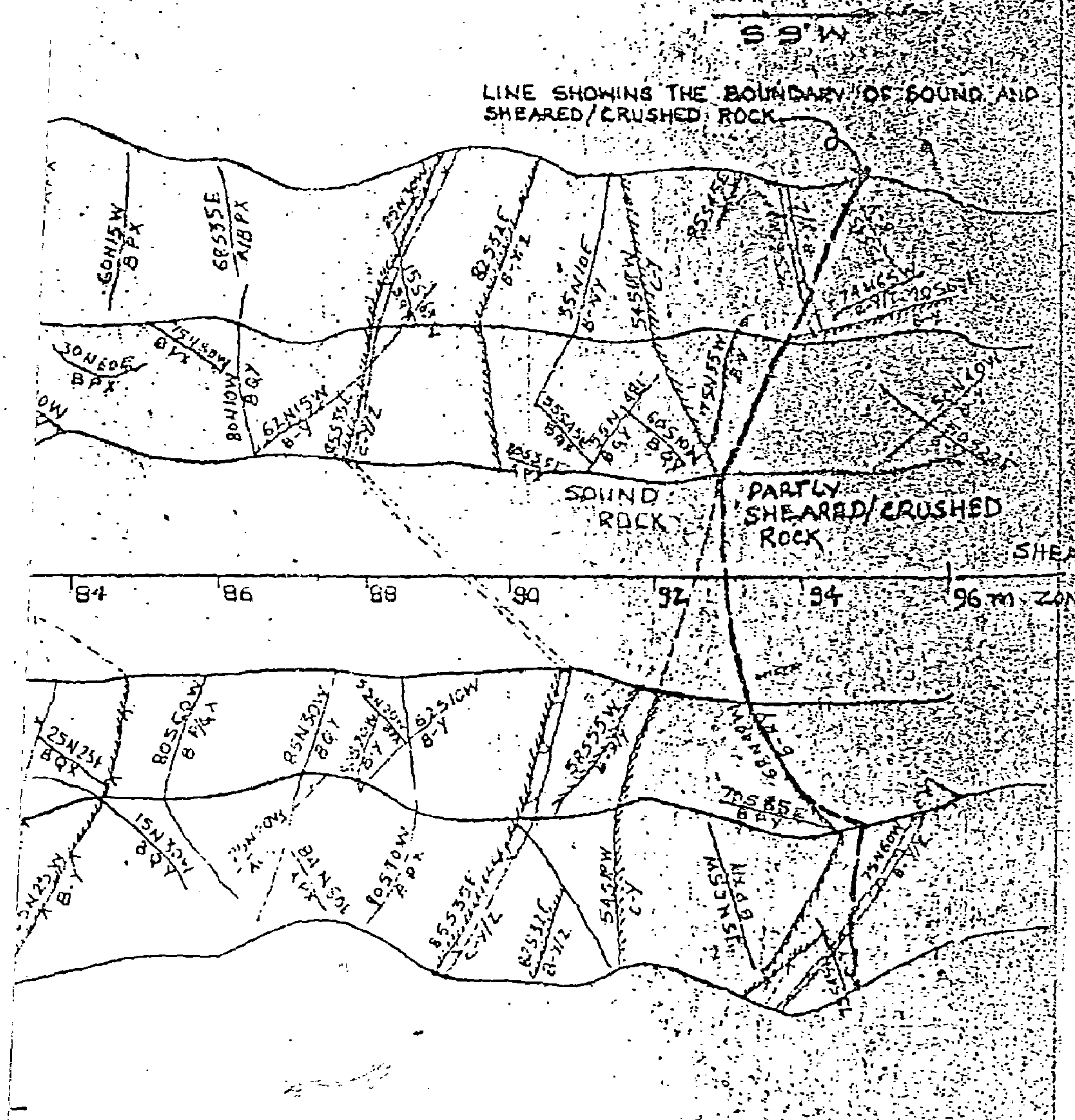


FIG 3·2. GEOLOGICAL SECTION ALONG LAKHWAR DAM SPILLWAYS

G BASIN (R.D. 65 - 96m)



JOINT PLANE

SHEAR ZONE

VEIN (CALCITE / QUARTZ)
(LESS THAN 10 cm/MORE THAN 10 cm)

NTINUITY CHARACTERS

TINUITY B, C.	SPACING P, Q, R	NATURE OF INFILLINGS X, Y, Z
HAN 1 m (A)	LESS THAN 20 cm (P)	CLEAN (X)
1 D (B)	20 cm TO 50 cm (Q)	ROCK FLOUR, GRANULAR (Y)
THAN 1 D (C)	MORE THAN 50 cm (R)	CLAYEY, PLASTIC, SLICKENSIDED (Z)

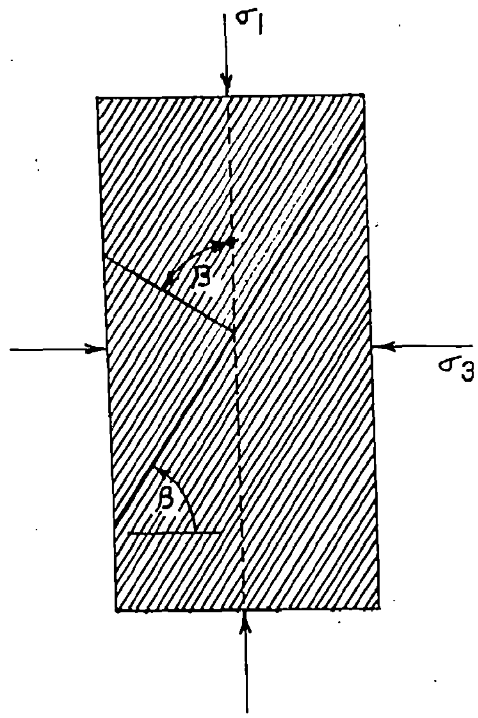


FIG 4·1(a). TRANSVERSELY ISOTROPIC SPECIMEN IN TRIAXIAL COMPRESSION

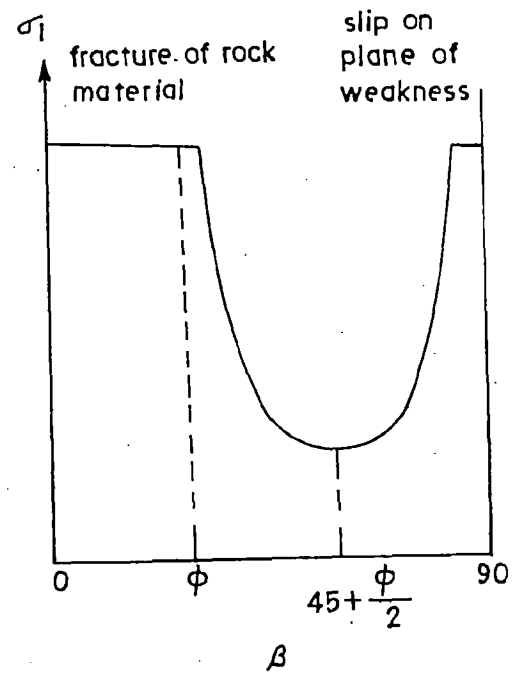


FIG 4·1(b). VARIATION OF PEAK STRENGTH AT CONSTANT CONFINING PRESSURE WITH THE ANGLE OF INCLINATION OF THE NORMAL TO THE PLANE OF WEAKNESS (β)

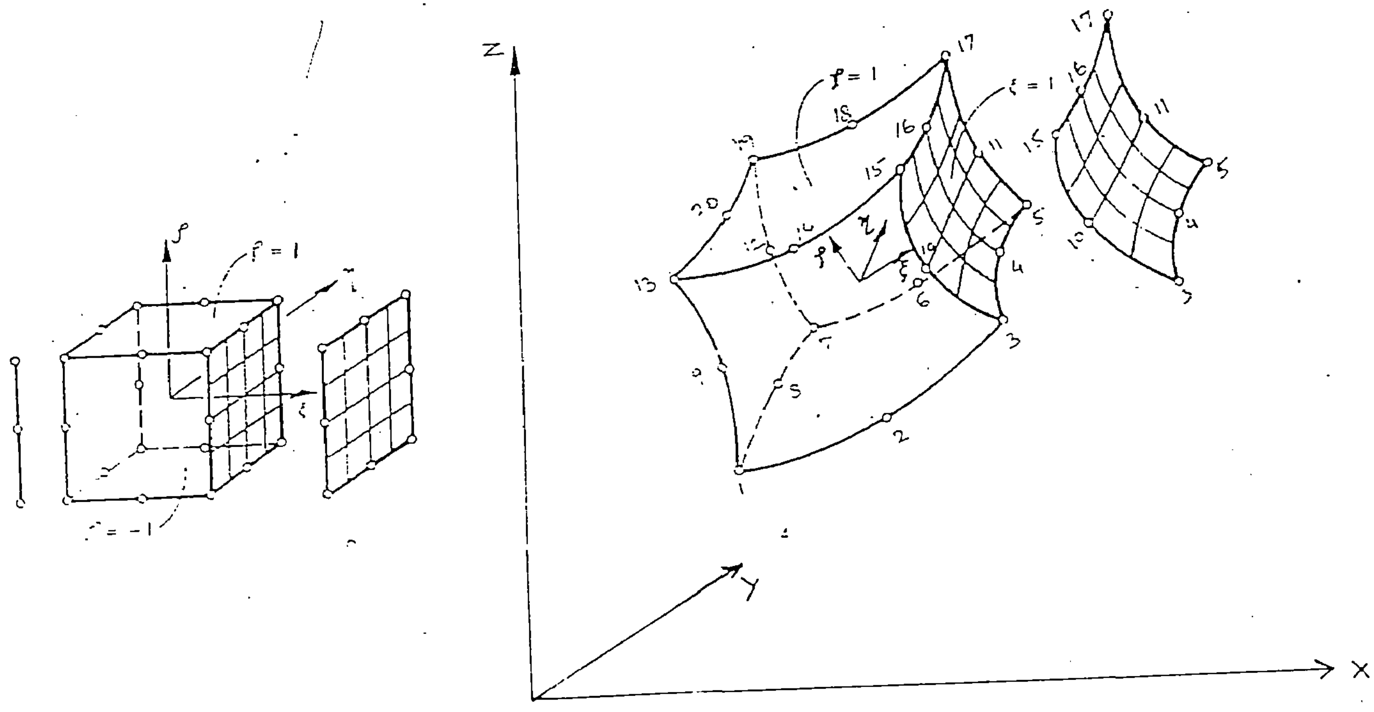


FIG 5·1. THREE DIMENSIONAL MAPPING OF QUADRATIC BRICK ELEMENT

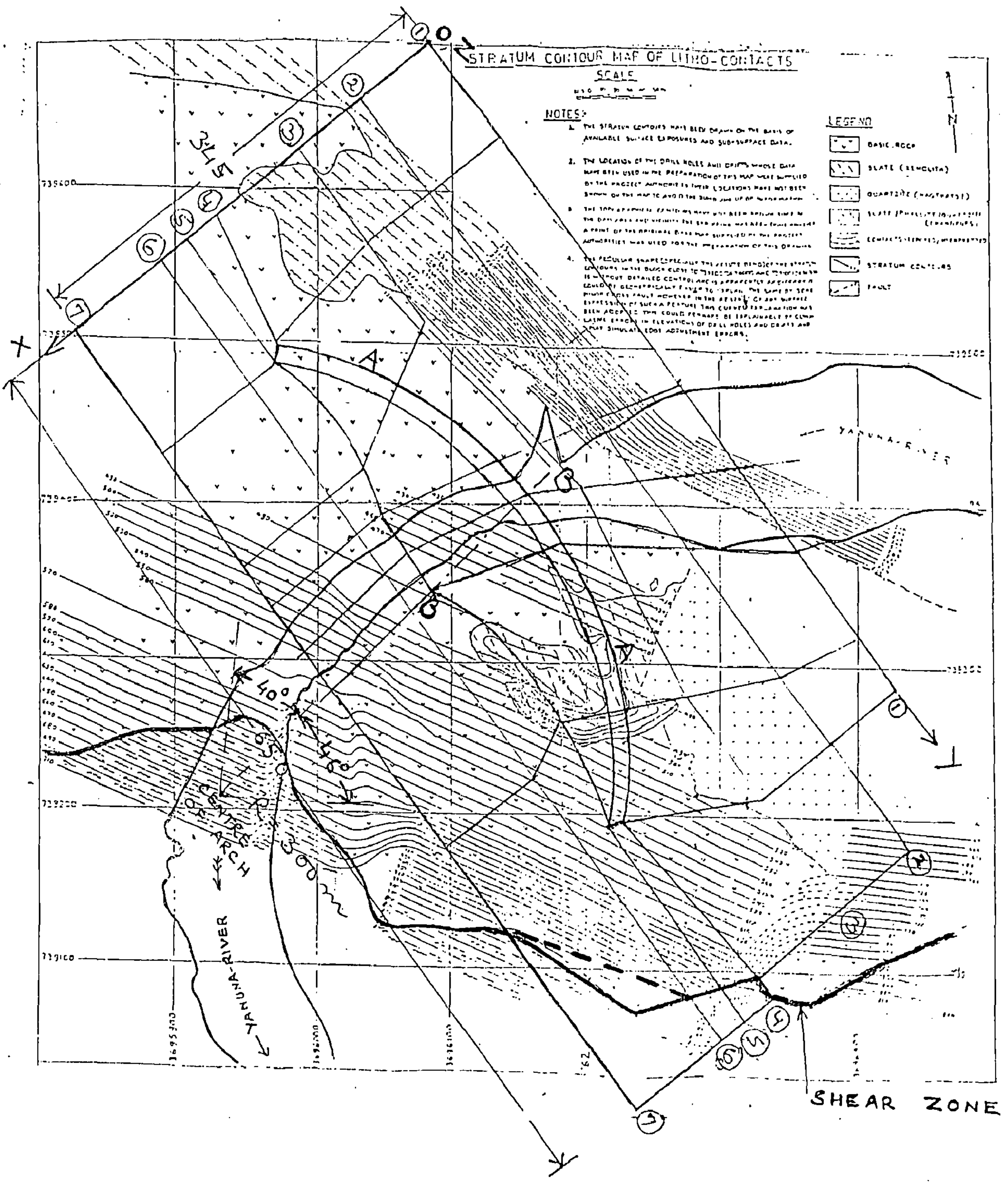


FIG 6·1. MESH FOR FINITE ELEMENT ANALYSIS

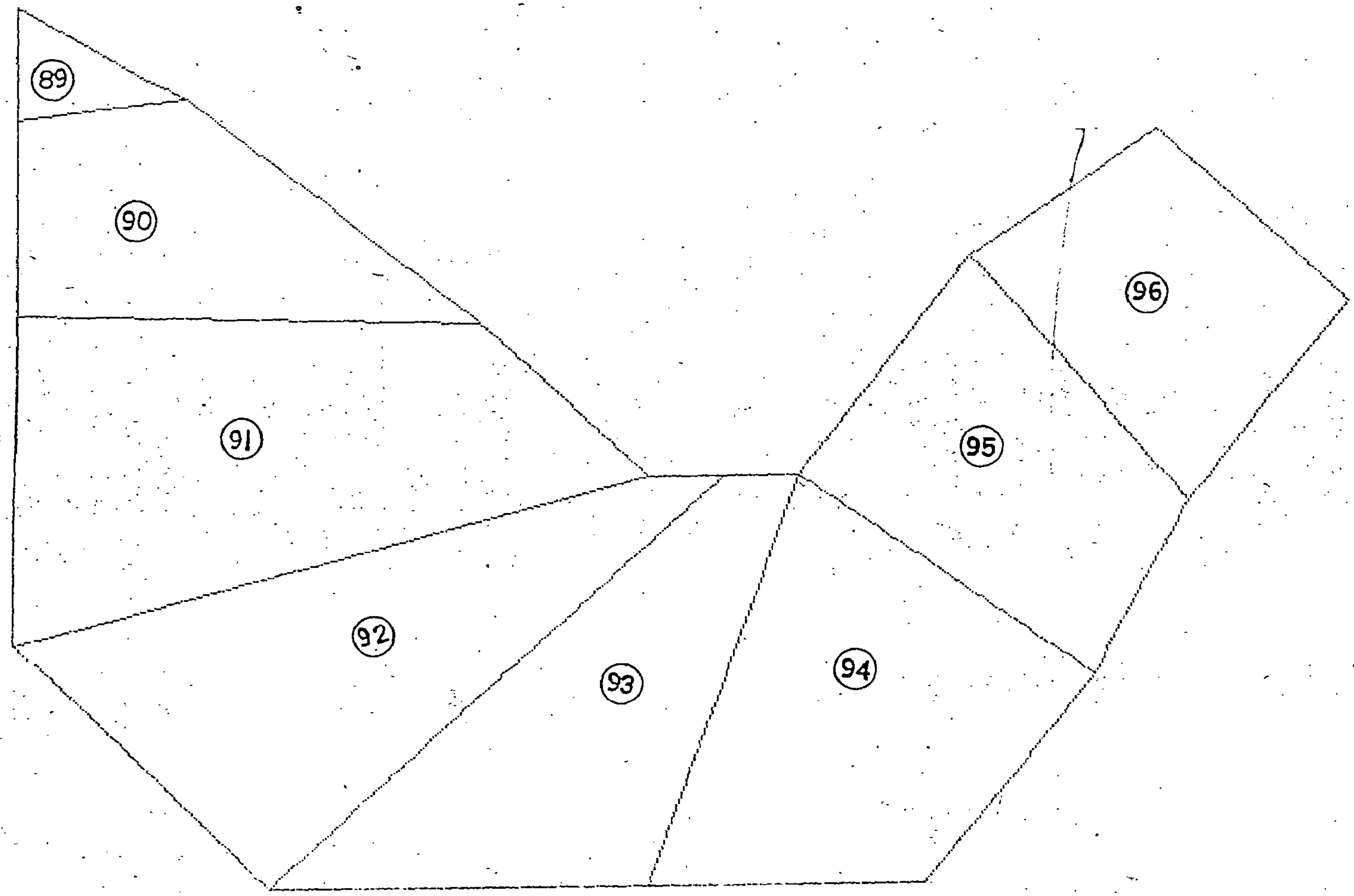


FIG 6-2. SECTION AT 1-1
SHOWING ELEMENT NOS.

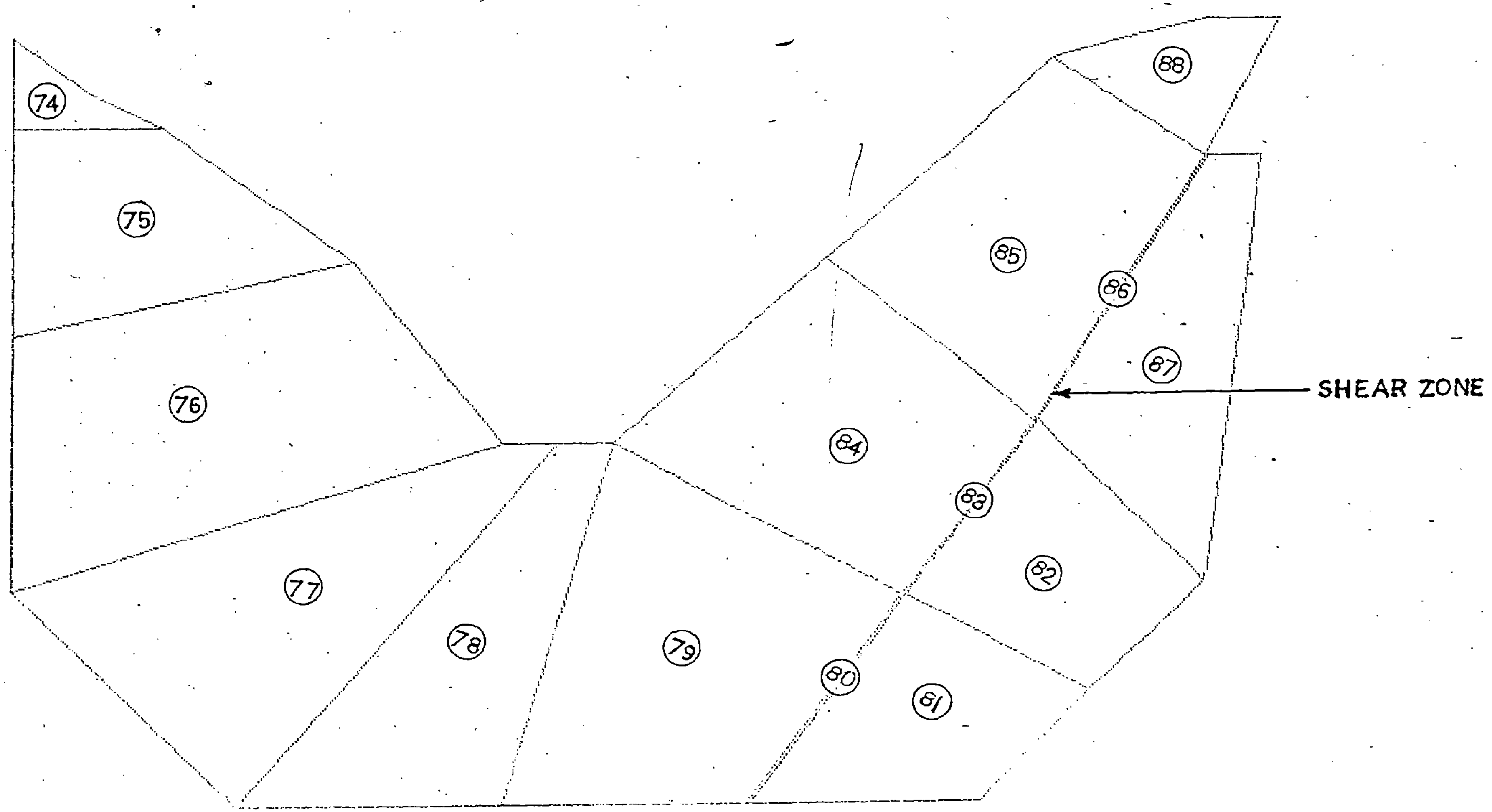


FIG 6·3. SECTION AT 2-2
SHOWING ELEMENT NOS.

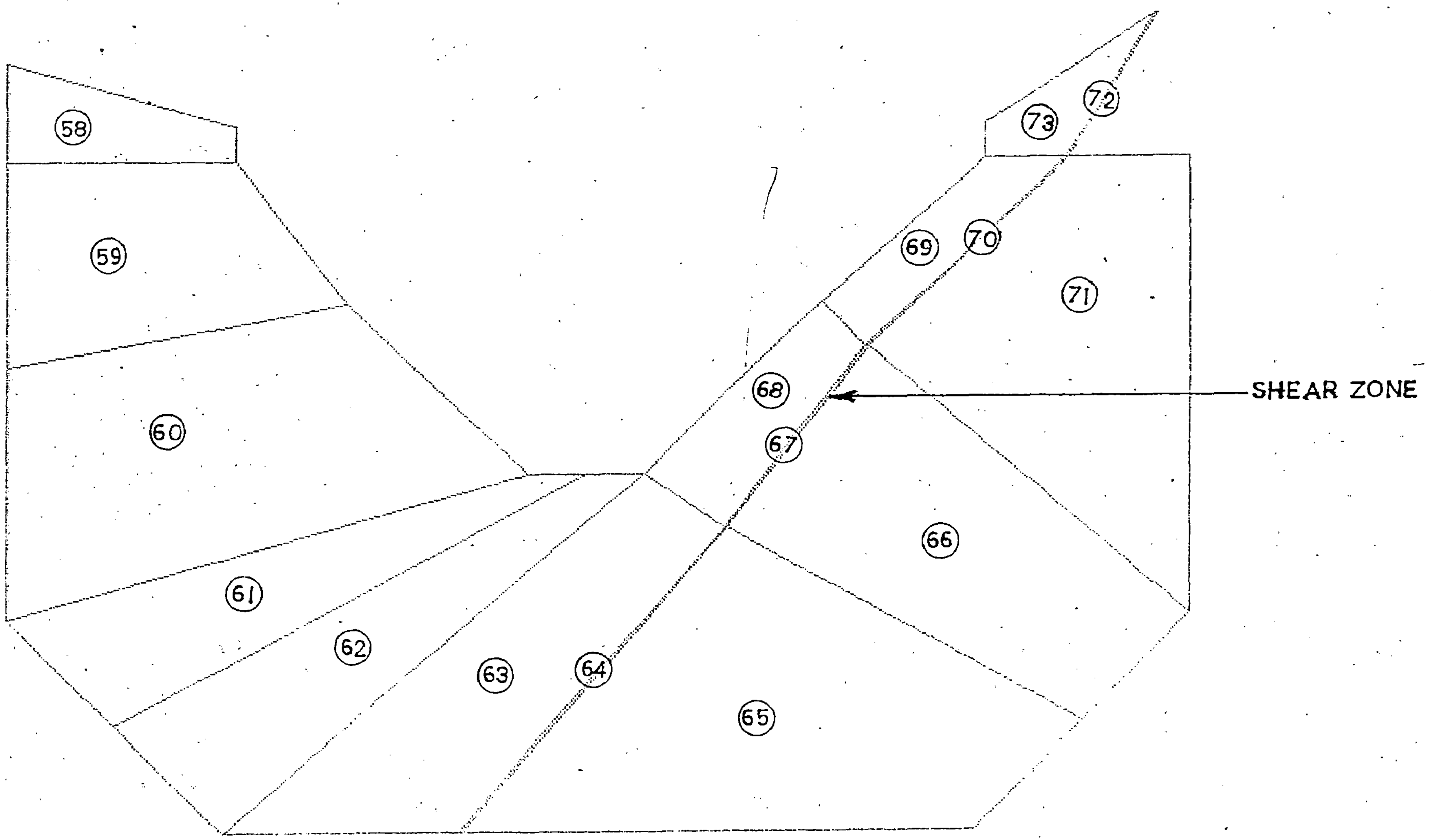


FIG 6·4. SECTION AT 3-3
SHOWING ELEMENT NOS.

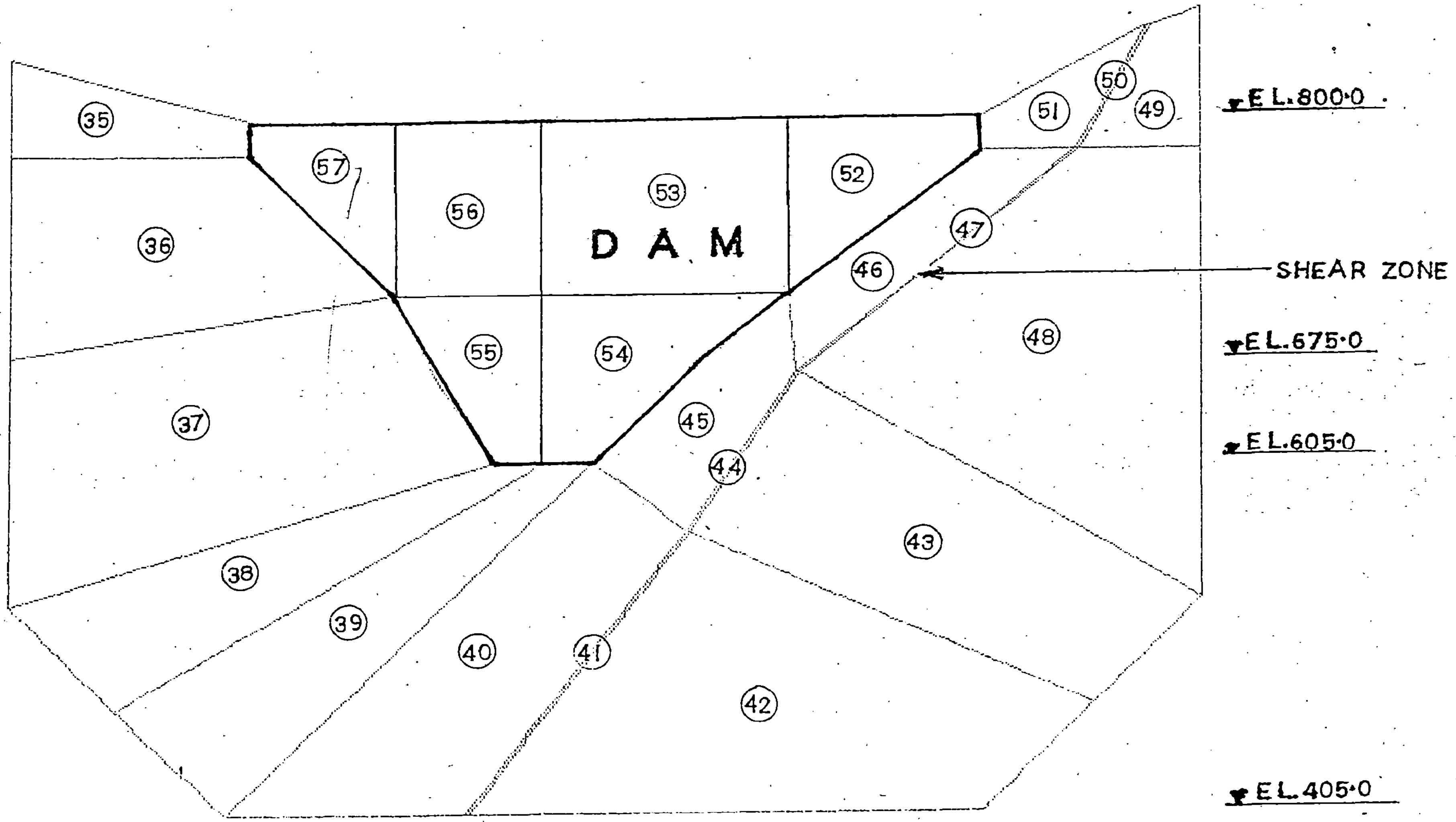


FIG 6·5. SECTION AT 4-4
SHOWING ELEMENT NOS.

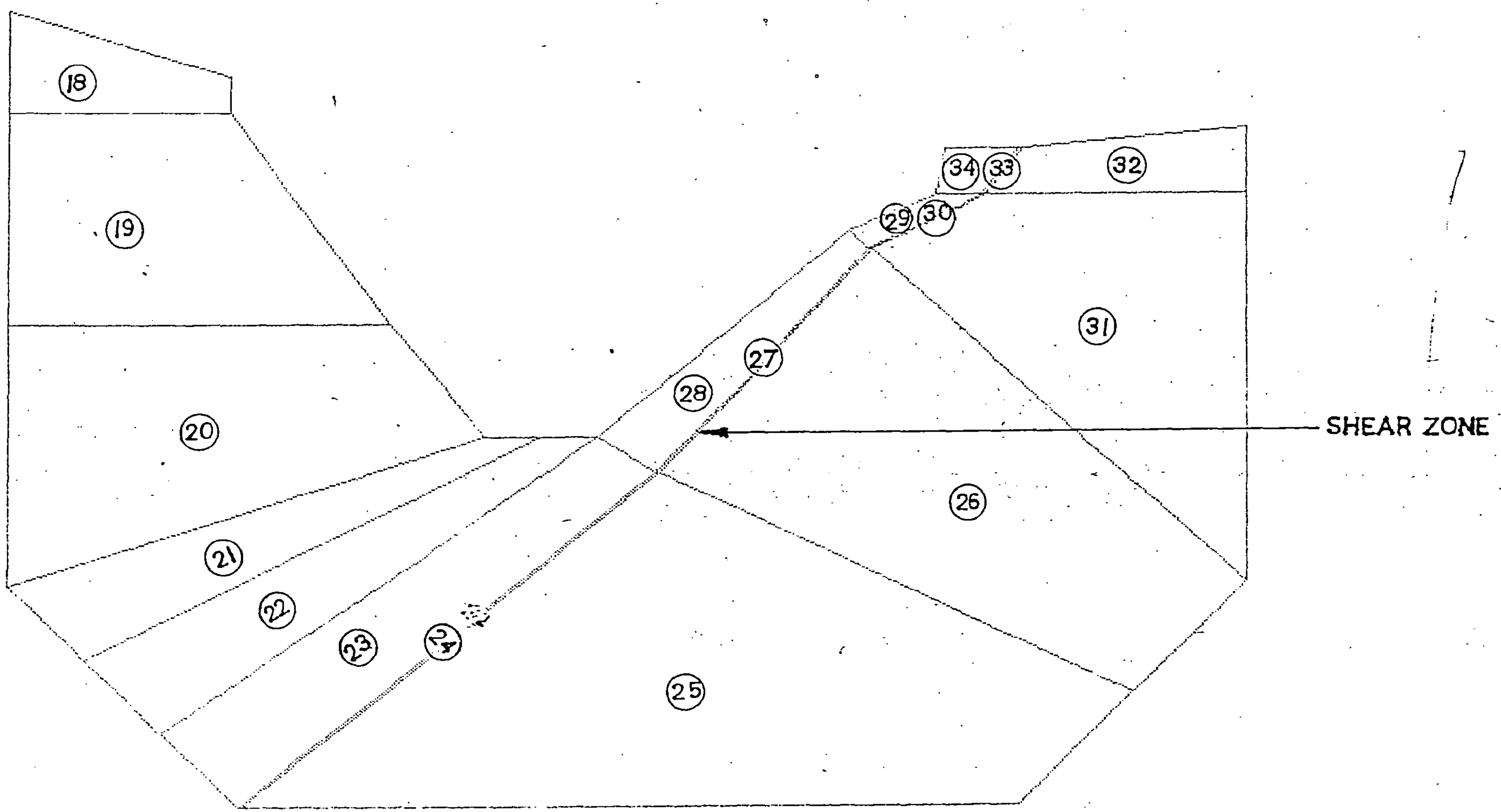


FIG 6·6. SECTION AT 5-5
SHOWING ELEMENT NOS.

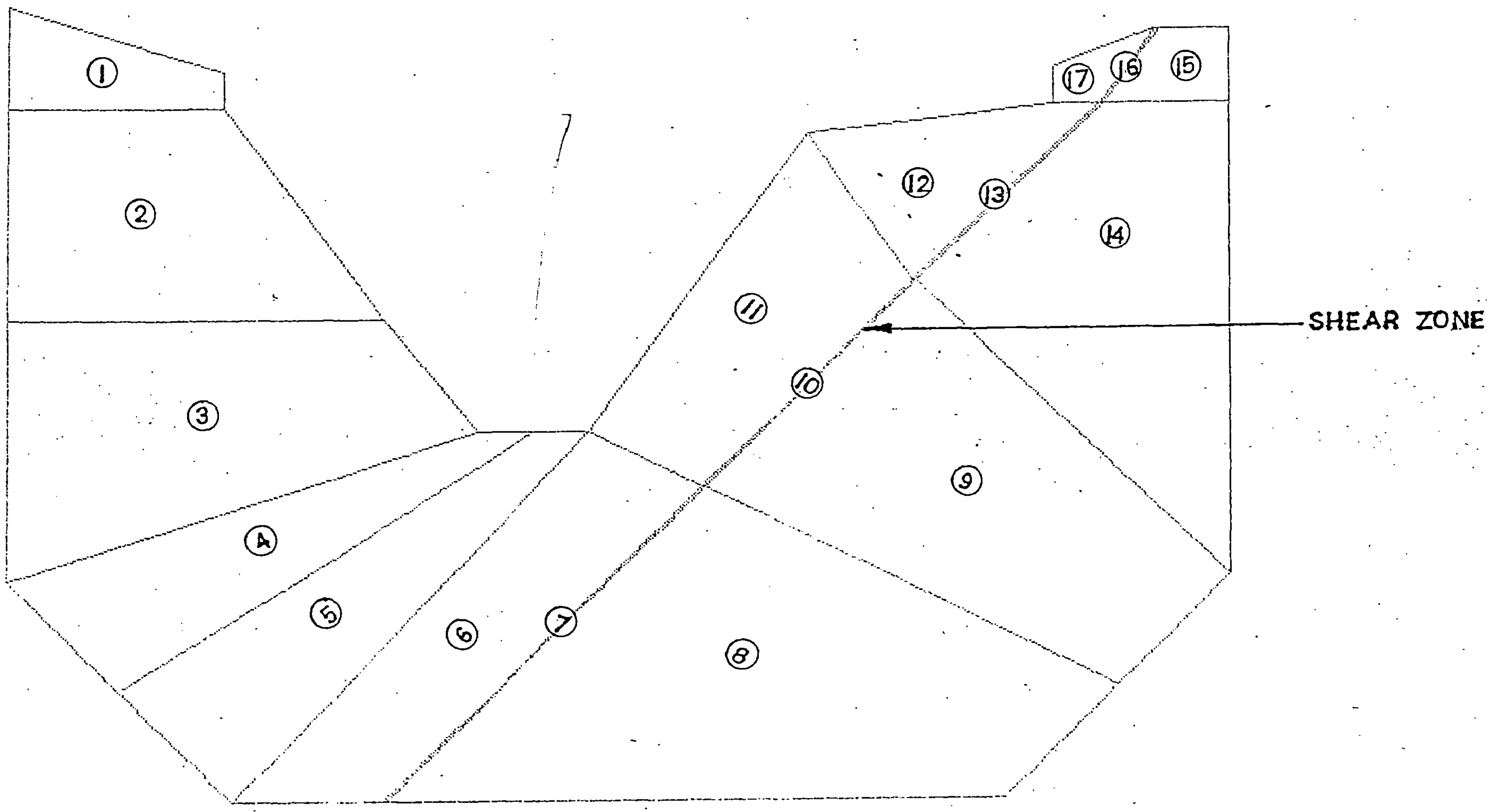


FIG 6·7. SECTION AT 6-6
SHOWING ELEMENT NOS.

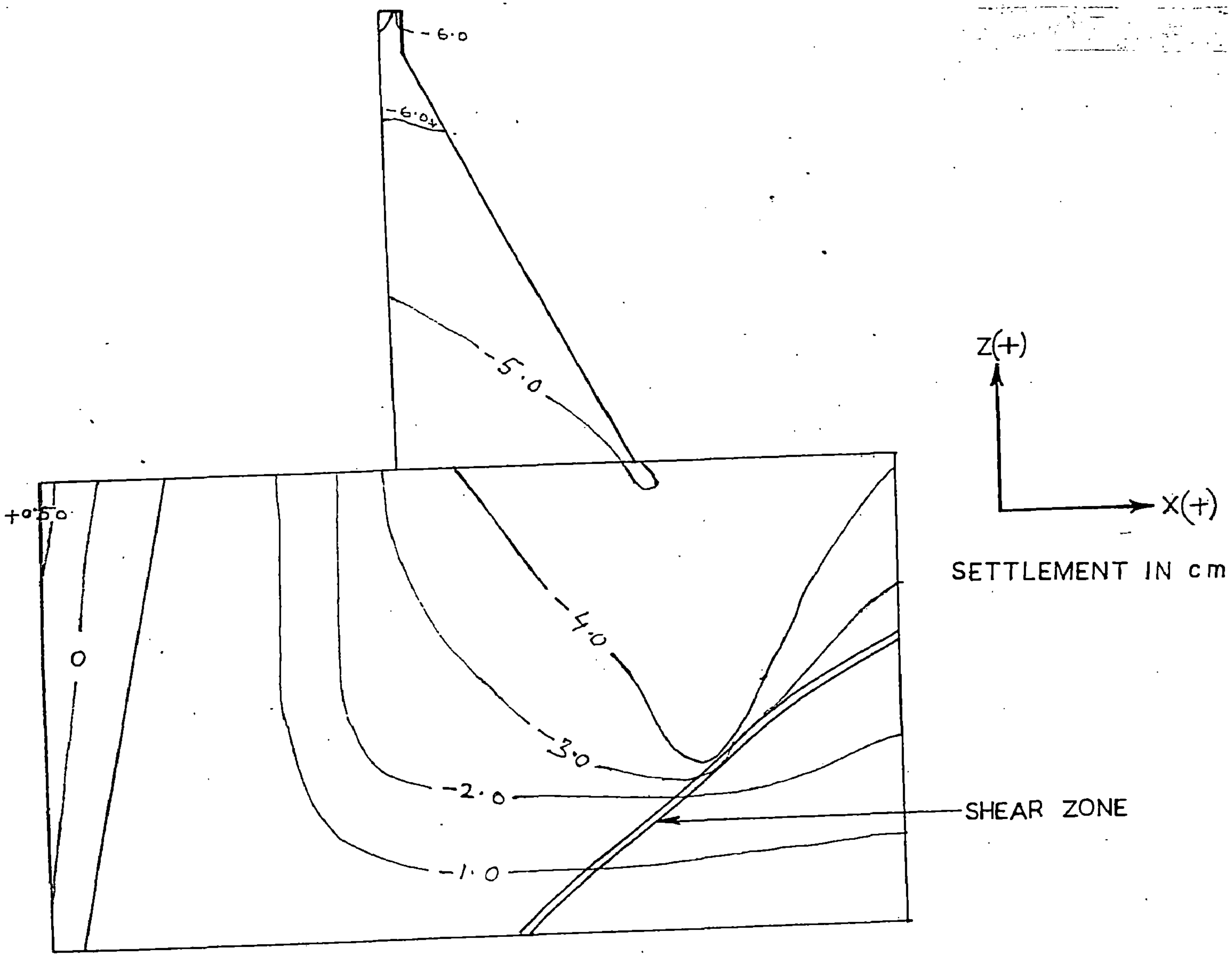


FIG 6·8. VERTICAL SETTLEMENT IN THE CENTRAL SECTION OF VALLEY SECTION (B–B)

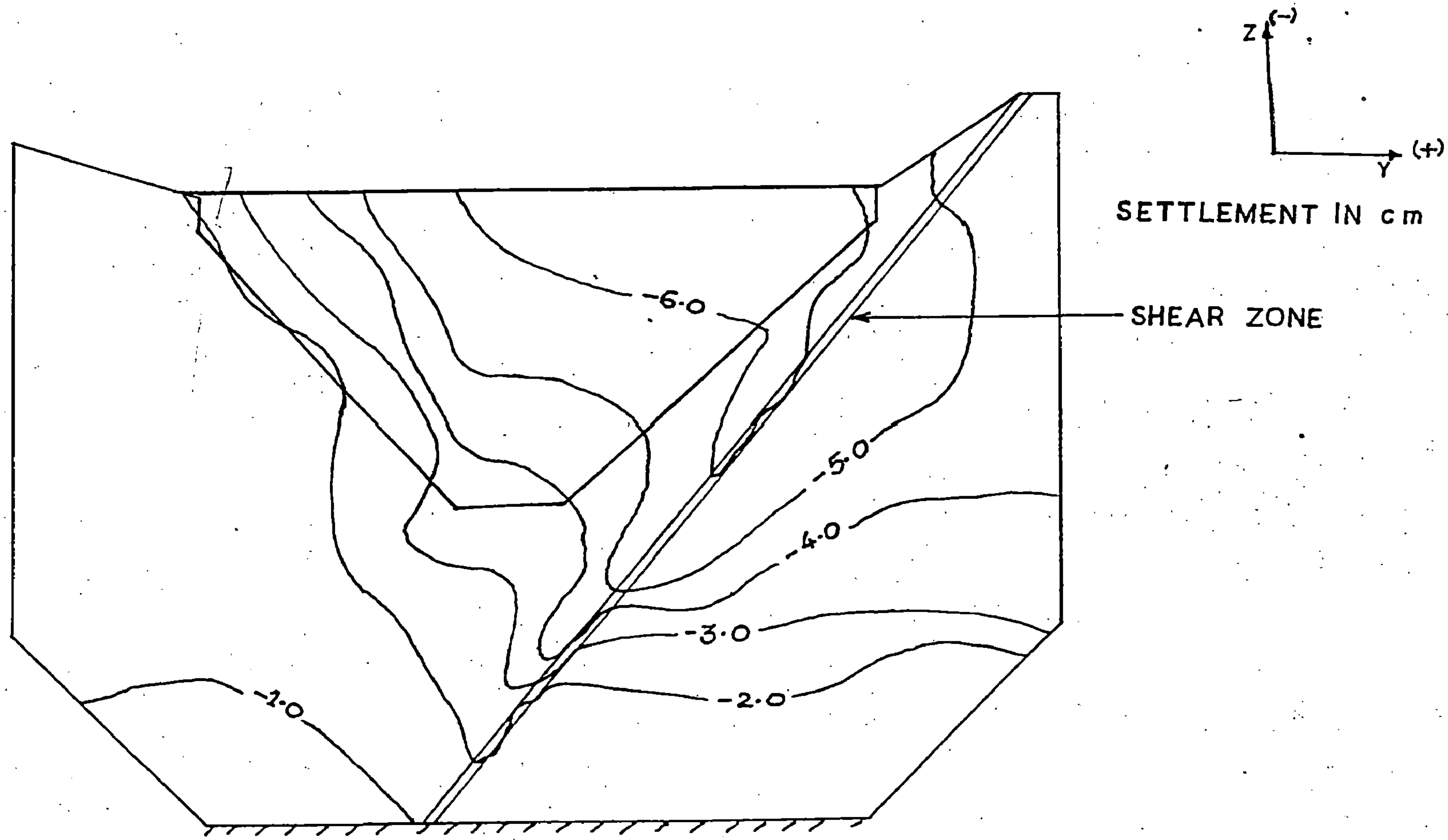


FIG 6·9. VERTICAL SETTLEMENTS ALONG U/S FACE OF DAM (SECTION A–A)

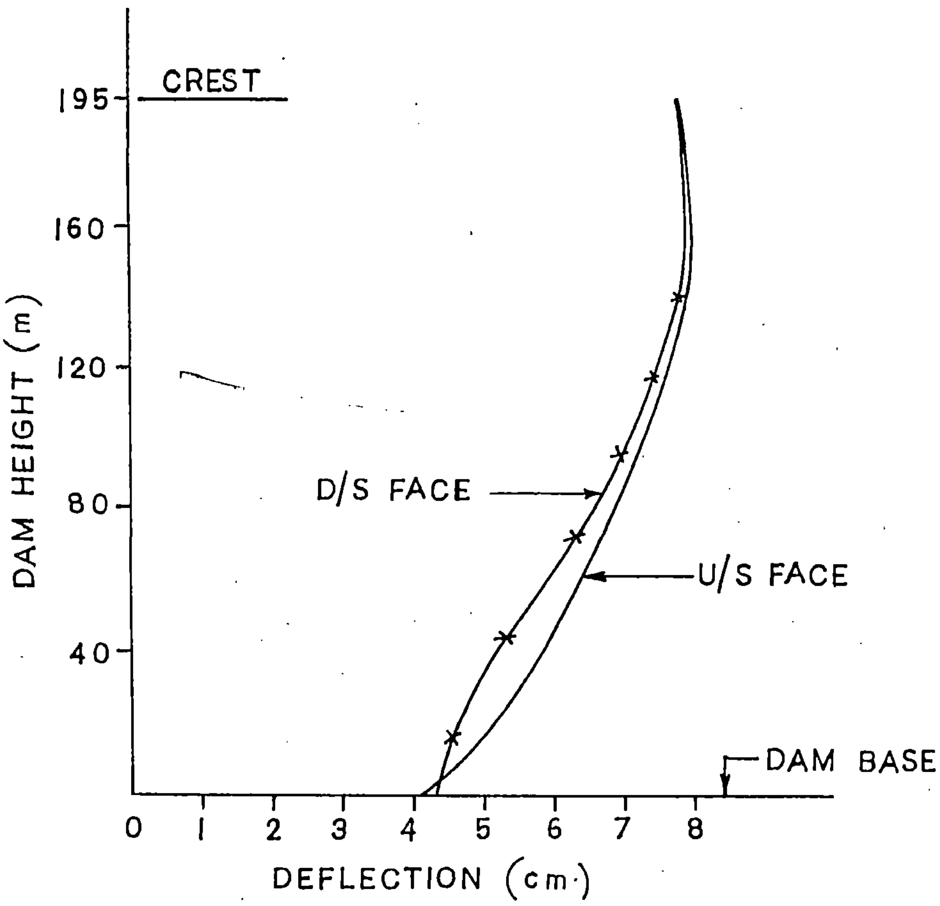


FIG 6·10. LATERAL DEFLECTION OF DAM FACES (SECTION B-B)

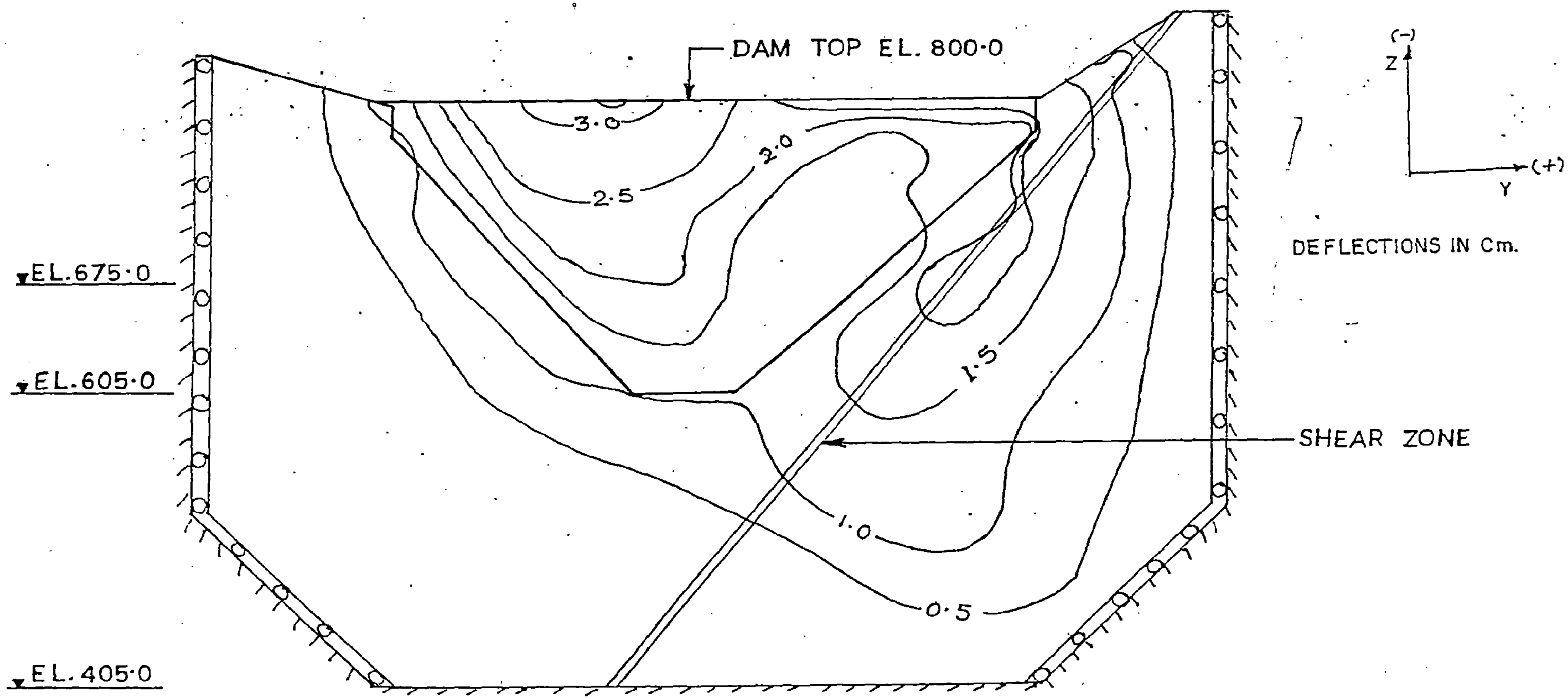


FIG 6·11. HORIZONTAL DEFLECTIONS ALONG U/S FACE (SECTION A-A)

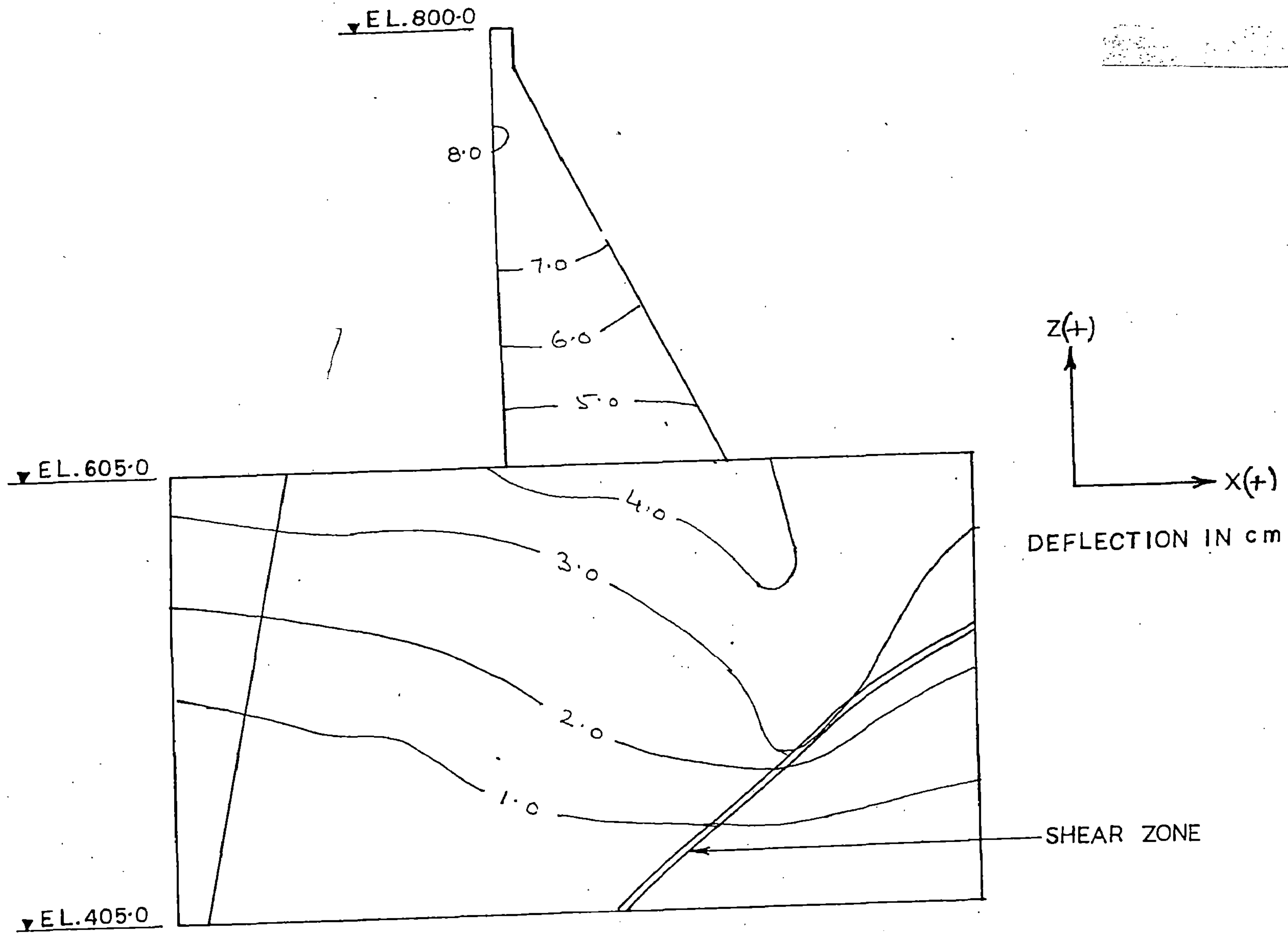


FIG 6·12. HORIZONTAL DISPLACEMENT IN THE CENTRAL SECTION OF VALLEY ALONG B-B

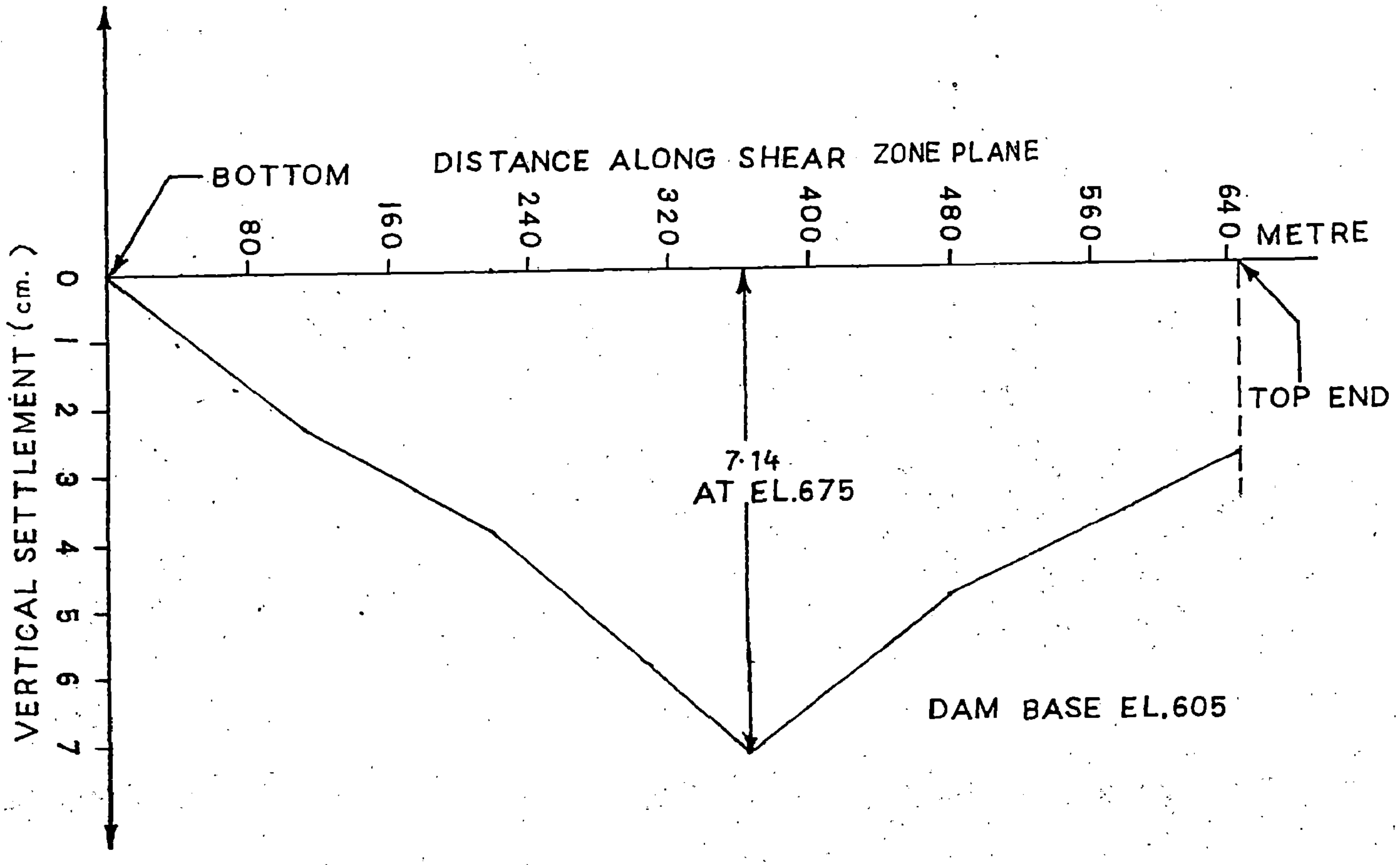


FIG 6·13. **NORMAL DEFORMATION ACROSS SHEAR ZONE**

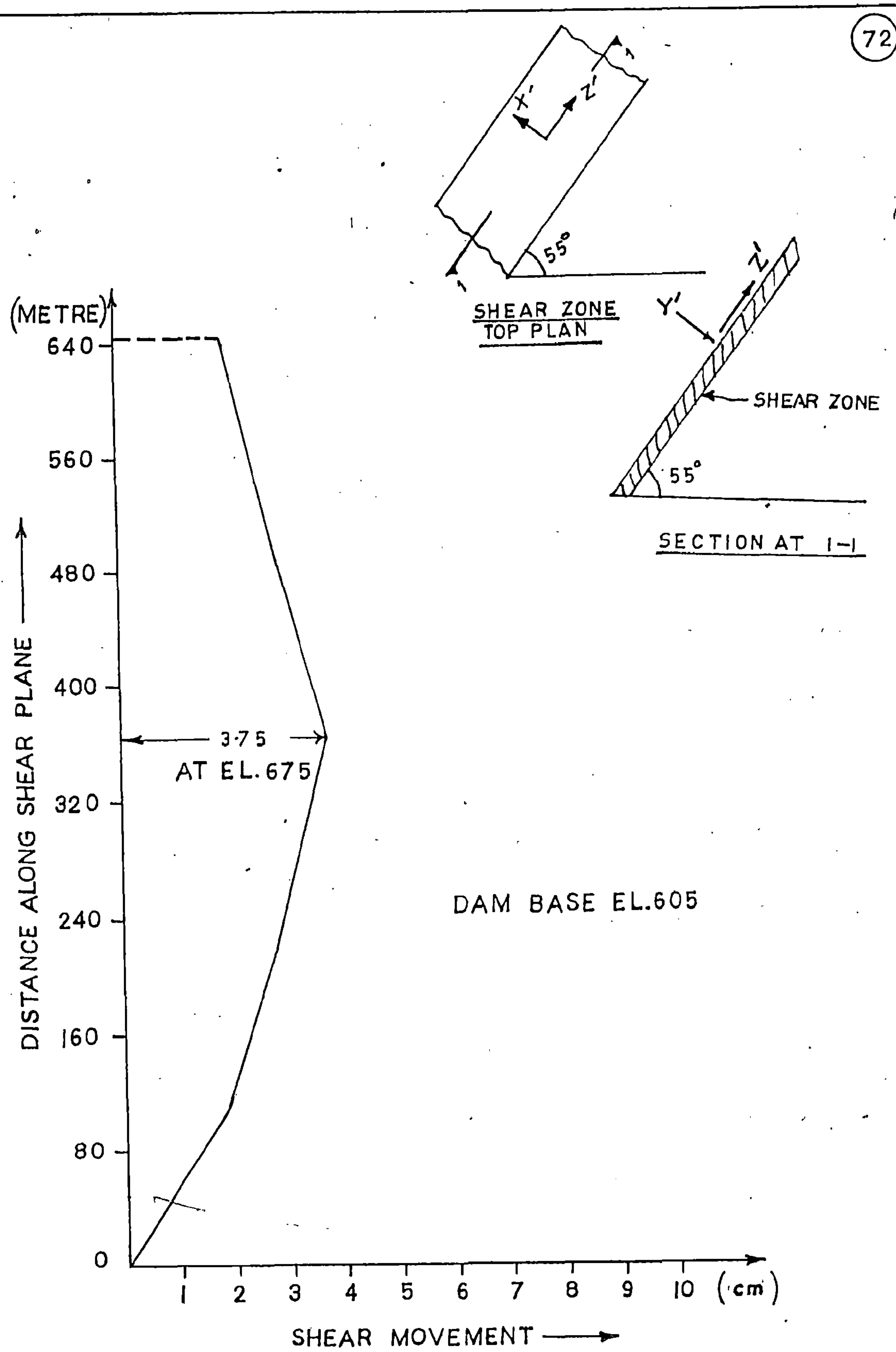


FIG 6·14. DISPLACEMENT ALONG SHEAR ZONE PLANE IN X'- DIRECTION

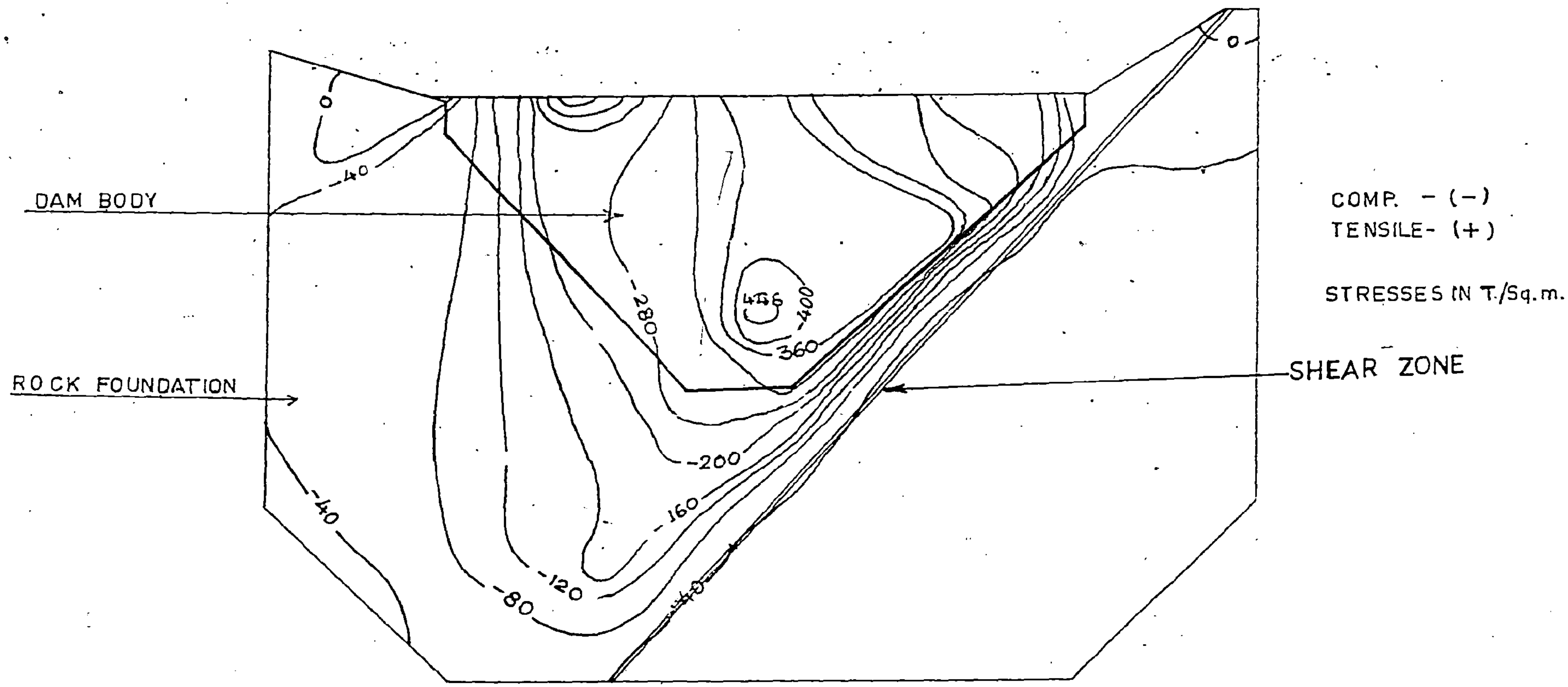


FIG 6·15. MAJOR PRINCIPAL STRESSES ALONG U/S FACE (SECTION A-A)

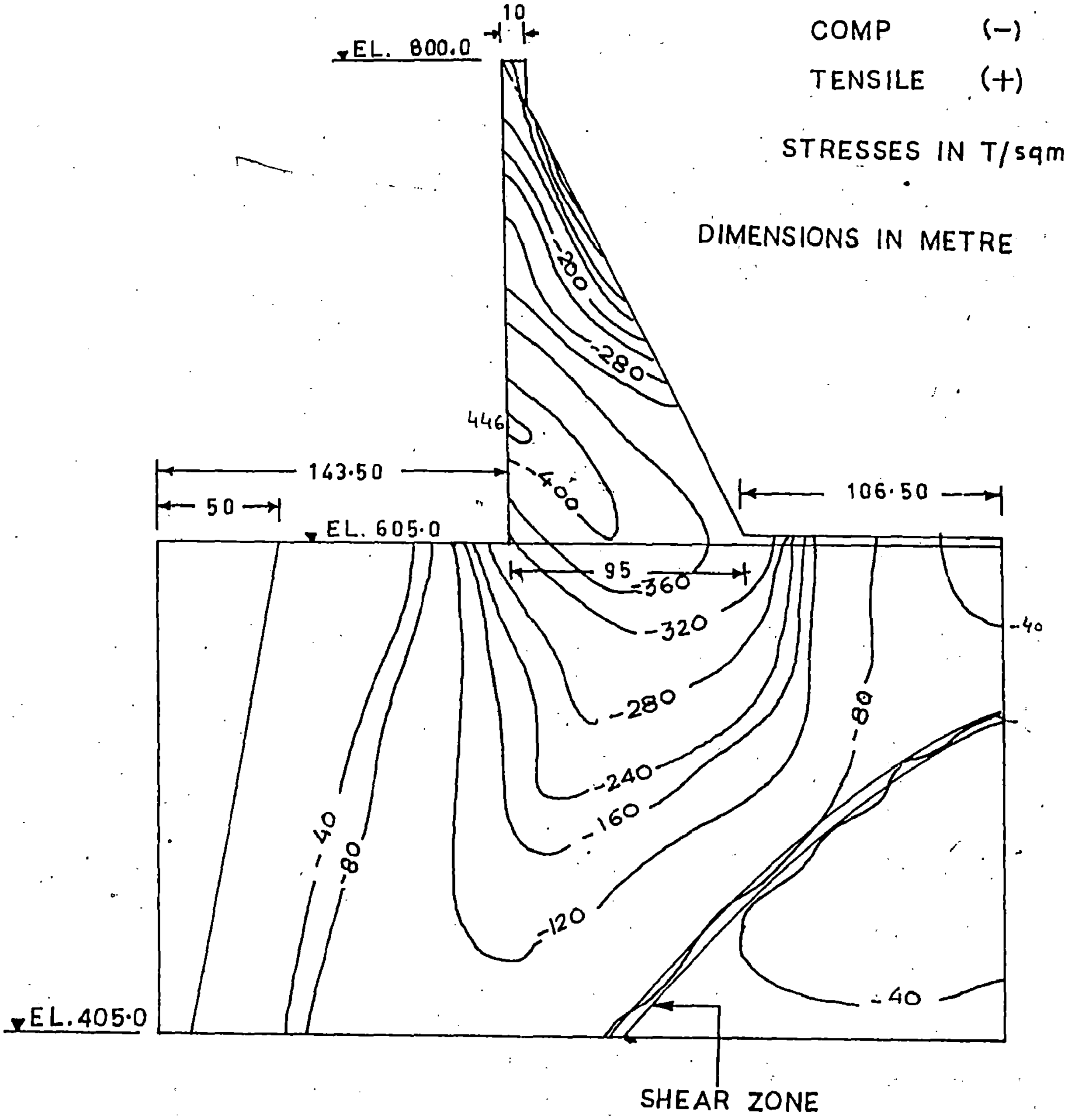


FIG 6·16. MAJOR PRINCIPAL STRESSES IN DAM BODY AND FOUNDATION ROCK AT SECTION B–B

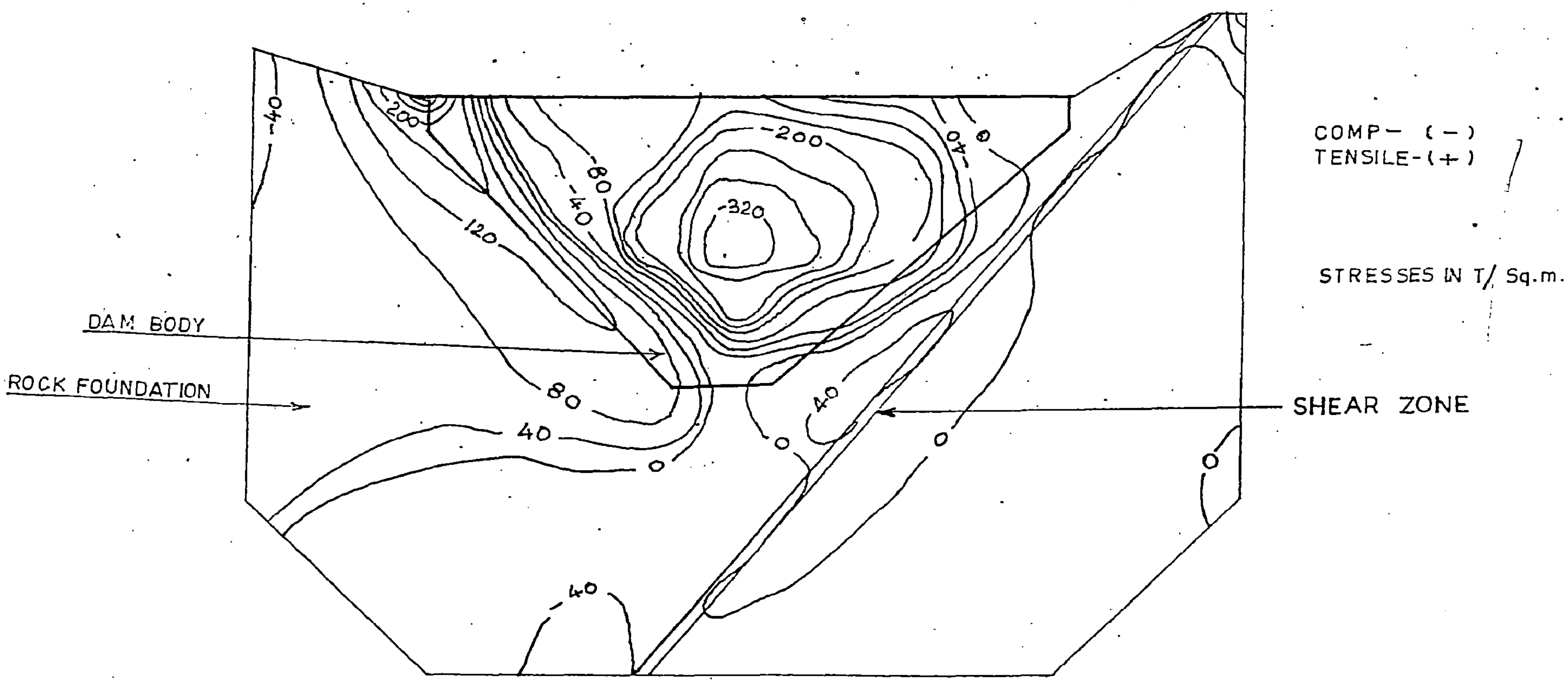


FIG 6·17. MINOR PRINCIPAL STRESSES ALONG U/S FACE (SECTION A-A)

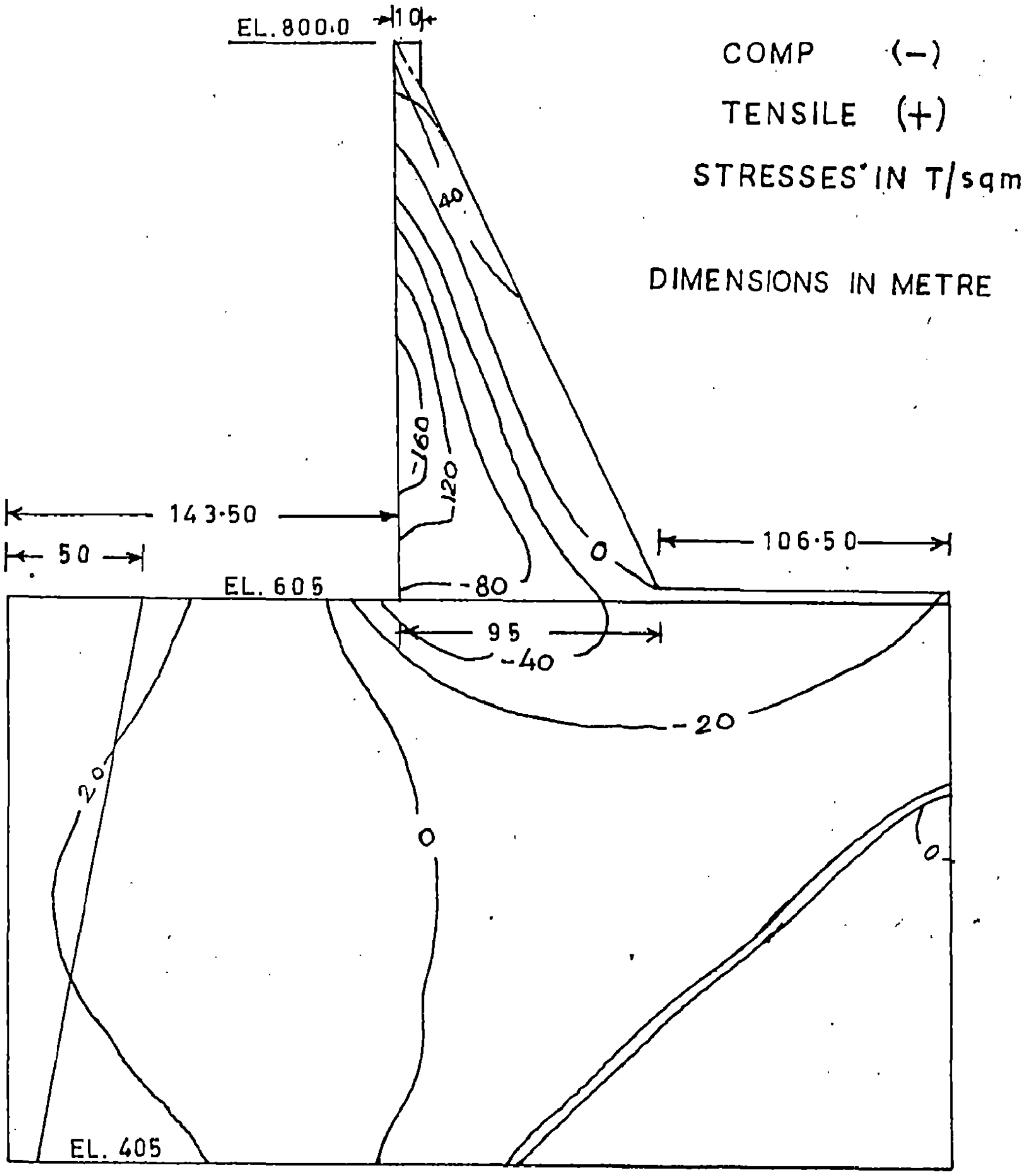


FIG 6·18. MINOR PRINCIPAL STRESSES IN DAM BODY AND FOUNDATION ROCK AT SECTION B-B

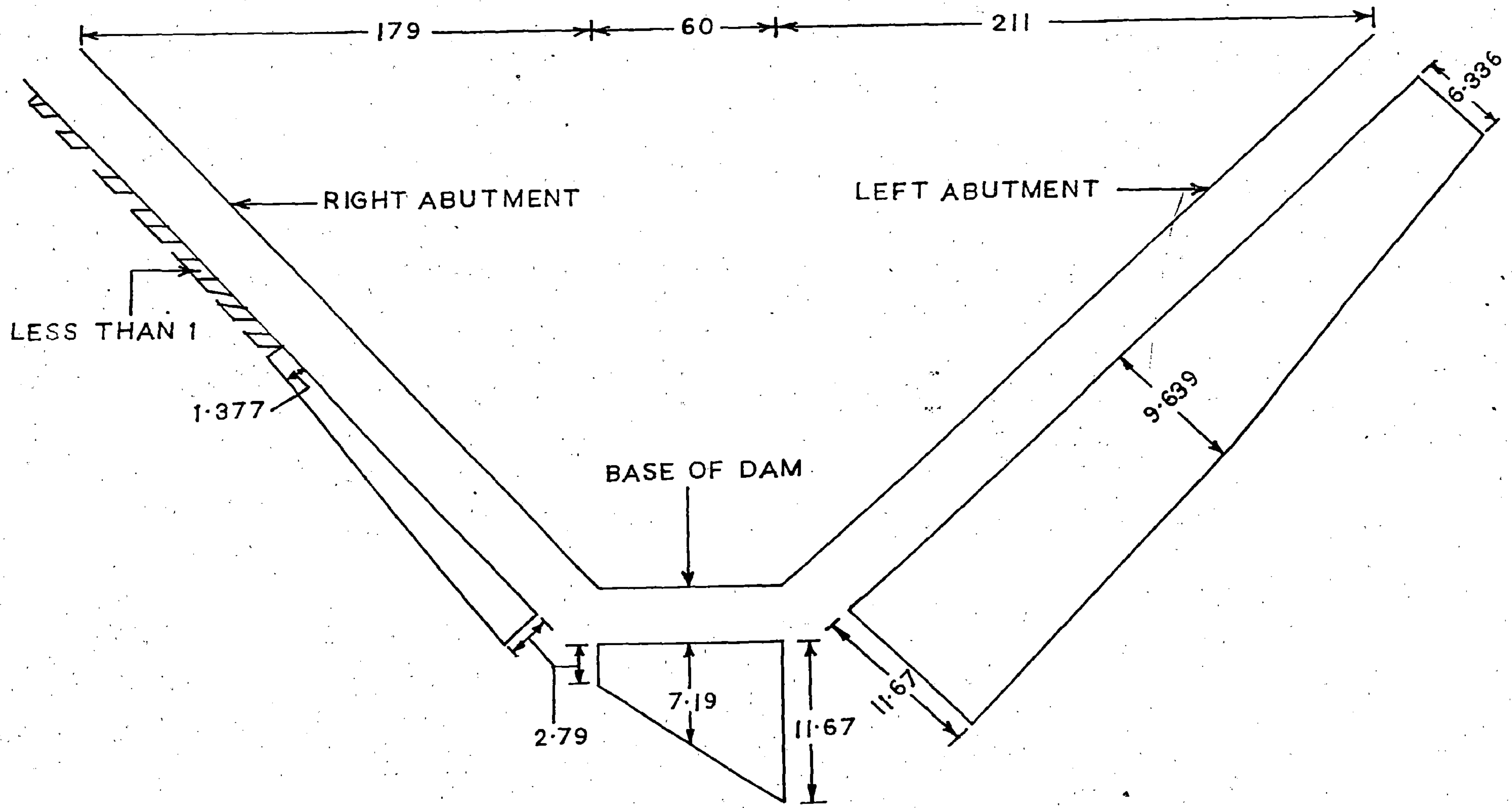


FIG 6·19. SHEAR MOBILISATION FACTOR ALONG CONTACT PLANE

CASES

(A) WITH EXTREME U/S AND D/S BOUNDARY FREE.

(B) ALL BOUNDARIES RESTRAINED

(C) CASE (B) WITH ED ADOPTED IN 2-D FEM ANALYSIS.

(D) 2-D FEM ANALYSIS FOR GRAVITY SECTION.

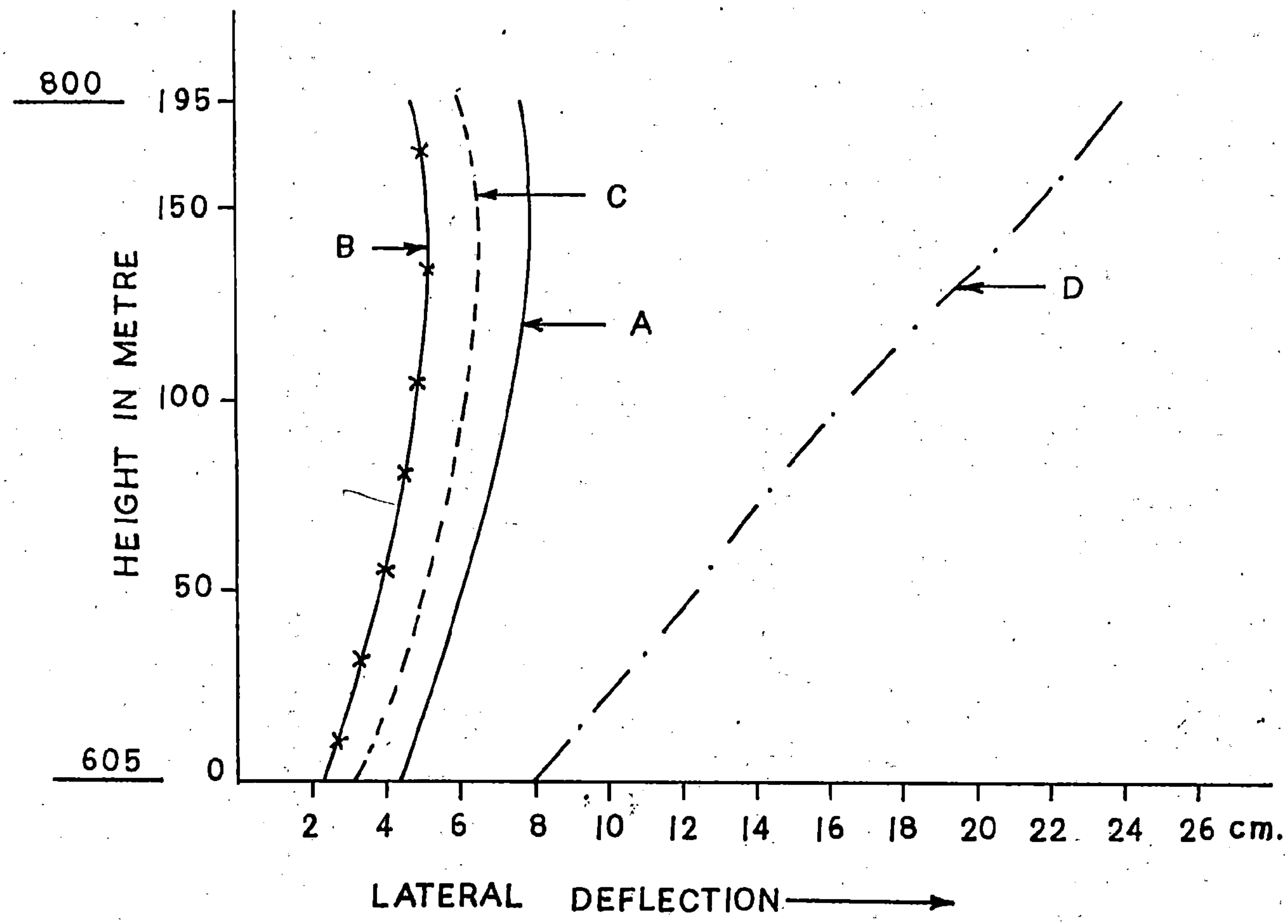


FIG. 6·20. COMPARISON OF RESULTS OF 2-D FEM ANALYSIS FOR GRAVITY SECTION AND 3-D FEM ANALYSIS FOR ARCH CUM GRAVITY SECTION AT B–B

Appendix I   Details of Deformation Modulus (Ed) Tests and their Results for trap Rock

Sl. No.	Agency which Conducted the tests	Test Report No./Year	Type of Test	Location of Test	Condition & Direction of test	Maximum Stress level in kg/cm²	No. of cycles	Modulus of Deformation (Ed) (Kg/cm²)			Modulus of Elasticity (Ee) (kg/cm²)	Remark
								As per 2nd Cycle		As per Total Deformation		
								Stress Level	Value			
1	2	3	4	5	6	7	8	9	10	11	12	13
1.	T&C Dn DKP	9/82	UJT	Drift L-7 At RL 790	N V	62.1	2	62.1	4680	2270	30000	UJT Uniaxial
2.	T&C Dn DKP	9/82	UJT	Drift L-7	N H	62.1	2	62.1	5440	4610	60900	Jacking test
3.	T&C Dn DKP	20/82	UJT	Drift L-3 At RL 776	N V	62.1	2	62.1	16820	6430	57200	FJT Flat
4.	T&C Dn DKP	20/82	UJT	Drift L-3	N H	62.1	2	62.1	19020	11620	66100	Jacking test
5.	T&C Dn DKP	35/82	FJT	Drift L-7 At RL 790	N V	20.0	1	20.0	25500	25500	N.A.	N - Natural
6.	T&C Dn DKP	35/82	FJT	Drift L-7	N H	20.0	1	20.0	30300	30300	N.A.	Condition
7.	T&C Dn DKP	11/86	UJT	Drift L-16 At RL 707	S V	70.7	5	28.3	6400	7000	36300	V - Vertical
8.	T&C Dn DKP	13/86	UJT	Drift L-16	S H	70.7	5	28.3	13210	11330	40000	H-Horizontal
9.	T&C Dn DKP	15/86	UJT	Drift L-16	S V	70.7	5	28.3	7420	9220	30300	S-Saturated
10.	T&C Dn DKP	16/86	UJT	Drift L-16	S V	67.2	5	24.7	8000	15110	61900	PG-Post Grouted
11.	T&C Dn DKP	18/86	UJT	Drift L-16	S H	70.7	5	28.3	24330	24760	94900	NA - Not
12.	T&C Dn DKP	19/86	UJT	Drift L-16	S H	70.7	5	28.3	25680	13870	43500	available
13.	T&C Dn DKP	20/86	UJT	Drift L-16	S H	70.7	5	28.3	4560	5890	13900	U.R - UNDER
14.	T&C Dn DKP	21/86	U	Drift L-17 At RL 642	S H	70.7	5	28.3	51360	41770	78000	RIVER RL 580
15.	T&C Dn DKP	25/86	UJT	Open Terrace L/B RL 640	S V	65.4	5	28.3	33820	18640	94000	R/B - RIGHT BANK
16.	T&C Dn DKP	26/86	UJT	Open Terrace L/B RL 640	S V	65.4	5	28.3	22010	16530	100000	L/B - LEFT BANK
17.	T&C Dn DKP	27/86	UJT	Open Terrace L/B RL 648	S V	65.4	5	30.0	23760	16440	91000	
18.	T&C Dn DKP	28/86	UJT	Open Terrace L/B RL 640	PG V	65.4	5	28.3	42020	18860	110000	
19.	T&C Dn DKP	29/86	UJT	Drift L-16	S V	70.7	5	28.3	17780	14390	66000	
20.	T&C Dn DKP	30/86	UJT	Drift L-16	S V	70.7	5	28.3	27730	21660	54000	
21.	T&C Dn DKP	31/86	UJT	Drift L-16	S H	70.7	5	28.3	55460	50980	87000	
22.	T&C Dn DKP	32/86	UJT	Drift L-16	S H	70.7	5	28.3	27730	26660	30000	

--- contd

1	2	3	4	5	6	7	8	9	10	11	12	13
23.	T&C Dn DKP	33/86	UJT	Drift L-16	S H	70.7	5	28.3	18490	23900	85000	
24.	T&C Dn DKP	34/86	UJT	Drift L-16	S H	70.7	5	28.3	35550	24760	56000	
25.	T&C Dn DKP	35/86	UJT	Drift L-16	S V	70.7	5	28.3	15410	12840	86000	
26.	T&C Dn DKP	36/86	UJT	Drift L-16	S H	70.7	5	28.3	7000	12300	33000	
27.	T&C Dn DKP	42/86	UJT	Open Terrace L/B RL 680	PG V	34.6	5	14.2	15070	14770	82000	
28.	T&C Dn DKP	16/87	UJT	Drift L-15 At RL 642	S V	63.6	2	63.6	28890	15920	59000	
29.	T&C Dn DKP	17/87	UJT	Drift L-15	S V	63.6	2	63.6	54740	34290	97000	
30.	T&C Dn DKP	18/87	UJT	Drift L-15	S H	63.6	2	63.6	65090	53150	96000	
31.	T&C Dn DKP	21/87	UJT	Drift L-16 At RL 707	PG H	63.6	2	63.6	38890	12730	62000	
32.	T&C Dn DKP	21/87	UJT	Drift L-16	PG V	63.6	2	63.6	8910	5200	38000	
33.	T&C Dn DKP	24/87	UJT	Drift L-16	PG V	63.6	2	63.6	14720	8860	41000	
34.	T&C Dn DKP	24/87	UJT	Drift L-16	PG H	63.6	2	63.6	34660	21370	57000	
35.	T&C Dn DKP	25/87	UJT	Drift L-16	PG V	63.6	2	63.6	7610	4950	45000	
36.	CWPRS PUNE	740/90	FJT	U.R.DRIFT	S V	94.1	3	70.6	129700	105000	143440	
37.	CWPRS PUNE	740/90	FJT	U.R.DRIFT	S V	88.2	3	70.6	197570	172200	217600	
38.	CWPRS PUNE	740/90	FJT	U.R.DRIFT	S V	82.4	3	50.0	26250	16430	29710	
39.	CWPRS PUNE	740/90	FJT	U.R.DRIFT	S V	94.1	3	70.6	80230	40810	90400	
40.	CWPRS PUNE	740/90	FJT	U.R.DRIFT	S V	70.6	3	50.0	306250	264750	311470	
41	CSMRS DELHI	8/89	UJT	Drift L-15 At RL 642	S V	50.0	4	20.0	111	40130	85350	
42	CSMRS DELHI	8/89	UJT	Drift L-15	S V	50.0	3	40.0	69560	72700	107860	Value of Ed
43	CSMRS DELHI	8/89	UJT	Drift L-15	S V	50.0	5	20.0	23530	23530	50220	for total
44	CSMRS DELHI	8/89	UJT	Drift L-15	S V	40.0	4	20.0	28500	28500	53240	deformation
45.	CSMRS DELHI	8/89	UJT	Drift L-15	S V	40.0	4	20.0	44500	44500	56420	not available
46.	CSMRS DELHI	8/89	UJT	Drift L-15	S V	40.0	4	20.0	57300	57300	79600	Hence value
47.	CSMRS DELHI	8/89	UJT	Drift L-15	S V	40.0	4	20.0	46030	46030	83000	of 2nd cycle
48.	CSMRS DELHI	8/89	UJT	Drift L-15	S V	40.0	4	20.0	26100	26100	34330	taken
49.	T&C Dn DKP	22/86	UJT	Drift L-17 At RL 642	S V	70.7	5	28.3	13000	13750	96000	
50.	T&C Dn DKP	23/86	UJT	Drift L-17	S V	70.7	5	28.3	21660	30600	97000	
51.	T&C Dn DKP	24/86	UJT	Drift L-17	S H	70.7	5	28.3	29500	26260	80220	
52.	T&C Dn DKP	44/86	UJT	Drift L-17	S H	70.7	5	28.3	46220	3537[illegible]	[illegible]	
53.	IRI ROORKEE	57/86	UJT	Drift L-16	S V	73.4	5	35.0	7620	5950	26000	

Appendix – II Details of Deformation Modulus (Ed) Tests and their Results for Thinly Foliated Slates

Sl. No.	Agency which Conducted the tests	Test Report No./Year	Type of Test	Location of Test	Condition Direction of Test	Maximum Stress level in kg/cm²	No. of cycles	Modulus of Deformation (Ed) (Kg/cm²) As per 2nd Cycle Stress Level	Value	As per Total Defor-mation	Modulus of Elasticity (Ee) (kg/cm²)	Remark
1	2	3	4	5	6	7	8	9	10	11	12	13
1.	T&C Dn DKP	50/87	UJT	OPEN TERRACE R/B RL 660	N V	17.6	1	17.6	1200	1200	16500	
2.	T&C Dn DKP	50/88	UJT	Drift R-8	S V	28.3	2	28.3	3150	1710	23100	
3.	T&C Dn DKP	4/89	UJT	Drift R-8	S V	63.7	2	63.7	2890	1820	20790	
4.	T&C Dn DXP	7/90	UJT	U.R.DRIFT RL 580	S V	24.7	5	10.6	1440	760	24760	
5.	T&C Dn DXP	9/90	JJT	U.R.DRIFT	S V	44.2	2	44.2	12040	3940	26100	
6.	T&C Dn DXP	11/90	UJT	U.R.DRIFT	S V	56.6	2	56.6	14600	6680	60280	At Bottom
7.	T&C Dn DXP	11/90	UJT	U.R.DRIFT	S V	28.4	2	28.4	6460	2730	17770	At top
8.	CWPRS PUNE	2740/90	FJT	U.R.DRIFT	S V	45.3	4	30.2	2100	1190	8710	
9.	CWPRS PUNE	2740/90	FJT	U.R.DRIFT	S V	23.5	2	23.5	3300	1870	9790	
10.	CWPRS PUNE	2740/90	FJT	U.R.DRIFT	S V	3.8	3	3.2	3490	2280	3750	
11.	CWPRS PUNE	2740/90	FJT	U.R.DRIFT	S V	7.6	3	4.5	1990	1430	3670	
12.	T&C Dn DXP	10/90	UJT	U.R.DRIFT	S V	56.6	2	28.3	!!!	!!!	!!!	!!! Not considered due to wide variation of Dial Gauges

Appendix – III

Details of Deformation Modulus (Ed) Tests and their Results for Quartzitic Slates

Sl. No.	Agency which Conducted the tests	Test Report No./Year	Type of Test	Location of Test	Condition Direction of Test	Maximum Stress level in kg/cm²	No. of cycles	Modulus of Deformation (Ed) (Kg/cm²) As per 2nd Cycle		Modulus of Deformation (Ed) (Kg/cm²) As per Total Deformation	Modulus of Elasticity (Ee) (kg/cm²)	Remark
								Stress Level	Value			
1	2	3	4	5	6	7	8	9	10	11	12	13
1.	T&C Dn DKP	37/86	UJT	OPEN BENCH R/B RL650	S V	65.4	5	28.3	22370	11900	59000	
2.	T&C Dn DKP	41/86	UJT	OPEN BENCH R/B RL 650	S V	63.6	5	28.3	6730	8040	37000	
3.	T&C Dn DKP	2/87	UJT	OPEN BENCH R/B RL 770	S V	30.0	2	30.0	22320	7750	71850	
4.	T&C Dn DKP	6/87	UJT	OPEN BENCH R/B RL 770	S V	30.0	2	30.0	40920	30070	70140	
5.	T&C Dn DKP	13/87	UJT	OPEN BENCH R/B RL 740	S V	33.6	2	33.6	17900	8490	43320	
6.	T&C Dn DKP	22/87	UJT	OPEN BENCH R/B RL 740	S V	31.8	2	31.8	13800	8250	42150	

Appendix - IV.

Shear test data for tests conducted between trap rock and concrete

(A) Natural Condition.

Sl No.	Drift Block No.	Cohesion C in Kg/Cm²	Friction angle ø (in Degrees)
1.	R-3	0.80	60.50
2.	R-4	2.20	52.50
3.	R-1	3.00	50.00
4.	L-1	2.20	55.00
5.	L-Y	1.45	58.50
6.	R-2	4.00	48.00
7.	L-3	5.40	46.00
8.	R-9	5.00	44.00
9.	L-7	4.40	46.00
10.	B-10-11	1.71	48.12
11.	B-17-19	2.93	48.80
12.	B-10	1.50	49.65
13.	B-15	2.52,2.50	45/44.92

(B) Saturated Condition

Sl. No.	Drift Block No.	Cohesion C in kg/Cm²	Friction angle ø (in Degrees)
1.	Left Bank bench	1.533	54.3

Appendix - V.

Shear test data for tests conducted on trap rock

(A) Natural Condition

Sl. No.	Site	Cohesion C in Kg/Cm^2	Friction angle ϕ (in Degrees)
1.	Open terrace El. 680 Left Bank	3.60	60.0
2.	Open terrace El. 640 Left Bank	3.00	53.0
3.	Drift L-15 El. 640 Left Bank	2.114	46.3
4.	Open terrace El. 740 Left Bank	3.80	58.0
5.	Open terrace El.640 Left Bank	1.03	58.6
6.	Open terrace El. 640 Left Bank	2.10	46.0
7.	Drift L-15 El. 640 Left Bank	0.66	59.4
8.	Drift R-3 El. 650 Right Bank	3.13	37.5
9.	Drift R-3 El. 650 Right Bank	2.37	37.5
10.	Drift R-3 El. 650 Right Bank	10.80	46.4
11.	Drift R-3 El. 650 Right Bank	1.51	63.0
12.	Drift L-4 El. 715 Left Bank	3.17	55.7
13.	Drift L-4 El. 715 Left Bank	1.971	55.8
14.	Open terrace El. 680 Left Bank	5.07	54.0

15. Drift L-4 El. 715 Left Bank	5.16	58.5
16. Open terrace El. 740 Left Bank	3.58	55.7
17. Drift L-4 El. 715 Left Bank	4.84	58.3
18. Drift L-4 El. 715 Left Bank	5.40	54.5
19. Drift L-16 El. 707 Left Bank	3.02	54.2
20. Drift L-16 El. 707 Left Bank	1.65	54.0
21. Drift L-16 El. 707 Left Bank	4.53	57.0
22. Drift L-16 El. 707 Left Bank	2.11	42.4

(B) Saturated Condition

1. Open terrace El. 680 Left Bank	5.50	60.0
2. Drift L-15	5.35	55.0
3. Drift L-16	10.00	52.0

(C) Post Grouted Condition

1. Drift L-15 El. 640 Left Bank	3.28	58.0
2. Drift L-15 El. 640 Left Bank	2.82	60.7

Appendix VI.

Shear test data for tests conducted between Xenolith rock and concrete.

(A) Natural Condition.

Sl. No.	Drift Block No.	Cohesion C in kg/Cm²	Friction angle ø (in Degrees)
1.	L-x	3.1Ø	52.5
2.	B-12-13	2.75	44.43
3.	B-14-15	1.54	38.5Ø
4.	B-15	2.Ø6	39.65

(B) Saturated Condition

1.	Site No. 125	2.34	37.5Ø
2.	Site No. 121	Ø.7Ø	44.ØØ

Appendix VII.

Shear test data for tests conducted on Xenolith rock.

(A) Natural Condition

Sl. No.	Site	Cohsion C in kg/Cm²	Friction angle ø (in Degrees)
1.	Open terrace El. 690 Left Bank	1.22	42.50
2.	Open terrace El. 700 Left Bank	6.20	47.00
3.	Open terrace El. 720 Left Bank	2.34	45.28
4.	Open terrace El. 700 Left Bank	2.33	46.38
5.	Open terrace El. 700 Left Bank	6.00	50.00

(B) Saturated Condition

1.	Open terrace El. 720 Left Bank	3.51	51.70
2.	Open terrace El. 700 Left Bank	1.44	43.80

Appendix VIII.

Shear test data for tests conducted between state rock and concrete

Sl. No.	Drift Block No.	Cohesion C in kg/Cm²	Friction angle ø (in Degrees)
(A) Natural			
1.	R-8	Ø.32	57.6
(b) Saturated		Nil	Nil
(c) Post Grouted		Nil	Nil

Appendix IX .

Shear test data for tests conducted between quartzitic slate and concrete

Sl. No.	Drift Block No.	Cohesion C in kg/Cm²	Friction angle ø (in Degrees)
(A) Natural		Nil	Nil
(B) Saturated			
1.	L.B.Bench	1.45	53.Ø
2.	L.B.Bench	2.237	49.5

Appendix X.

Shear test data for tests conducted for slate rock

Sl. No.	Drift Block No.	Cohesion C in kg/Cm^2	Friction angle ø (in Degrees)
A. Natural			
B. Saturated			
1.	R-B, Rl. 696 R/B bench	2.63	23.Ø
2.	R-8,Rl.696 R/B bench	1.58	37.8
3.	R-8,Rl.696	1.Ø1	52.Ø

Appendix XI.

Shear test data for tests conducted on quartzitic slate

A. Natural			
1.	El. 678 L/B bench	8.2Ø	46.Ø
2.	El. 678	8.ØØ	46.Ø
B. Saturated			
1.	El. 678 L/B bench	4.35	58.6

Appendix XII.

TRIAXIAL COMPRESSION TESTS ON SLATE SPECIMEN

Location : The slate specimen was taken from the under river drift below the proposed Lakhwar Dam site. The depth of drift below river bed is about 45 m and the principal insitu stresses at that locations are σh_1 = 162 . σh_2 = 11Ø & σv_3 = 9Ø kg/Cm^2.

The specimen was prepared and tested in Rock Mechanics Lab of university of Roorkee, Roorkee.

Size of Specimen : 37 mm diameter and 8Ømm long.

Observations :

Sl. No.	σ_3 (kg/Cm^2	strain, ϵ (mm)	(%)	Vertical load (KN)	$\sigma_1-\sigma_3$ (kg/Cm^2
1.	75	Ø	Ø	Ø.6	ØØ5.57
2.	75	Ø.1	Ø.125	12.3	114.35
3.	75	Ø:2	Ø.25Ø	12.9	119.92
4.	75	Ø.3	Ø.375	13.Ø	12Ø.85
5.	75	Ø.4	Ø.5ØØ	13.2	122.71
6.	75	Ø.5	Ø.625	13.3	123.65
7.	75	1.6	2.ØØØ	13.8	128.29
8.	75	1.7	2.125	14.3	132.94
9.	75	2.3	2.875	14.8	137.59
1Ø.	75	2.9	3.625	15.1	14Ø.38