

ANALYSIS OF ARCH DAM UNDER SEISMIC INFLUENCE

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ABSTRACT

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In this report, a procedure to analyse an arch dam under the action of earthquake forces, and a design method to withstand the effect of earthquakes are presented. The method can be applied when the reservoir is partially or completely empty and the action of earthquake is such that it tends to move the dam towards the reservoir. Under this situation the classical method of analysis fails, because the arch elements are in tension and by assumptions that tensile strength of concrete is negligible, they do not participate in resisting the earthquake forces. Under severe seismic action, the arch dam cantilever elements may shake loose of the vertical construction joints and vibrate as a set of independent cantilevers. The design of a "reinforced arch" to limit the stress level and to prevent excessive deformation of the dam, due to this independent cantilever action is presented in this report.

ACKNOWLEDGEMENTS

ACKNOWLEDGEMENTS

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TABLE OF CONTENTS

TABLE OF CONTENTS

	PAGE
ABSTRACT	i
ACKNOWLEDGEMENTS	ii
LIST OF TABLES	vi
LIST OF FIGURES	vii
LIST OF DIAGRAMS	viii
NOTATIONS	ix
I INTRODUCTION	1
II INDEPENDENT CANTILEVER VIBRATIONS	6
III FOUNDATION AND ABUTMENT DEFORMATION CONSTANTS	9
3.1 Assumptions	9
3.2 Deformation Equations	12
3.3 Diagrams	16
3.4 Spring Constants	18
IV STRESSES IN THE DAM DUE TO EARTHQUAKE	30
V LOCATION OF THE "REINFORCED ARCH"	36
VI REINFORCED ARCH ANALYSIS; ARCH AND CANTILEVER ADJUSTMENT METHOD	38
6.1 Assumptions	38
6.2 Trial and Error Method	39
6.2.1 Unit load method	40
6.2.2 Adjustment equations	41
VII COMPUTER ANALYSIS RESULTS	49
VIII DESIGN OF REINFORCEMENT	50
8.1 Detailed Design	50
IX CONCLUSION	54

	PAGE
REFERENCES	55
APPENDIX "A" - COMPUTER ANALYSIS	56
A-I Block Deformation Calculation	56
A-II Arch Analysis	80
A-III Plate Diagrams	92
<u>Plate 1A</u> - Maximum compressive and tensile stresses under own weight - cantilever analysis	93
<u>Plate 1B</u> - Maximum compressive and tensile stresses under earthquake - cantilever analysis	95
<u>Plate 1C</u> - Maximum cracked compressive stress at any point in the dam under earthquake cantilever analysis	97
<u>Plate 2</u> - Idikki Arch Dam - plan view and loca- tion of the dam	99
<u>Plate 3</u> - Idikki Arch Dam - geometrical defini- tion	101
<u>Plate 4</u> - Idikki Arch Dam - elevation and pro- jection on y-y axis	103
<u>Plate 5A</u> - Idikki Arch Dam - Cantilevers 1 to 8.	105
<u>Plate 5B</u> - Idikki Arch Dam - Cantilevers 9 to 15..	107
<u>Plate 6A</u> - Idikki Arch Dam - Arches el. 2285 to el. 2415	109
<u>Plate 6B</u> - Idikki Arch Dam - Arches el. 1980 to el. 2220	111
<u>Plate 7</u> - Principal stresses in the dam - Hydro- static load, dead load, temperature, and 0.10 G earthquake	113
A-IV Drawings	114

PAGE

<u>Drawing 1</u> - Idikki Dam - Seismic Bars, forces and displacements at elevation 2140'	116
<u>Drawing 2</u> - Idikki Dam - Seismic Bars, rein- forcement layout	118

LIST OF TABLES

LIST OF TABLES

NUMBER	DESCRIPTION	PAGE
3.1	Foundation Constants - Cantilever Elements	23
3.2	Foundation Constants - Unit Horizontal Elements	24
3.3	Arch Abutment Constants	24
4.1	Stresses in the block due to own weight, (left bank)	34
4.2	Stresses in the block due to own weight, (right bank)	35
6.1	Load in the arch element due to seismic action, by trial and error method and unit load	47
6.2	Stresses in the dam under earthquake intensity .05 G with reinforced archy	48

LIST OF FIGURES

LIST OF FIGURES

NUMBER	DESCRIPTION	PAGE
1.1	Key Elevation - Idikki Dam	3
1.2	Idikki Dam - view from upstream	4
1.3	Idikki Dam during construction, view from top	5
2.1	Independent Cantilever Failure and Reinforced Arch	8
3.1	Idikki Dam during construction, view from downstream	11
3.2	Loaded area of the foundation surface	17
3.3	Unit vertical element	21
3.4	Unit horizontal element	21
4.1	Readjusted stresses - Assuming concrete cracks up to the point of zero tension	31
6.1	Arch and Cantilever Deformation	43
6.2	Cantilever element radial movement	45

LIST OF DIAGRAMS

DIAGRAMS

NUMBER	DESCRIPTION	PAGE
3.1	Value of 'k' in Equation (3.19) to calculate α'	25
3.2	Value of 'k' in Equation (3.20) to calculate β'	26
3.3	Value of 'k' in Equation (3.21) to calculate γ'	27
3.4	Value of 'k' in Equation (3.22) to calculate δ'	28
3.5	Value of 'k' in Equations (3.23) and (3.24) to calculate α'' and γ''	29

NOTATIONS

NOTATIONS

α' = average rotation in a plane normal to foundation surface due to bending moment load

β' = average deformation normal to foundation surface due to normal load

γ' = average deformation in plane of foundation surface due to tractive or shear load

δ' = average rotation in plane of foundation surface due to twisting moment load

α'' = average rotation in a plane normal to foundation surface due to tractive load

γ'' = average deformation in a plane of foundation due to bending moment load

μ = Poisson's ratio for foundation material

E_R = modulus of elasticity of foundation material in direct stress

p = normal or tractive load per unit area

a = short dimension of loaded area ab

b = long dimension of loaded area ab

M = bending moment, in direction of a , per unit length of b = $pa^2/6$

H = normal force per unit length of b = pa

V = shear force, in direction of a , per unit length
of b pa

M_T = twisting moment per unit length of b pa²/6

M_x = moment in vertical plane

M_y = moment in tangential plane

M_z = moment in horizontal plane

H = tangential force

V = horizontal force in vertical plane

W = vertical force

α' , γ' , α'' and γ'' are at right angles to the
directions of the foundation deformations not designated
by the circles.

θ_x = angular movement in vertical plane

θ_y = angular movement in tangential plane

θ_z = angular movement in horizontal plane

Δ_x = tangential movement

Δ_y = horizontal movement in vertical plane

Δ_z = vertical movement

- ψ = angle between vertical plane and plane of foundation surface at its intersection with the unit element
- h = height of the reinforced arch in feet
- KFX = force in pounds per unit displacement in feet in the X-direction (Fig. 3.4)
- KFY = force in pounds per unit displacement in feet in the Y-direction
- KMZ = moment in feet pounds per unit rotation in radians about the Z-axis

CHAPTER I
INTRODUCTION

CHAPTER I

INTRODUCTION

Idikki dam site, located in a narrow gorge of the Peryar River valley in South India, has steep banks extending 1,100 ft. above the river elevation. The width of the valley varies approximately from 30 ft at low water level at elevation 1,880, to 1,050 ft. at elevation 2,400 ft. The shape of the dam is defined by horizontal parabolas with focusses following in a vertical plane polynomial curve of the 6th degree. Arch at crest of the dam, which is located at elevation 2,415 has a length of 1,175 ft. The height of the dam is 535 ft. at crown section. The maximum water level is at elevation 2408.5 ft. The geometrical definition and other details of the dam are shown in Plates 2 to 8.

The analysis of the arch dam was done using the 'Trial Load Method'. In this method of analysis, the dam was assumed to be replaced by a number of horizontal and vertical sections called 'arches' and 'cantilevers'. This system was assumed to occupy the entire volume of the dam. The analysed structure then became a grid of intersecting arches and cantilevers (design elements).

The horizontal loads due to water pressure were divided out between the two systems of elements in such a way, as to satisfy conditions of equilibrium, and to produce equal arch and cantilever deflection at each cross point of the loaded structure. The vertical load including the weight of the water on the face of the dam, as well as the weight of the concrete, were assigned to the cantilever elements and were assumed to be transmitted to the foundation. Since the vertical construction joints in the dam were grouted before water load was applied, the structure was considered to act as a monolith and that arch action began as soon as the reservoir was loaded.

The 'Trial Load Method' was able to handle all loading conditions of the dam except when the dam was empty, and the action of the earthquake was upstream to downstream forcing the dam to move upstream. In this particular case, since the arches were pulled upstream, they were under tensile stress conditions and by the assumption that tensile strength in concrete was to be neglected, could not share any load, thus leaving the cantilevers to take the full earthquake load. This report deals with the 'Independent Cantilever Analysis' and the analysis and design of the 'Reinforced Arch' to withstand the effect of this particular earthquake action.

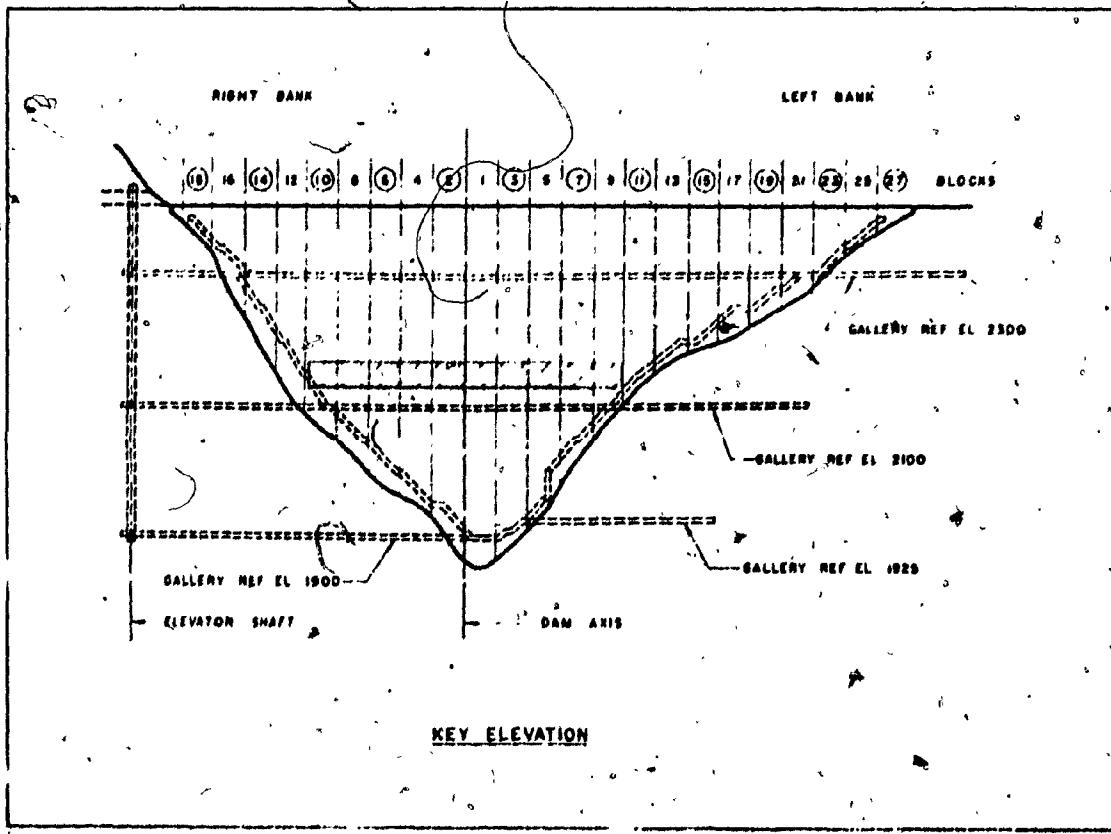


FIG. 1.1 Key elevation - Idikki Dam.

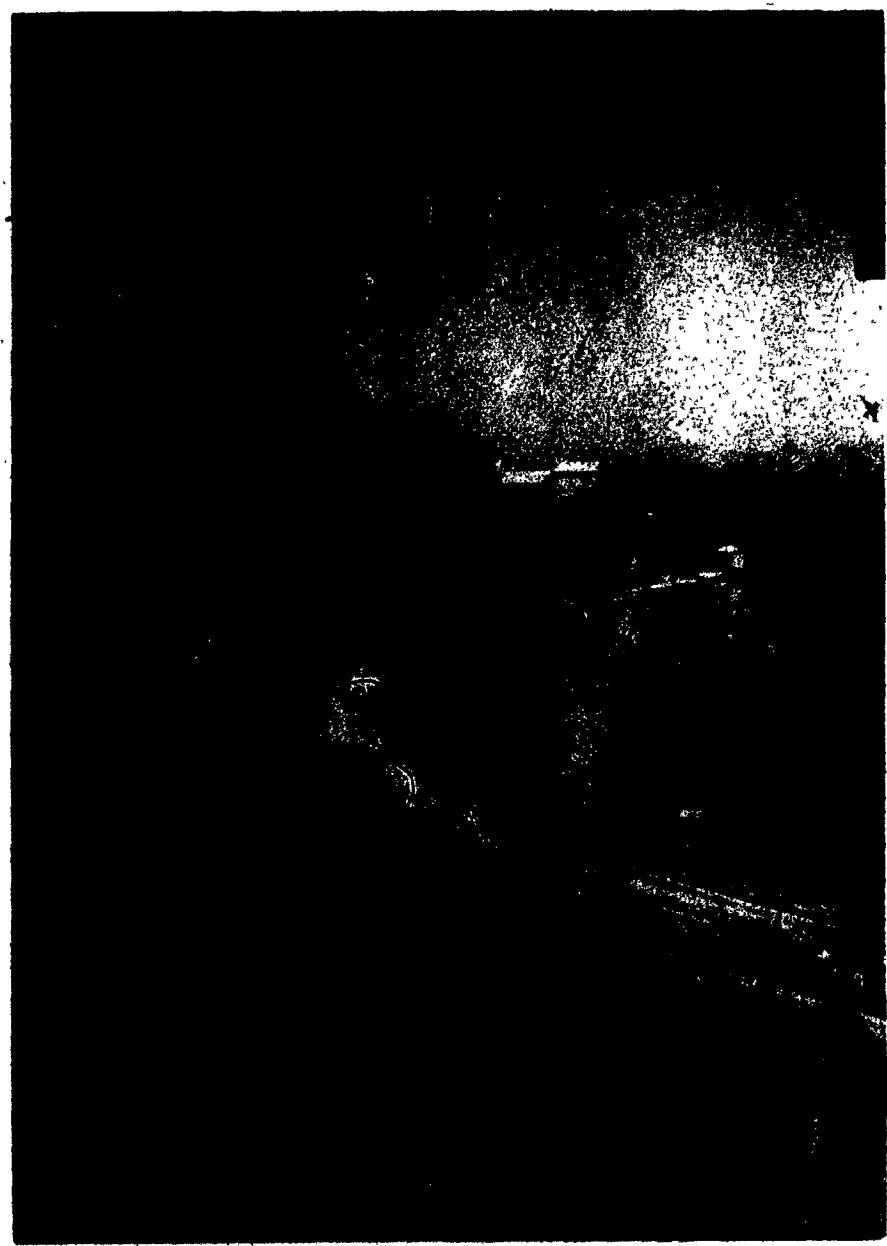


FIG. 1.2 Idikki Dam. View from upstream.



FIG. 4.3 Idikki Dam during construction.
View from top.

CHAPTER II
INDEPENDENT CANTILEVER VIBRATIONS

CHAPTER II

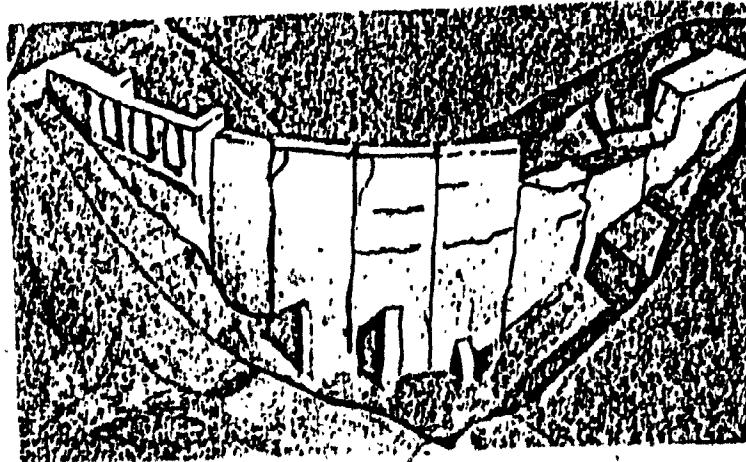
INDEPENDENT CANTILEVER VIBRATIONS

The arch dam was constructed in blocks with vertical construction joints in between the two adjacent blocks. The blocks were 50 to 55 ft. wide along the length of the arch. These joints were later grouted with cement mortar, when the height of the block reached a certain height of 60 ft. to 90 ft.

Vibration tests on earthquake resistance model of an arch dam carried out at the ISMES laboratory [8] brought out one particular possibility of failure. Under partial or no storage conditions, the upper part of each block which was not pressed together by the hydrostatic force has oscillated as a system of independent cantilevers. It has to be noted that the shear keys which provide links between the blocks were not reproduced in the tests and the failure occurred only after the loosening of the cantilever blocks. Even though the shear boxes and grouting could help retain the bond between blocks to a certain extent, due to the severe vibration and deflection of the dam under an earthquake of 0.05 G or more, reliability on the shear keys was somewhat doubtful.

The most obvious and economical solution would be to retain enough water in the reservoir to keep the structure pressed together by hydrostatic load. At the time of investigation it was not clear whether the field conditions would allow such a solution, to have water impounded at all times, even during construction.

Among different potential solutions, an arrangement which would prevent the excessive upstream deflection of the cantilevers under combined effect of gravity and earthquake was considered as a possible solution. The concept was to provide a continuous steel reinforcement running through the blocks and joints. The result was not a steel loop subjected to deformation, but a 'Reinforced Arch' on which the cantilevers can rest when pushed upstream under earthquake conditions.



Independent Cantilever Failure [8]

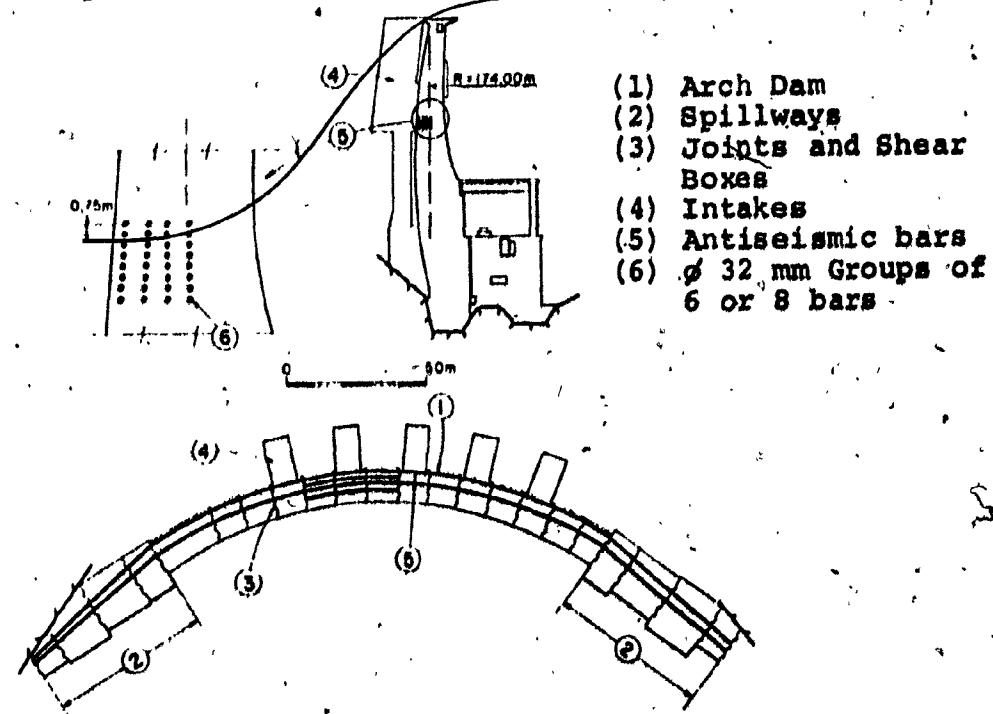


FIG. 2.1 Reinforced Arch - Rapel Dam [9]

CHAPTER III
FOUNDATION AND ABUTMENT DEFORMATION CONSTANTS

CHAPTER III

FOUNDATION AND ABUTMENT DEFORMATION CONSTANTS

The development of the diagrams and formulae for foundation deformation are discussed in this section. When the elastic movements in the foundation and abutment rock are included, the stresses are usually decreased at the abutments and foundation, but may be increased in other parts of the structure.

3.1 ASSUMPTIONS

The following are the major assumptions, to evaluate the foundation constants.

- 1) Foundation deformations are independent of the shape of the foundation surface.
- 2) Movements of a foundation area are due only to the loads applied directly on that area.
- 3) Water loads on the reservoir walls do not cause differential movements at the dam site. In other words, the effect of reservoir water load on the foundation and abutments are disregarded.

Thus the dam was assumed to be supported by a series of independent springs. The elastic constants of

the springs were determined by reference to deformation of an infinite foundation with plane surface. Based on these basic relations, Dr. Fredrik Vogt (Vogt, Dr. Fredrik "Veber die Berechnung der Fundament deformation", Det Norske Videnskaps Akademi, Oslo, 1925) developed equations for average deformation of the loaded rectangular area of the foundation surface. These fundamental equations express deformation due to a bending in a plane normal to the surface, a force normal to the surface, a tractive or shear force in the plane of the surface, and a twisting moment in the plane of the surface.

Using these equations the deformation of a plane rectangular foundation surface for various types of loadings can be evaluated. For ease of computation, each fundamental foundation equation is transformed into an expression that includes a function of two variables, Poisson's ratio and the ratio of the dimensions of the loaded surface. This function is multiplied by two or more constants to give foundation deformation. In order to determine these functions rapidly, curves have been developed by the United States Bureau of Reclamation, covering the range of values usually encountered in the concrete dams [6]. These curves are reprinted and shown at the end of this Chapter, in Diagrams 3.2 to 3.6.

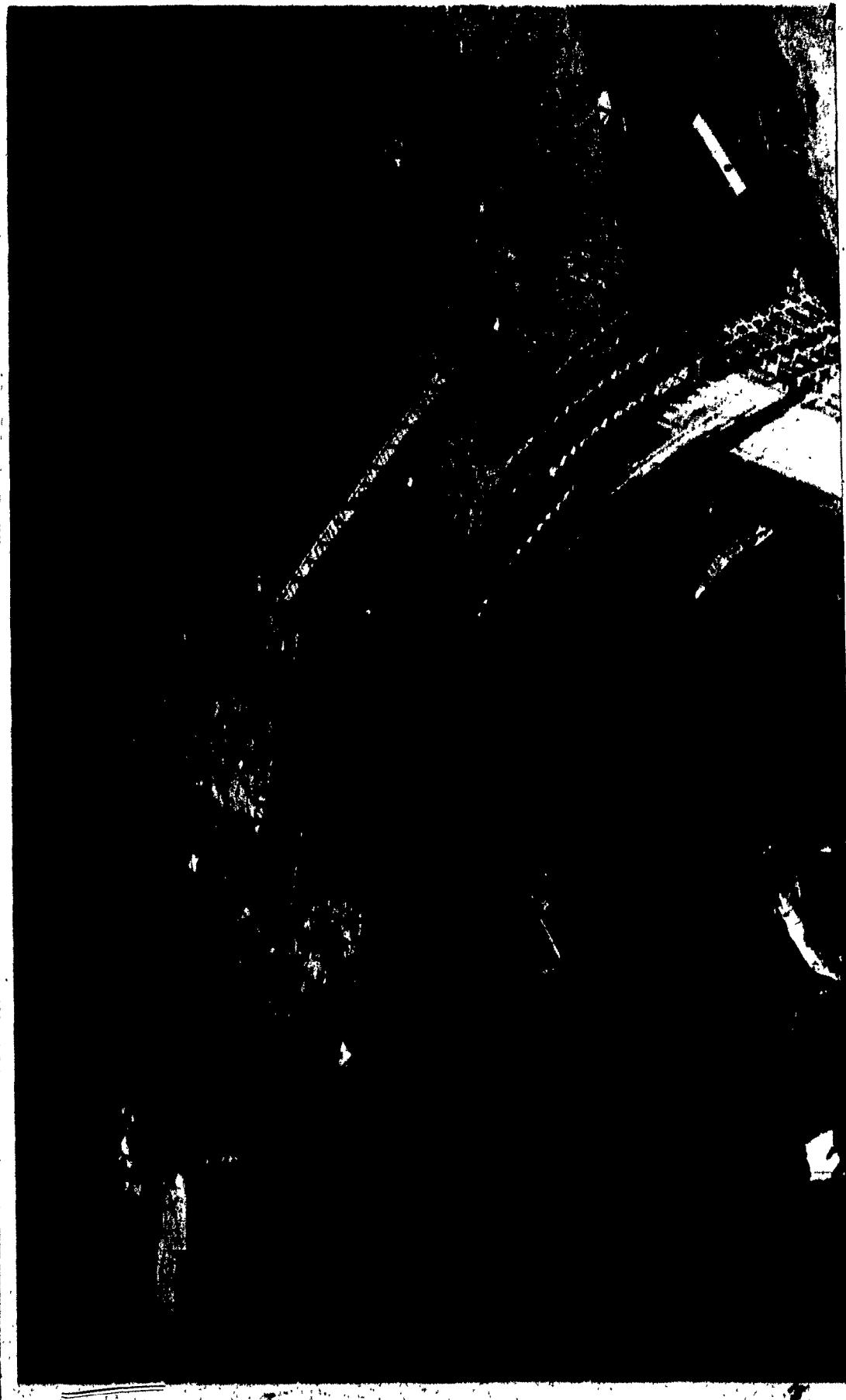


FIG. 3-1 Iditki Dam during construction.
View from downstream.

3.2 DEFORMATION EQUATIONS

The development of the equations are based on a uniform distribution of load in the direction of the long dimension, b , of a loaded rectangular area. (See Fig. 3.2, page 17). In the direction of the short dimension, a , the distribution of loading is linear for moment loading and uniform for shear and normal loading. Satisfying this condition, the average deformation of the area for any type of load is a function of the shape of the area, the load per unit area, and the elastic properties of the foundation material. The following equations give average deformations in terms of these variables. The shape of the area is expressed by the dimensions a and b , the load per unit area, by p and the elastic properties by ' μ ' Poisson's ratio, and E_R , the modulus of elasticity in direct stress.

$$\alpha' = (1-\mu^2) \frac{p}{E_R} \frac{3}{\pi} \left[\frac{4}{3} \cdot \frac{\sqrt{a^2+b^2}-a}{b} - \frac{8}{9} \cdot \frac{(\sqrt{a^2+b^2})^3 - (a^3+3/2b^3)}{a^2b} - \frac{8}{45} \cdot \frac{(\sqrt{a^2+b^2})^5 - (a^5+b^5)}{a^4b} + \frac{4}{3} \cdot \frac{b}{a} \cdot \log_e \left(\frac{a + \sqrt{a^2+b^2}}{b} \right) \right] \quad (3.1)$$

$$\beta' = (1-\mu^2) \frac{p}{E_R} \frac{2}{\pi} \left[a \cdot \log_e \left(\frac{b + \sqrt{a^2+b^2}}{a} \right) + b \cdot \log_e \left(\frac{a + \sqrt{a^2+b^2}}{b} \right) - \frac{(\sqrt{a^2+b^2})^3 - (a^3+b^3)}{3ab} \right] \quad (3.2)$$

$$\gamma' = (1-\mu^2) \frac{P}{E_R} \cdot \frac{2}{\pi} \left[\left(a \cdot \log_e \left(\frac{b + \sqrt{a^2+b^2}}{a} \right) + b \cdot \log_e \left(\frac{a + \sqrt{a^2+b^2}}{b} \right) - \frac{(a^2+b^2)}{3ab} - \frac{(a^3+b^3)}{1-\mu} \right) + \frac{\mu}{1-\mu} b \cdot \log_e \left(\frac{a + \sqrt{a^2+b^2}}{b} \right) - \frac{(\sqrt{a^2+b^2})(2b^2-a^2)}{3ab} - \frac{(2b^3-a^3)}{1-\mu} \right] \quad (3.3)$$

$$\delta' = \frac{P}{E_R} \cdot \frac{1}{30} \left[\left(90 - \frac{24a}{b} + \frac{5a^2}{b^2} - \frac{3a^4}{8b^4} + \frac{9a^6}{112b^6} \right) + \mu \left(90 - \frac{48a}{b} + \frac{15a^2}{b^2} - \frac{15a^4}{8b^4} + \frac{63a^6}{112b^6} \right) - \mu^2 \left(\frac{24a}{b} - \frac{10a^2}{b^2} + \frac{12a^4}{8b^4} - \frac{54a^6}{112b^6} \right) \right] \quad (3.4)$$

$$\alpha''' = (1-\mu-2\mu^2) \frac{P}{E_R} \left[1 - \frac{1}{2\pi} \left(4 \cdot \frac{a^2+b^2}{a^2} \cdot \tan^{-1} \left(\frac{a}{b} \right) - \frac{3b}{a} - \left(\frac{b}{a} \right)^3 \log_e \left(\frac{a^2+b^2}{b^2} \right) + \frac{a}{b} \cdot \log_e \left(\frac{a^2+b^2}{a^2} \right) \right) \right] \quad (3.5)$$

$$\gamma''' = (1-\mu-2\mu^2) \frac{P}{E_R} \cdot \frac{a}{6} \left[1 - \frac{1}{2\pi} \left(4 \cdot \frac{a^2+b^2}{a^2} \cdot \tan^{-1} \left(\frac{a}{b} \right) - \frac{3b}{a} - \left(\frac{b}{a} \right)^3 \log_e \left(\frac{a^2+b^2}{b^2} \right) + \frac{a}{b} \cdot \log_e \left(\frac{a^2+b^2}{a^2} \right) \right) \right] \quad (3.6)$$

Replacing b/a by x

$$\alpha' = (1-\mu^2) \frac{P}{E_R} \cdot \frac{4}{\pi} \cdot \frac{(\sqrt{1+x^2})(3-14x^2-2x^4)}{15x} + 15x^3 + 2x^5 - 3 + x \log_e \left(\frac{1 + \sqrt{1+x^2}}{x} \right) \quad (3.7)$$

$$\beta' = (1-\mu^2) \frac{p}{E_R} \cdot \frac{2}{\pi} \cdot a [\log_e(x + \sqrt{1+x^2}) + x \cdot \log_e(\frac{1+\sqrt{1+x^2}}{x}) - \frac{\sqrt{1+x^2})^3 - 1 - x^3}{3x}] \quad (3.8)$$

$$\gamma' = - \frac{p}{E_R} \cdot \frac{2}{\pi} \cdot a [(1-\mu^2) (\log_e(x + \sqrt{1+x^2}) + x \cdot \log_e(\frac{1+\sqrt{1+x^2}}{x}) - \frac{\sqrt{1+x^2})^3 - 1 - x^3}{3x}) + (u+u^2) \{x \cdot \log_e(\frac{1+\sqrt{1+x^2}}{x}) - \frac{(\sqrt{1+x^2})(2x^2-1) - 2x^3 + 1}{3x}\}] \quad (3.9)$$

$$\delta' = \frac{p}{E_R} \cdot \frac{1}{30\pi} [(90 - \frac{24}{x} + \frac{5}{x^2} - \frac{3}{8x^2} + \frac{9}{112x^6}) + \mu(90 - \frac{48}{x} + \frac{15}{x^2} - \frac{15}{8x^4} + \frac{63}{112x^6}) - \mu^2(\frac{24}{x} - \frac{10}{x^2} + \frac{12}{8x^4} - \frac{54}{112x^6})] \quad (3.10)$$

$$\alpha'' = (1-\mu-2\mu^2) \frac{p}{E_R} [1 - \frac{1}{2\pi} 4(1+x^2) \tan^{-1}(\frac{1}{x}) - 3x - x^3 \cdot \log_e(\frac{1+x^2}{x^2}) + \frac{1}{x} \cdot \log_e(1+x^2)] \quad (3.11)$$

$$\gamma''' = (1-\mu-2\mu^2) \frac{P}{E_R} \cdot \frac{a}{6} \left[1 - \frac{1}{2\pi} \{ 4(1+x^2) \tan^{-1} \left(\frac{1}{x} \right) - 3x - x^3 \cdot \log_e \left(\frac{1+x^2}{x^2} \right) + \frac{1}{x} \cdot \log_e (1+x^2) \} \right] \quad (3.12)$$

$$\alpha' = \frac{M}{E_R a^2} [(1-\mu^2) A] \quad (3.13)$$

$$\beta' = \frac{H}{E_R} [(1-\mu^2) B] \quad (3.14)$$

$$\gamma' = \frac{V}{E_R} [(1-\mu^2) B + (\mu+\mu^2) C] \quad (3.15)$$

$$\delta' = \frac{M_T}{E_R a^2} [E + \mu F - \mu^2 G] \quad (3.16)$$

$$\alpha'' = \frac{V}{E_R a} [(1-\mu-2\mu^2) D] \quad (3.17)$$

$$\gamma'' = \frac{M}{E_R a} [(1-\mu-2\mu^2) D] \quad (3.18)$$

The expressions in brackets are functions of μ and b/a . Replacing these functions by k and making a equal to T , width of dam, equations (3.13) to (3.18) become, for moments and forces of unity, per unit length,

$$\alpha' = \frac{k}{E_R T^2} \quad \text{where } k = (1-\mu^2)A \quad (3.19)$$

$$\beta' = \frac{k}{E_R} \quad \text{where } k = (1-\mu^2)B \quad (3.20)$$

$$\gamma' = \frac{k}{E_R} \quad \text{where } k = (1-\mu^2)B + \\ + (\mu+\mu^2)C \quad (3.21)$$

$$\delta' = \frac{k}{E_R T^2} \quad \text{where } k = (E+\mu F-\mu^2 G) \quad (3.22)$$

$$\alpha'' = \frac{k}{E_R T} \quad \text{where } k = (1-\mu-2\mu^2)D \quad (3.23)$$

$$\gamma'' = \frac{k}{E_R T} \quad \text{where } k = (1-\mu-2\mu^2)D \quad (3.24)$$

Note that expressions represented by k are different except for Equations (3.23) and (3.24).

3.3 DIAGRAMS

Values of k , in Equations (3.19) to (3.24) were computed for different combinations of μ , and b/a or x , and these curves are shown in Diagrams 1 to 5. For the application of curves to the analysis of a unit element of

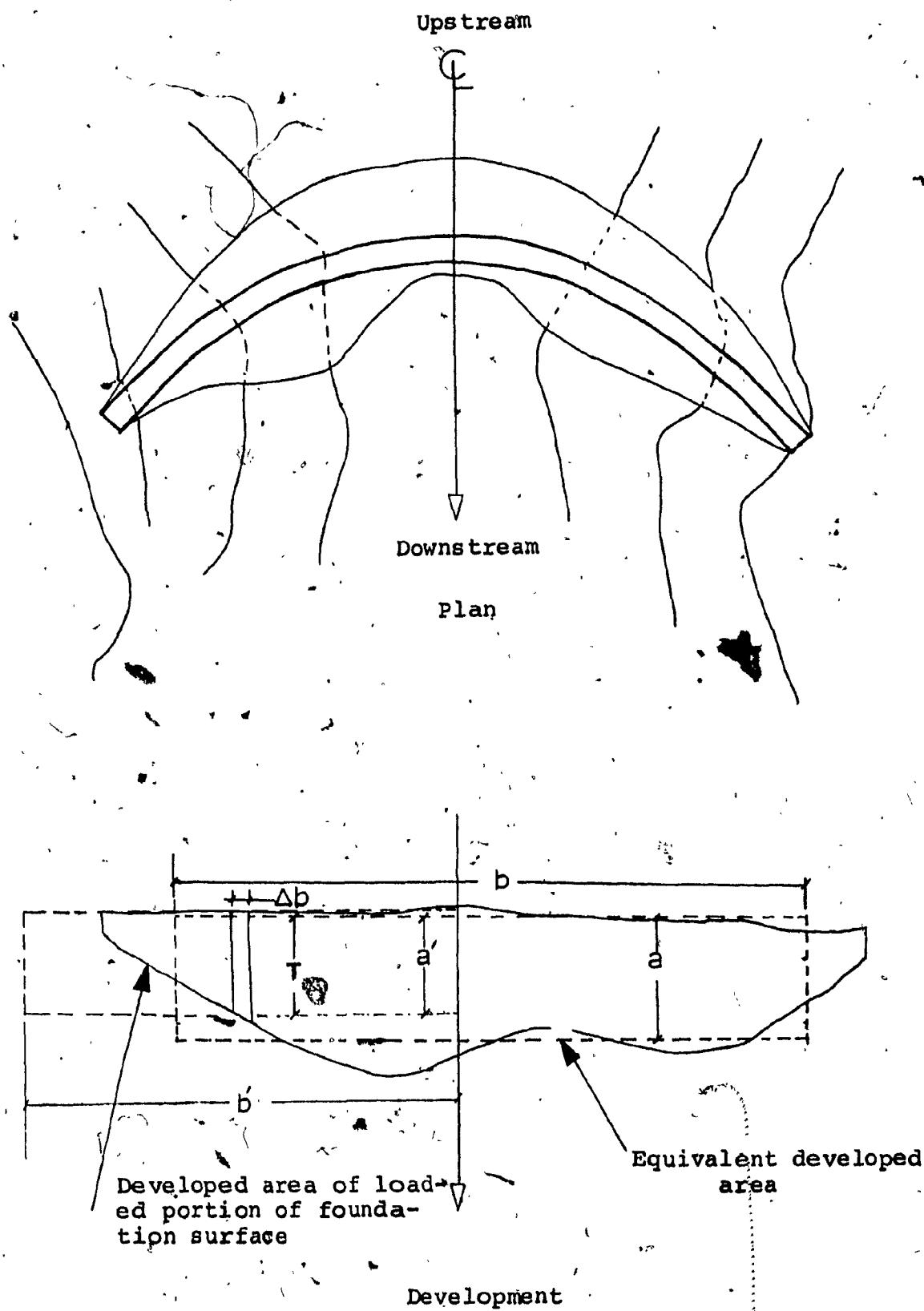


FIG. 3.2 Loaded area of foundation surface

a dam all the curves were made for a unit element whose dimension T (see Fig. 3.2) was less than the dimension a of the total area.

3.4 SPRING CONSTANTS

The above deformation equations contain μ , E_R , p and the ratio b/a , which is determined as follows.

Consider the surface of contact between the dam and the foundation, developed and plotted, as in Fig. 3.2 with the axis as a straight line. This is replaced by a rectangle of the same area and approximately the same proportions, called the 'equivalent developed area'. The ratio of length to width of this rectangle is taken as the b/a ratio for the foundation in question. The value of b/a is, therefore, a constant for any one dam. In computing deformations, the width of a' in b/a is made equal to T , the thickness of the dam at the element considered, thus making b' equal to $(b/a) \times T$.

The reasoning behind this is simple. Movements of the equivalent developed area are assumed equal to movements of the actual foundation. Then by taking the ratio, length/width, of this equivalent area as the b/a value throughout, the average of the deformation of all elements equals the average deformation of the equivalent developed area. This follows directly from the fact that the only

variables in the equations, exclusive of moment effects, are the b/a ratio and the load intensity.

Once the ratio b/a is determined for the dam, the value of k can be taken from the curves, in Diagrams 3.1 to 3.5. It may be noted that the unit area on one side of the dam should be the average values for the equivalent developed area of that side of the dam. If the dam site is approximately symmetrical about the maximum section, dimensions of the equivalent area for either or both sides of the dam are a and $b/2$. For this reason ratios $\frac{b/2}{a}$ and $\frac{a}{b/2}$ are to be substituted for ratio b/a in obtaining some values from the curves.

For	Use
$a', a'', \gamma', \gamma''$	ratio b/a
β', δ'	ratio $\frac{b/2}{a}$
$\gamma' \quad a'$	ratio $\frac{a}{b/2}$

where α' and γ' are in the tangential direction.

In Idikki Dam the following parameters were used.

$$E_R = 3 \times 10^6 \text{ lbs/sq.ft}$$

$$\mu = 0.2$$

$$b/a = 11.5$$

$$L_b = 50'$$

where

E_R = Modulus of elasticity of rock

μ = Poisson's ratio

L_b = Length of block

The spring constant KMZ (moment in ft.lbs.per unit radial rotation) for any block can be calculated as follows.

(1) Sample Calculation - Block No. 1 (See Fig.1.1)

For block 1, $T = 67.0$ ft.

$\psi = 84.0$ degrees

From Equation (3.19)

$$\alpha' = \frac{k}{E_R \times T^2}$$

From Diagram 3.1

$$k = 5.35 \text{ for } b/a = 11.5$$

$$\alpha' = \frac{5.35 \times \sin 84.0^\circ}{3 \times 10^6 \times 144 \times 67^2 \times 50}$$

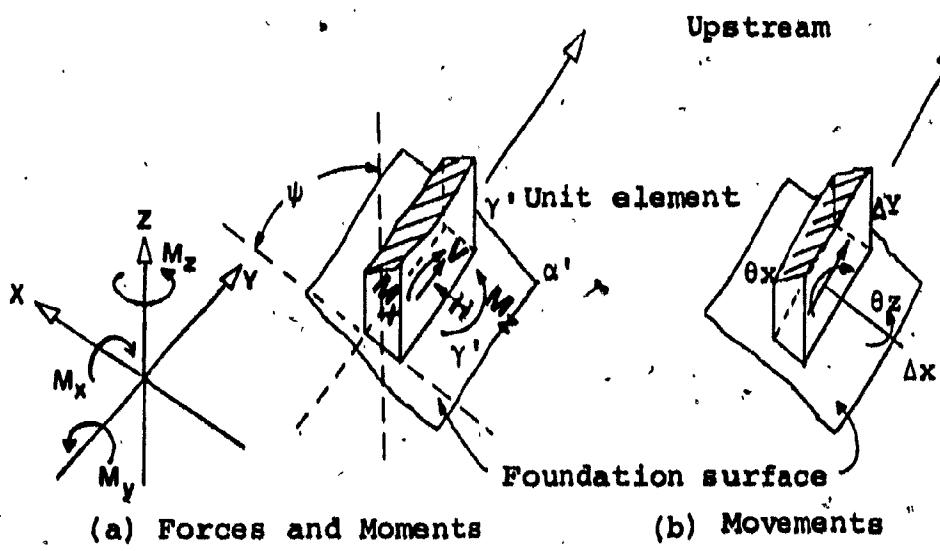


FIG. 3.3 Unit Vertical Element

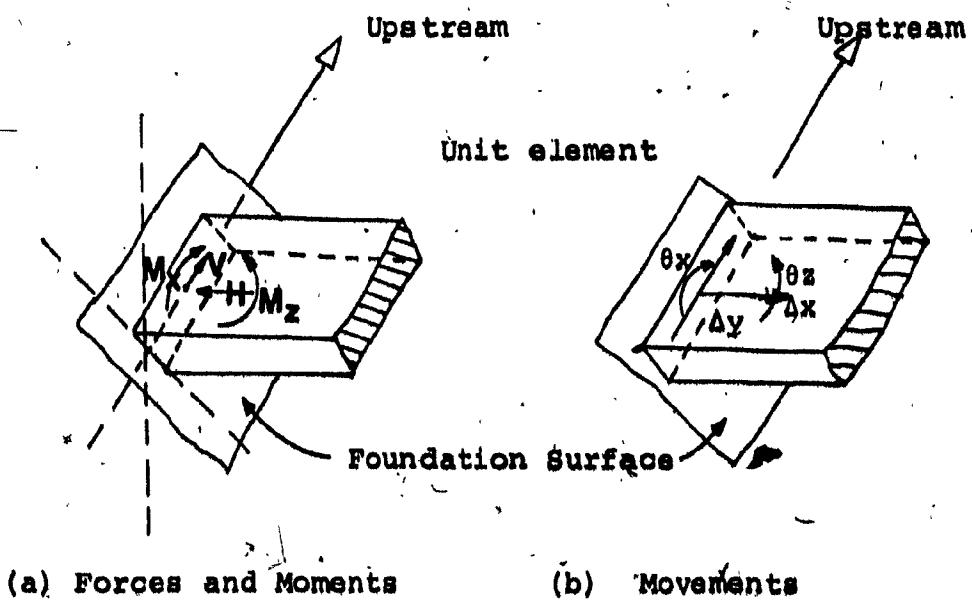


FIG. 3.4 Unit Horizontal Element

$$\frac{1}{a'} = KMZ = 1.82 \times 10^{13} \text{ ft.lbs/unit rotation in radians.}$$

Similarly, the rest of the foundation constants were derived using the corresponding value of T and k , and are shown in Table 3.1.

(2) Tangential Direction

Since the foundation constant was in the tangential direction, $\frac{a}{b/2}$ was used instead of b/a to enter the curve in Diagram 3.1 to evaluate the k value, that is, for

$$\frac{a}{b/2} = 0.17 \quad k \text{ from Diagram 3.5 is } 1.5$$

$$a' = \frac{k \sin\psi}{E_R L^2 T}$$

$$\frac{1}{a'} = KMY$$

$$= \frac{E_R L^2 T}{K \sin\psi}$$

(3) Sample Calculation - Spring Constant KMY

Block 1 (see Fig. 1)

$$KMY = \frac{3 \times 10^6 \times 144 \times 50^2 \times 67}{1.5 \times 0.99452}$$

$$= 4.85 \times 10^{13} \text{ ft.lbs/unit rotation in radians}$$

The constants KMY and KFY are also shown in Table 3.1 for all the blocks in the tangential direction.

TABLE 3.1 FOUNDATION CONSTANTS. CANTILEVER ELEMENTS

Block No.	RADIAL DIRECTION		TANGENTIAL DIRECTION	
	$KMz \times 10^{13}$	$KFx \times 10^{10}$	$KMy \times 10^{13}$	$KFy \times 10^{11}$
1	1.8	0.8	4.85	0.8
2	2.7	1.4	9.61	1.4
3	2.2	0.9	5.10	0.9
4	2.1	0.9	5.10	0.9
5	3.1	1.4	8.62	1.4
6	2.2	0.8	5.10	0.8
7	2.6	1.1	6.49	1.1
8	2.2	0.8	5.18	0.8
9	2.7	1.2	7.25	1.2
10	2.2	0.9	5.81	0.9

Note: All units are in feet and pounds.

(4) Foundation Constants - Abutment

The following foundation constants for unit horizontal element were calculated using the same procedure as that of vertical elements,

TABLE 3.2 FOUNDATION CONSTANT. UNIT HORIZONTAL ELEMENT

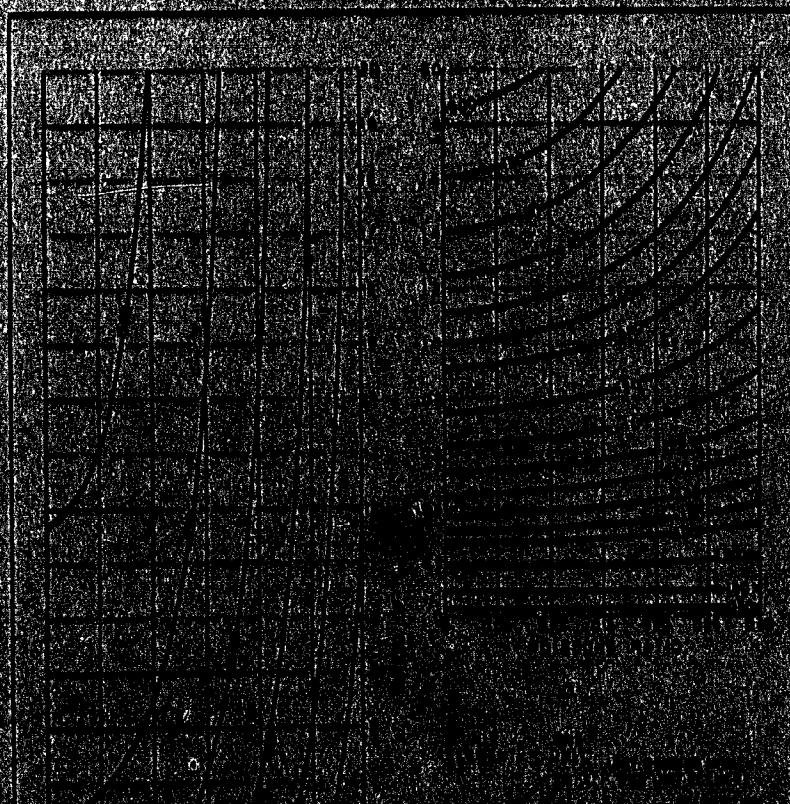
Right Abutment	Left Abutment
$KFX = 6.41 \times 10^7 \times h$	$KFX = 7.63 \times 10^7 \times h$
$KFY = 6.41 \times 10^7 \times h$	$KFY = 7.19 \times 10^7 \times h$
$KMz = 1.44 \times 10^{11} \times h$	$KMz = 1.74 \times 10^{11} \times h$

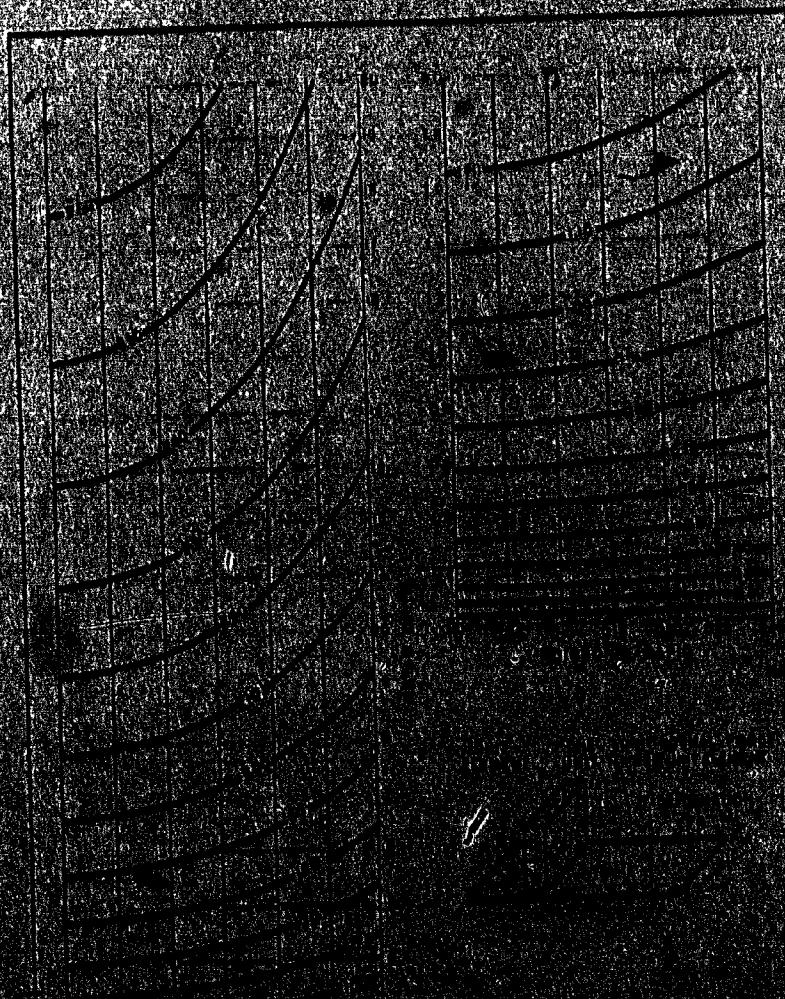
Using the thickness of the arch equal to 30 ft.,
the following constants were obtained.

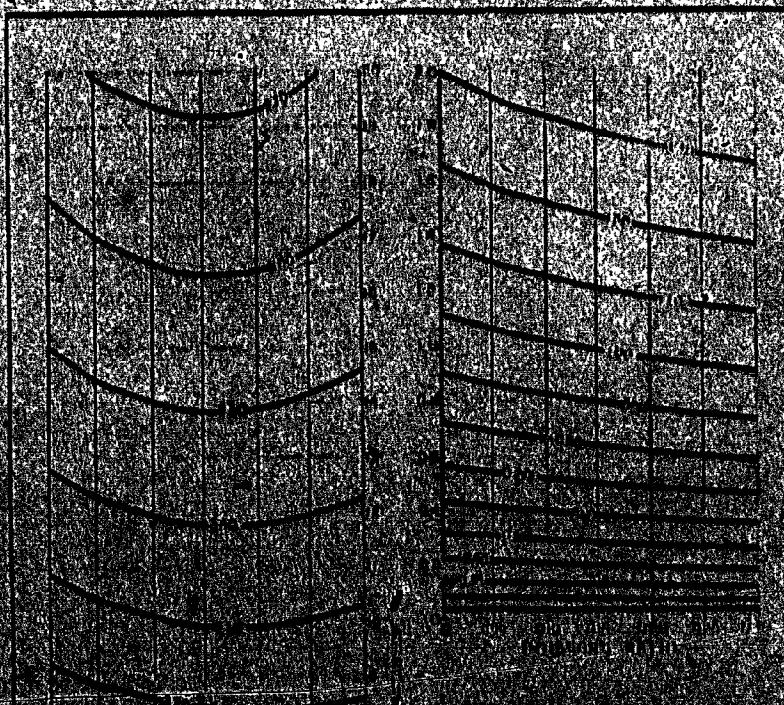
TABLE 3.3 ARCH ABUTMENT CONSTANTS

Right Abutment	Left Abutment
$KFX = 1.92 \times 10^9$	$KFX = 2.28 \times 10^9$
$KFY = 1.92 \times 10^9$	$KFY = 2.15 \times 10^9$
$KMz = 4.32 \times 10^{11}$	$KMz = 5.2 \times 10^{11}$

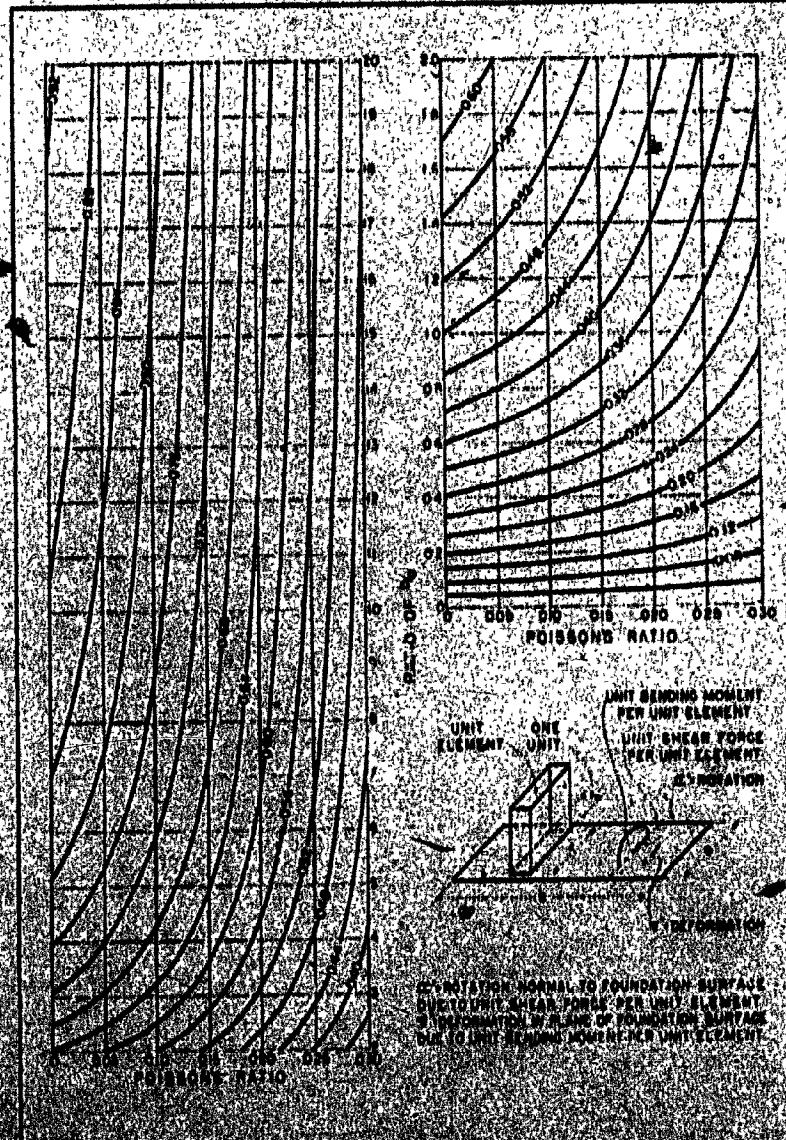
NOTE: Units are in feet, pounds and radians











VARIATION OF VARIOUS PARAMETERS (IN %) WITH
POISSON'S RATIO FOR A BEAM ELEMENT OF UNIT LENGTH

CHAPTER IV
STRESSES IN THE DAM DUE TO EARTHQUAKE

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STRESSES IN THE DAM DUE TO EARTHQUAKE

The stresses in the cantilevers were calculated with the dam completed to various levels during construction, under the action of their own weight and various earthquake intensities. Maximum stresses were induced in cantilevers 1 and 3 (see Fig. 1.1) located in the middle area of the dam having the maximum height. The stresses are shown in Tables 4.1, 4.2 and 6.2. Since the concrete is capable of resisting only small tensile stress, the cantilever was assumed to crack to the point of zero stress, so that the uncracked sections were entirely in compression. This assumption is very conservative, and hence the cracked compressive stresses and the length of crack calculated under this assumption were viewed only as a guide and not as actual. (See Plates 1a, 1b, 1c).

f_1 = Maximum stress

f_2 = Minimum stress (tensile)

f_c = Cracked compressive stress

E = Eccentricity (assuming no tension)

f_2 = $(-f_2$ for tension) (See Fig. 4.1)

$$f_1 = \frac{p}{d} (1+6E) \quad \text{per ft} \quad (4.1)$$

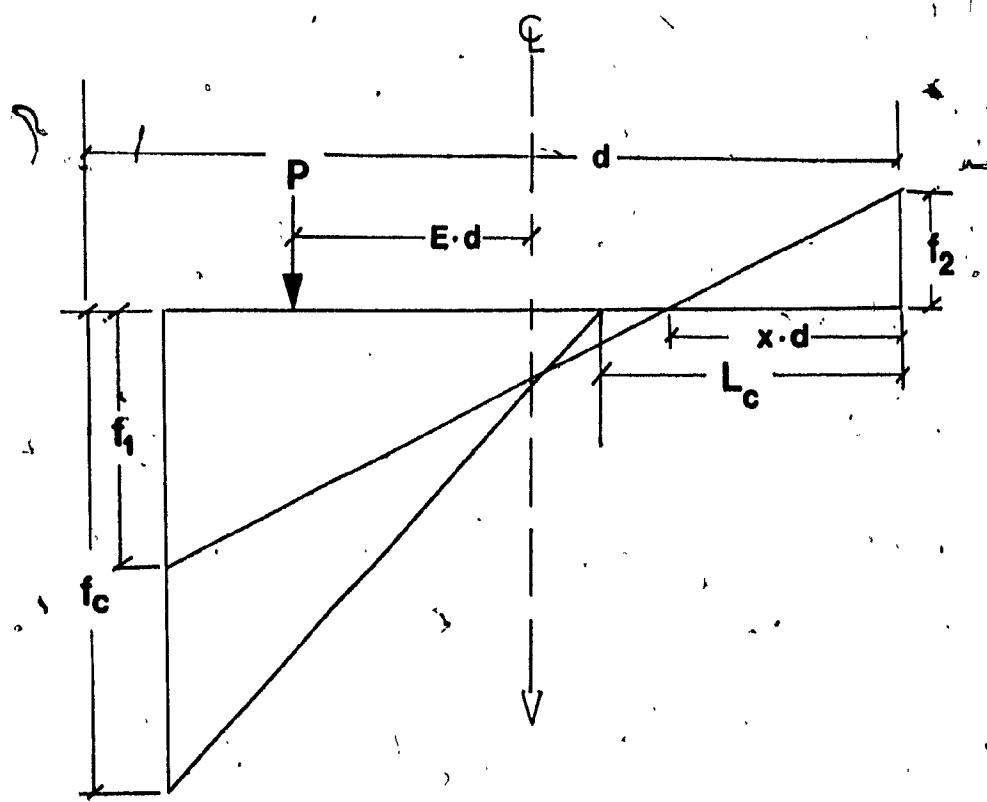
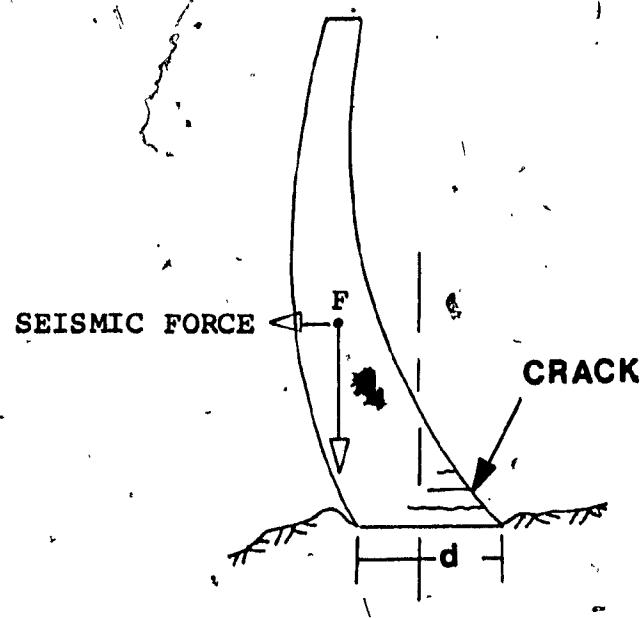


FIG. 4.1 Readjusted stresses

$$f_2 = \frac{P}{d}(1-6E) \text{ per ft} \quad (4.2)$$

$$\frac{d}{2} - E.d = \frac{(d-L_c)}{3}$$

$$\begin{aligned} L_c &= d - \frac{3d}{2} + 3E.d \\ &= d(3E - \frac{1}{2}) \end{aligned} \quad (4.3)$$

$$r = \frac{f_1}{f_2} = \frac{1+6E}{1-6E} \quad (4.4)$$

$$E = \frac{r-1}{6(r+1)}$$

$$\frac{f_2}{x.d} = \frac{f_1}{(d-x.d)}$$

$$\frac{x}{(1-x)} = \frac{f_2}{f_1} = \frac{1}{r}$$

$$x = \frac{1}{r+1}$$

$$3\left(\frac{d}{2} - E.d\right)f_c = f_1(d - x.d)$$

$$f_c = f_1 \cdot \frac{(d-x.d)}{3d(\frac{1}{r}-E)}$$

$$f_c = \frac{f_1(1-x)}{3(\frac{1}{r}-E)} \quad (4.5)$$

The maximum cracked cantilever stresses at any point in the dam are shown in Plate 1c.

Also see plates la and lb for a maximum tensile and compressive stress at any point in the dam due to own weight and under the influence of earthquake (0.05G).

The following example will illustrate the use of the above formulae to calculate the readjusted stresses (cracked compressive stresses) once the dam section is cracked to a certain depth 'L_c'. (See Fig. 4.1)

Example:

$$f_1 = 1114 \text{ psi}$$

$$f_2 = -209 \text{ psi}$$

$$-f_2 = 200 \text{ psi}$$

$$r = \frac{f_1}{-f_2} = 5.34$$

$$x = \frac{1}{5.34 + 1} = \frac{1}{6.34} = .158$$

$$E = \frac{r - 1}{6(r+1)} = \frac{-6.34}{6(-4.34)} = .244$$

$$f_c = 1114 \cdot \frac{1 - 1/6.34}{3(\frac{1}{6.34} - .244)}$$

$$= 1114 \times 1.1 = \underline{\underline{1220}} \text{ psi}$$

$$\text{Length of crack } L_c = (3E - \frac{1}{2})d$$

$$= (.732 - .5)d$$

$$= \underline{\underline{.232d}}$$

TABLE 4.1 STRESSES IN THE BLOCK DUE TO OWN
WEIGHT (LEFT BANK) (See Fig.1.1)

Block No.	Tension (psi)		Compression (psi)	Elevation	
	At the Bottom	Maximum	At the bottom	At the Bottom	For Max.(Tension) El.Dam Completed
1	-288	-374	1110	1880	2295
3	-441	-460	1348	1910	2325
5	-224	-232	909	1995	2355
7	- 74	- 79	589	2085	2085
9	-180	-180	650	2120	2120
11	-212	-212	615	2155	2155
13	-194	-194	542	2190	2190
15	-141	-141	497	2220	2220
17	-141	-141	407	2245	2245
19	-124	-124	375	2265	2265
21	- 93	- 93	306	2295	2295
23	0	147	0	2325	2325

NOTE: Negative (-) denotes tensile stress

TABLE 4.2 STRESSES IN THE BLOCK DUE TO OWN
WEIGHT (RIGHT BANK) (See Fig. 1.1)

Block No.	Stresses (psi)		Elevation at Bottom
	Minimum	Maximum	
2	321	395	1965
4	225	407	2015
6	170	377	2055
8	- 17	489	2100
10	- 60	478	2145
12	- 24	375	2190
14	80	120	2290
16	50	55	2355

NOTE: Negative (-) denotes tensile stress

CHAPTER V
LOCATION OF THE "REINFORCED ARCH"

CHAPTER V

LOCATION OF THE "REINFORCED ARCH"

The readjusted compressive stresses of the cantilever blocks at different stages of completion, and under various earthquake intensities are calculated and plotted in Plate 1c. The 'concrete line' shows the height of the dam during construction and the 'water line' indicates the corresponding water level. It may be noted that the construction of the dam was started in January, 1970, but only in May, 1972, the reservoir was filled with water, and hence the stresses in the dam up to that point, corresponds to the independent cantilever behaviour. Equation (4.5) was used in the calculation of the block stresses, and, as pointed out before, gives only an indication of the stress level, due to the conservative assumptions involved. In Plates 1a and 1b, the maximum compressive and tensile stresses at any point in the cantilever blocks under own weight and under earthquake intensity of 0.05G are shown, respectively. Since these are the maximum at any point, the curve represents only an upper bound stress level and not the actual uniform stress in the dam. The areas shown hatched denote the stress above the allowable design stress in tension and compression. It can be observed from the plates that the stresses increase very rapidly as the height of the dam constructed increases and also that the reservoir loading is very effective in bring-

ing the stresses down close to acceptable level. The effect of the reservoir filling on the stresses in the dam can also be observed from Plate 1C. Once the dam is full, even an earthquake intensity of 0.1 G does not influence the stress level to any great extent.

For the location of the reinforced arch, the maximum cracked compressive stress was limited to 2500 psi in the dam. The elevation of the dam corresponding to this stress level from Plate 1c is el. 2140, approximately. Hence, the reinforced arch was located at this level.

CHAPTER VI

**REINFORCED ARCH ANALYSIS:
ARCH AND CANTILEVER ADJUSTMENT METHOD**

CHAPTER VI

REINFORCED ARCH ANALYSIS:
ARCH AND CANTILEVER ADJUSTMENT METHOD

The "Arch and Cantilever Adjustment Method" essentially consists of equalizing the displacements of the arch and cantilever. This analysis gives the stresses induced due to the earthquake in the steel loop, and it is very similar to the adjustment method used for the arch dam analysis.

The following assumptions were used in the arch and cantilever adjustment method.

6.1 ASSUMPTIONS

- (a) Blocks were assumed to behave like independent cantilevers, elastically supported at the bottom.
- (b) A portion of the dam between the horizontal sections at elevation 2155 and elevation 2125 was taken as the arch which partially restrains the cantilever movements.
- (c) Elevations of the bottom of the cantilevers were fixed on the basis of the blockings.

- (d) Cracked concrete stresses were calculated assuming concrete cracks to the point of zero tensile stress.

6.2 TRIAL AND ERROR METHOD

In this method the displacements of the cantilever under the combined action of its own weight and earthquake were calculated at the reinforced arch level (el.2140.) and plotted. As the first trial, the arch displacements were assumed and plotted on the cantilever displacements. A positive difference between the cantilever and arch displacement would mean a positive cantilever load on the arch. This means in our case a pull on the arch upstream. A negative displacement similarly gives a load on the arch towards downstream. The differences in the displacements (misfits) were converted into positive or negative arch loads using the elastic properties of the cantilever. Using these arch loads the new arch displacements were calculated and plotted. From the difference between the new arch displacements and cantilever displacements, the new misfits, and hence the new arch loads, were calculated in both radial and tangential directions. The trials were repeated until the difference between the two arch displacements became negligible. From the final arch displacement, the final differences between the arch and cantilever displacements were obtained, and hence the arch and cantilever loads were

calculated. The results are shown in Table 6.1. In this trial and error method, even though good results were obtained in the radial direction, convergence in the tangential direction was more difficult to obtain.

6.2.1 Unit Load Method

This method involves the solution of a system of simultaneous equations. At the reinforced arch level, there were 10 cantilever blocks, and hence the arch was divided into 10 segments for the analysis. Displacements in the radial and tangential directions were calculated for unit radial and tangential loads acting on the arch. For the purpose of the analysis, the arch thickness was taken as 30 ft. The cantilever displacements were calculated for the unit radial and tangential loads, unit loads being applied at el. 2140 on the cantilever. A system of 20 equations was obtained with 20 unknowns. The solution gave the 10 radial loads and the 10 tangential loads on the arch, from the cantilever. With the arch loads, the stresses in the arch and the area of reinforcement were calculated. STRUDL-2° computer program was used for the calculation of the displacements and another computer program for the solution of the simultaneous equations.

6.2.2 Adjustment Equations

In the arch dam, the cantilever and arch deflection of any point which is common to an arch and cantilever will be the same. So, where the independent cantilever deflection is more than the arch deflection due to earthquake or own weight, the cantilever will be restrained by the arch, and where the arch deflection is more than the cantilever deflection, the arch will be restrained by the cantilever.

The principle of the arch and cantilever adjustment can be explained as follows. The final arch deflection at any point is equal to the total cantilever deflection minus the deflection of the cantilever block due to the cantilever load from the arch. That is (See Fig.6.1)

$$\sum_{j=1}^{10} p_j d_i^j = \Delta_i^c - p_i d_i^c \quad (6.1)$$

where

p_j = Arch load at the j^{th} segment of the arch

d_i^j = Arch deflection at 'i' due to unit load
at 'j'

p_i = Load applied on the cantilever block 'i'
by the arch

d_i^c = Cantilever block deflection at 'i' due to
unit load

Δ_i^c = Total cantilever block deflection due to
earthquake and deadload action at 'i'

Using the above logic, a system of 20 equations can be obtained with 20 unknowns. Let P_1 to P_{10} be the 10 radial arch loads and P_{11} to P_{20} be the 10 tangential arch loads. Expanding the basic formula (6.1) the following equations were obtained:

$$P_1 D_{rr_1}^{-1} + P_2 D_{rr_1}^{-2} + \dots P_j D_{rr_1}^{-j} + \dots P_{10} D_{rr_1}^{-10} +$$

$$P_{11} D_{rt_1}^{-1} + \dots P_{20} D_{rt_1}^{-10} = \Delta_{r_1}^c - P_1 D_{r_1}^c$$

That is

$$P_1(D_{rr_1}^{-1} + D_{r_1}^c) + P_2 D_{rr_1}^{-2} + \dots P_j D_{rr_1}^{-j} + \dots$$

$$P_{10} D_{rr_1}^{-10} + P_{11} D_{rt_1}^{-1} + \dots P_{20} D_{rt_1}^{-10} = \Delta_{r_1}^c \quad (6.2)$$

$$P_1 D_{rr_2}^{-1} + P_2 (D_{rr_2}^{-2} D_{r_2}^c) + \dots P_j D_{rr_2}^{-j} + \dots$$

$$P_{10} D_{rr_2}^{-10} + P_{11} D_{rt_2}^{-1} + \dots P_{20} D_{rt_2}^{-10} = \Delta_{r_2}^c \quad (6.3)$$

$$P_1 D_{rr_{10}}^{-1} + P_2 D_{rr_{10}}^{-2} + \dots P_j D_{rr_{10}}^{-j} + P_{10} (D_{rr_{10}}^{-10} + D_{r_{10}}^c) +$$

$$P_{11} D_{rt_{10}}^{-1} + \dots P_{20} D_{rt_{10}}^{-10} = \Delta_{r_{10}}^c \quad (6.4)$$

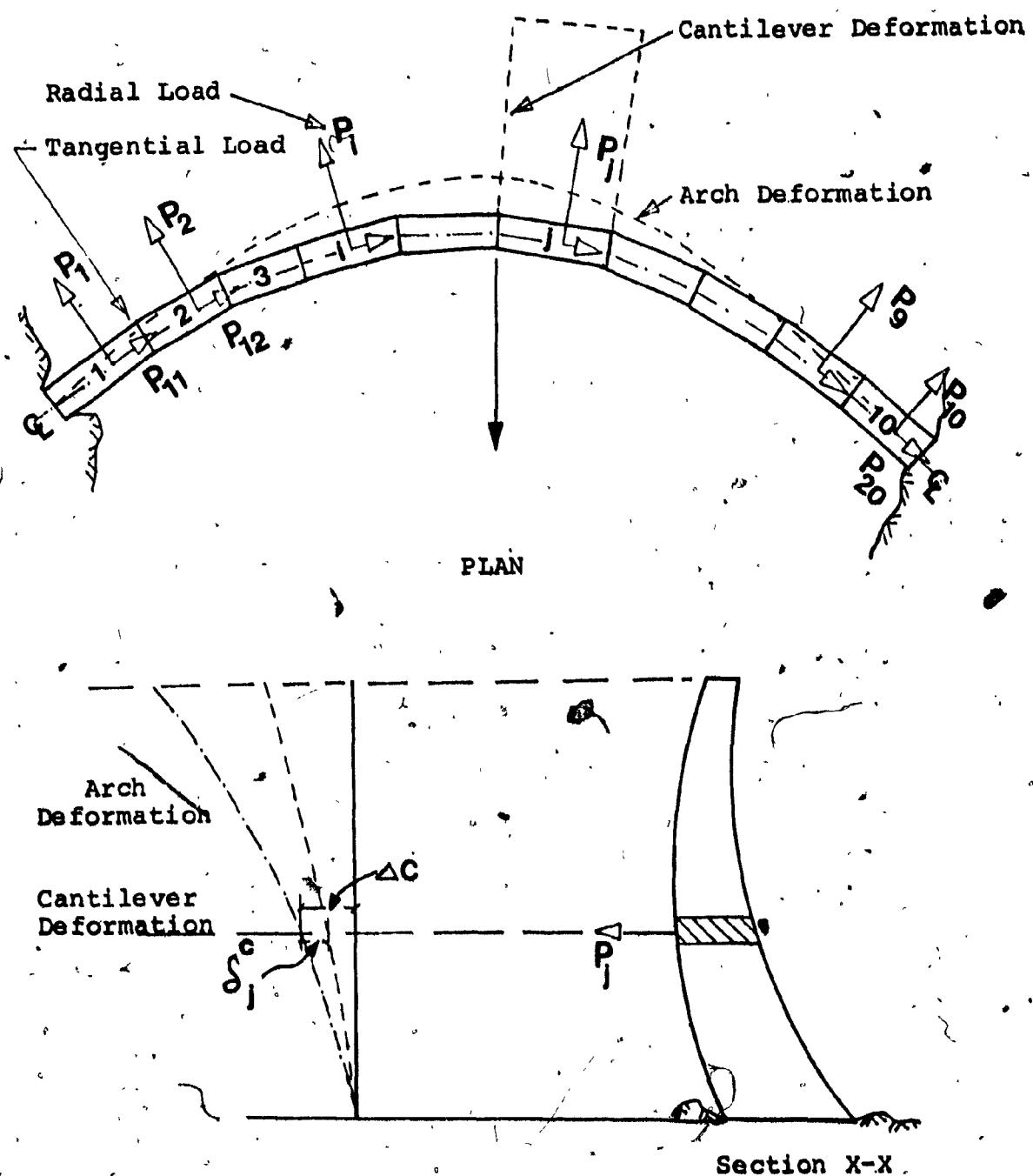


FIG. 6.1 Arch and Cantilever Deformation

$$P_1 D_{tr_i}^1 + P_2 D_{tr_i}^2 + \dots P_j D_{tr_i}^j + \dots P_{10} D_{tr_i}^{10} + \dots$$

$$P_{11} D_{tt_i}^1 + \dots P_{20} D_{tt_i}^{10} = \Delta_t^c - P_{11} D_{t_i}^c$$

or

$$P_1 D_{tr_i}^1 + P_2 D_{tr_i}^2 + \dots P_j D_{tr_i}^j + \dots P_{10} D_{tr_i}^{10} + \dots$$

$$P_{11}(D_{tt_i}^1 + D_{t_i}^c) + \dots P_{20} D_{tt_i}^{10} = \Delta_{t_i}^c \quad (6.5)$$

$$P_1 D_{tr_2}^1 + P_2 D_{tr_2}^2 + \dots P_j D_{tr_2}^j + \dots P_{10} D_{tr_2}^{10} + \dots$$

$$P_{11} D_{tt_2}^1 + P_{12}(D_{tt_2}^2 + D_{t_2}^c) + \dots P_{20} D_{tt_2}^{10} = \Delta_{t_2}^c \quad (6.6)$$

$$P_1 D_{tr_{10}}^1 + P_2 D_{tr_{10}}^2 + \dots P_j D_{tr_{10}}^j + P_{10} D_{tr_{10}}^{10} + \dots$$

$$P_{11} D_{tt_{10}}^1 + \dots P_{20}(D_{tt_{10}}^{10} + D_{t_{10}}^c) = \Delta_{t_{10}}^c \quad (6.7)$$

where

$P_1 = P_{10}$ = Radial arch loads (See Fig. 6.1)

$P_{11} = P_{20}$ = Tangential arch loads

$D_{rr_i}^j$ = Radial arch deflection at 'i' due to
Radial arch unit load at 'j'

$D_{rt_i}^j$ = Radial arch deflection at 'i' due to
tangential arch unit load at 'j'

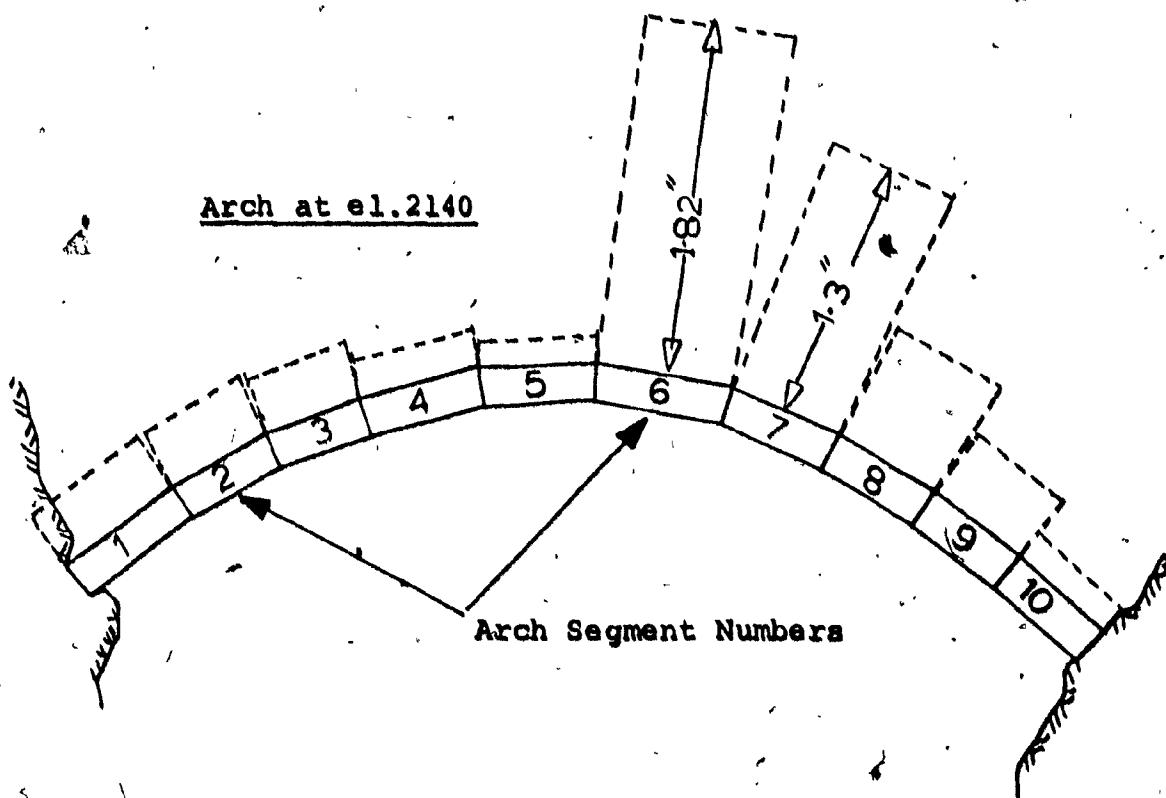


FIG. 6.2. Cantilever Element. Radial Movements

D_{tri}^j = Tangential arch deflection at i
due to radial arch unit load at j

D_{tti}^j = Tangential arch deflection at i due
to tangential arch unit load at j

D_{ri}^c = Radial cantilever deflection at i due
to radial cantilever unit load at i

D_{ti}^c = Tangential cantilever deflection due
to tangential cantilever unit load

A_{ri}^c = Total cantilever block deflection due
to earthquake and dead load action

A_{ti}^c = Total cantilever block deflection in
the tangential direction due to earth-
quake and D.L. action

The solution gives 10 radial arch loads and 10 tangential arch loads. Using these arch loads and the boundary conditions of the arch abutment, a computer analysis was made to obtain the internal stresses in the arch at various segments. The computer input and output are shown in Appendix A.

A comparison of the arch loads obtained from the trial and error method and unit load method are shown in

Table 6.1.

From Table 6.1, it may be noted that the radial loads in both methods are very close, but the tangential loads in the trial and error method need further trials for convergence.

TABLE 6.1 LOAD IN THE ARCH ELEMENT

Segment No.	Arch Load in kips			
	Trial and Error Solution		Unit Load Solution	
	<u>Radial</u>	<u>Tangential</u>	<u>Radial</u>	<u>Tangential</u>
10	-9050.	-11400.	-8536.	-20016.
8	15200.	-8700.	15216.	2537.
6	3200.	-3000.	3005.	-4340.
4	-1060.	-3100.	-1190.	-3350.
2	4500.	-1400.	-4450.	-1510.
1	8400.	-19.	8420.	-40.
3	8700.	110.	8740.	93.
5	9000.	1200.	8980.	1146.
7	1800.	4400.	1620.	4362.
9	-1900.	20800.	1920.	20235.

TABLE 6.2 STRESSES IN THE DAM UNDER EARTHQUAKE
INTENSITY .05 G WITH REINFORCED ARCH
(See Fig. 1.1)

Block No.	Maximum Tension	Maximum Compression	Elevation At the Bottom
1	-301	1221	1882
2	-58	629	2015
3	-358	1217	1910
4	155	444	2015
5	-138	807	1995
6	-13	560	2055
7	-160	711	2080
8	-88	538	2085
9	-241	668	2120
10	-106	520	2145
11	-248	648	2155
12	-92	615	2190
13	-217	564	2190
14	60	134	2295
15	-160	455	2225
16	46	60	2355
17	-149	414	2245
18	-121	356	2265
21	-111	325	2280
23	-140	303	2310

CHAPTER VII
COMPUTER ANALYSIS AND RESULTS

CHAPTER VII COMPUTER ANALYSIS AND RESULTS

STRUDL-II application program was used for the arch analysis. The matrix solution gave the arch loads which were used as the input to the arch analysis program.

The STRUDL-II program data and answers are given in Appendix A. The axial tension, moments and displacements from these results, are shown in Drawing No.1. From loading-3 RADIAL AND TANGENTIAL COMBINED, member 5, joint 6 has the maximum axial tensile force of 28210 kips. Towards the abutments, a part of the axial forces was taken by the cantilevers by tangential deformations according to their stiffness. Hence at the ends of the arch, the forces were reduced to 1122 kips and 2361 kips at joints 1 and 11, respectively. The twisting moments were relatively very small at the centre of the arch but quite considerable towards the abutments. This is largely due to the fact that the elasticity of the joints was not considered, and in fact a large portion of these moments in many joints will be transferred to the cantilever blocks as twist moments.

The maximum shear stress was 70 psi.

CHAPTER VIII
DESIGN OF REINFORCEMENT

CHAPTER VIII
DESIGN OF REINFORCEMENT

Using the computer output, reinforcement was designed to take the tensile force in the arch.

The reinforcement was placed off-centre of the arch centre line so as to avoid, to a great extent, the formation of twisting moment in the section. Drawing No. 2 shows the reinforcement details.

8.1 DETAILED DESIGN

Using deformed bars (60 ksi yield stress):

Joint 6:

Maximum Tension	= 28,210 kips
Allowable stress 80% of yield stress	= $6 \times .8 = 48$ ksi
Area of steel required	= 588 sq.in. say 590 sq.in.
Group of 6 (36 mm) bars area	= 9.45 sq.in.
Number of groups required	= 62.2 , say 63
Using 7 in one row	
Number of rows	= $63/7 = 9$

Joints 5, 7, 8, 9:

Maximum Tension	=	27,400 kips	Total <u>63</u>
Area of steel	=	571 kips	
Number of bundles	=	60.2, say 61	
Strength of bundle	=	9.48×48	<u>450.5</u> kips

Joint 4:

Maximum Tension	=	24,300	
Number of bundles	=	53.4, say <u>54</u>	

Joint 10:

Maximum Tension	=	23,100	
Number of bundles	=	50.7, say <u>51</u>	

Joint 3:

Maximum Tension	=	20,760	
Number of bundles	=	45.6, say <u>46</u>	

Joint 2:

Maximum Tension	=	12,700	
Number of bundles	=	27.8, say <u>28</u>	

Bond Stress:

Average bond stress	=	12 kg/cm ²
	=	172 lb/in ²
(40% more) for deformed bars	=	$172 \times 1.4 = 241$
	=	$\frac{48,000}{4 \times 241} = 49.7 \text{ Dia.}$
		50 Diameter
Diameter of a bar	=	1.41"
Length of bar	=	50×1.41
	=	70.5
For bundled bars increase 50%	=	70.5" <u>35.25"</u> 105.75"
Length of bars	=	$\frac{105.75"}{12}$, say <u>9'</u> to either side

Laps:

Length of lap	=	bar diameter $\times \frac{\text{actual tensile stress}}{4 \times \text{permissible average bond stress}}$
Length of lap	=	$\frac{1.41 \times 9.6 \times 1,000}{4 \times 30 \text{ Dia.}} = 16"$

50% increase for
bundled bars

- $30 \times 1.41 \times 1.5$
- 63.5
- 5'-3 $\frac{1}{2}$ ", say 5'

CHAPTER IX
CONCLUSION

CHAPTER IX

CONCLUSION

From the different possible solutions, reservoir loading was selected to counteract the particular earthquake action inducing forces in the direction downstream to upstream in Idikki Dam. From Plate 1C, it can be seen that earthquake intensity up to 0.1 G has relatively small effect on the stresses in the dam, once the reservoir is one third full or more. Economic and other considerations outweighed the probable risk of a partially completed dam under severe vibrations tending to fall into an empty reservoir, during construction. The possibility of an earthquake during a period in which the reservoir might remain empty, was also considered. But, it was noted that once the reservoir was impounded, complete emptying was not feasible due to the topography of the reservoir.

Nevertheless, the concept of the 'Reinforced Arch' analysis and design proved an effective solution to resist the seismic effect on double curvature arch dams. Such a solution was employed in the design of the 'Rappel Dam' located in Chile, under similar earthquake considerations.

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APPENDIX A
COMPUTER ANALYSIS

A-I Block Deformation Calculation

INTEGRATED CIVIL ENGINEERING SYSTEM

- ICES -

1981-1982 1979 1978-79, 40,469

PERSONNEL-ICES INGRESSES EXECUTIVE SYSTEM

IMPLEMENTED ON FEB 1979 AT

INFORMATION AUTOMATION COMPANY

STATEMENT OF MISCELLANEOUS

DATA FILE 100

57

RE

S.N.C. COMPUTATION

STREET 1995, SCANTLERS BLOCK NO 5

ICES STAND-11

THE STRUCTURAL INSTITUTE
CIVIL ENGINEERING MATERIALS LABORATORY
MASSACHUSETTS INSTITUTE OF TECHNOLOGY
CAMBRIDGE, MASSACHUSETTS
MEMBER, 1964
17 15 48 2/16/70

5 change to 2500-01-202-02-09
5 notches or key profile & p. clear
5 checked by S. L. G.

5 min 3 min 75

5 rate no 12901 of "SAFETY" CROWN, 1/2000, 6-200, AIA

units FEET KIPS INCHES

TYPE	PLATE	PLATE	PLATE
NOTCH	CRIMP	CRIMP	CRIMP
1	300	495.	FREE
2	2220	375.	
3	34.97	515.	
4	942	255.	
5	769	395.	
6	10120	255.	
7	57.61	75.	
8	25.719	35.0	500
9	25.719	35.0	500
10	2	2	2
11	3	2	
12	4	3	
13	5	4	

58

CONSTANTS

1 AX 22246.3 32 .192200
2 AX 22246.3 32 .022200

3 AX 22246.3 32 .000000
4 AX 22246.3 32 .000000
5 AX 22246.3 32 .000000

6 AX 32247.2 32 .213333
7 AX 32247.2 32 .199997

8 AX 32247.2 32 .000000
9 AX 32247.2 32 .000000

FORces & REACTIONS

CONSTANTS

1 2 3000. 400.

(G 1250. 400.

WEIGHTS POUNDS

1 FORCE X CLEAR DISTANCE -2150.
2 FORCE X CLEAR DISTANCE -2000.
3 FORCE X CLEAR DISTANCE -2000.
4 FORCE X CLEAR DISTANCE -2000.

5 FORCE X CLEAR DISTANCE -2150.
6 FORCE X CLEAR DISTANCE -2150.
7 FORCE X CLEAR DISTANCE -2150.
8 FORCE X CLEAR DISTANCE -2150.

LAWD'S RESTRAINTS 1000 .950.

MEMBER LOADS

1 FORCE X CLEAR DISTANCE -2150.
2 FORCE X CLEAR DISTANCE -2150.
3 FORCE X CLEAR DISTANCE -2150.

5 space x global uniform -19.9
5 space x global uniform -22.0
6 space x global uniform -23.5
7 space x global uniform -24.5
LAYERED LAYER AT EL. 2140.0

5 space x global curv. 1 25.
LEAVING CONVENTIONAL, SURFACE, NO. 1, 2 2000 5 25 DEGREE

TYPESET LIST PAGE

STRUCTURE ANALYSIS
LAYERED LAYER

LAYERED LAYER
LIST REACTIONS ALL

S.N.C. COMPUTATION



61

SECTION OF CANTILEVER

PROBLEM - 101211 TITLE - CANTILEVER BLOCK NO 9

ACTIVE UNITS PEER KIP AND DEEP SEC

ACTIVE STRUCTURE TYPE PLANE FRAME

ACTIVE COORDINATE AXES X Y

LOADING - 1

SEED LOAD OF THE BLOCK

REACTANT JOINT LOADS - SUPPORTS

JOINT	FORCE			MOMENT		
	X FORCE	Y FORCE	Z FORCE	X MOMENT	Y MOMENT	Z MOMENT
1	0.0000	89449.56900	0.0000	-1000000.0000	-1000000.0000	-1000000.0000

LOADING - 2

EARTHQUAKE LOAD .05G

REACTANT JOINT LOADS - SUPPORTS

JOINT	FORCE			MOMENT		
	X FORCE	Y FORCE	Z FORCE	X MOMENT	Y MOMENT	Z MOMENT
2	7919.68647	-9.09923	0.0000	-1421168.0000	-1421168.0000	-1421168.0000

LOADING - 3

CONCENTRATED LOAD AT BL- 2146.

REACTANT JOINT LOADS - SUPPORTS

JOINT	FORCE			MOMENT		
	X FORCE	Y FORCE	Z FORCE	X MOMENT	Y MOMENT	Z MOMENT
3	-10000.00000	-0.00001	0.00000	-10000.00000	-0.00001	0.00000

LOADING - 4

CRANE 0.00000

62

S.N.C. COMPUTATION

EN

62

STRAIGHT - seven under initial

start

X FORCE

X FORCE

HOLDING

REPORT

15617-4219 09569.5000

-2299039.5000

REPORT

X FORCE

HOLDING

REPORT

UNITS INCHES	LIST DISPLACEMENTS ALL							

63

S.N.C. COMPUTATION

RESULTS OF LATEST ANALYSES

PROBLEM 101KKI TITLE - CANTILEVER BLOCK NO 5

ACTIVE UNITS INCH KIP RAD DEG SEC

ACTIVE STRUCTURE TYPE PLANE FRAME

ACTIVE COORDINATE AXES X Y

LOADING - 1 DOME LOAD OF THE BLOCK

RESULTANT JOINT DISPLACEMENTS - SUPPORTS

JOINT	DISPLACEMENT		
	X DISP.	Y DISP.	Z DISP.
1	0.0000	0.0	0.0
2			

RESULTANT JOINT DISPLACEMENTS - FREE JOINTS

JOINT	DISPLACEMENT		
	X DISP.	Y DISP.	Z DISP.
1	0.3013	-0.3068	
2	0.1040	-0.2715	
3	-0.0581	-0.2322	
4	-0.1524	-0.1964	
5	-0.1682	-0.1547	
6	-0.1202	-0.1053	
7	-0.0478	-0.0506	

LOADING - 2 EARTHQUAKE LOAD .05G

RESULTANT JOINT DISPLACEMENTS - SUPPORTS

JOINT	DISPLACEMENT		
	X DISP.	Y DISP.	Z DISP.
1	0.0068	0.0	0.0
2			

64
0.0000



RESULTANT JOINT DISPLACEMENTS - FREE JOINTS

JOINT	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
1	-1.7844	0.0863				0.0001
2	-1.4250	0.0396				0.0005
3	-1.0781	-0.0033				0.0005
4	-0.7594	-0.0515				0.0004
5	-0.4726	-0.0403				0.0004
6	-0.2559	-0.0303				0.0003
7	-0.0889	-0.0107				0.0002

LOADING - 3 CONCENTRATED LOAD AT EL. 2140

RESULTANT JOINT DISPLACEMENTS - SUPPORTS

JOINT	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
8	0.0008	0.0				-0.0001

RESULTANT JOINT DISPLACEMENTS - FREE JOINTS

JOINT	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
1	1.1168	-0.0298				-0.0002
2	0.9413	-0.0070				0.0002
3	0.7658	0.0148				0.0002
4	0.5903	0.0303				-0.0002
5	0.4148	0.0399				-0.0002
6	0.2401	0.0283				-0.0002
7	0.0907	0.0109				-0.0002

LOADING - 4 GRAVE Q. 105

RESULTANT JOINT DISPLACEMENTS - SUPPORTS

JOINT	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
8	-0.0061	0.0				0.0001

RESULTANT JOINT DISPLACEMENTS - FREE JOINTS

JOINT	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
						65



	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
1	-1.3136	-0.2267				0.0002
2	-1.1896	-0.2397				0.0002
3	-1.0338	=0.2351				0.0002
4	-0.8387	-0.2229				0.0003
5	-0.6013	-0.1911				0.0004
6	-0.3479	-0.1328				0.0003
7	-0.1283	-0.0603				0.0003

66



S.N.C. COMPUTATION LTD

S TO INCLUDE BLOCK NO 6.

CHANGES

CHANGE ID VIDIRRI "CANTILEVER BLOCK NO 6"

J01 8 REL KFX 6.66E5 KMZ 2.62E11

UNITS FEET

J01 8 C00 X 15.32 Y 75.0

ADDITIONS

LOADING LIST ALL

STIFFNESS ANALYSIS

LOADING LIST ALL

UNITS INCHES

LIST DISPLACEMENTS ALL

67

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S.N.G. COMPUTATION

RESULTS OF LATEST ANALYSES

PROBLEM - IDIKI! TITLE - CANTILEVER BLOCK NO 6

ACTIVE UNITS INCH KIP RAD DEGF SEC

ACTIVE STRUCTURE TYPE PLANE FRAME

ACTIVE COORDINATE AXES, X Y

LOADING - 1 DEAD LOAD OF THE BLOCK

RESULTANT JOINT DISPLACEMENTS - SUPPORTS

JOINT	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
1	-0.0000	0.0	0.0	0.0000	0.0	0.0

RESULTANT JOINT DISPLACEMENTS - FREE JOINTS

JOINT	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
1	0.7638	-0.2683	0.0	0.0001	0.0001	0.0001
2	0.4974	-0.2261	0.0	0.0001	0.0001	0.0001
3	0.2662	-0.1782	0.0	0.0001	0.0001	0.0001
4	0.1028	-0.1343	0.0	0.0001	0.0001	0.0001
5	0.0179	-0.0923	0.0	0.0001	0.0001	0.0001
6	-0.0033	-0.0460	0.0	0.0001	0.0001	0.0001
7	-0.0000	-0.0002	0.0	0.0001	0.0001	0.0001

LOADING - 2 EARTHQUAKE LOAD .05G

RESULTANT JOINT DISPLACEMENTS - SUPPORTS

JOINT	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
1	-0.0097	0.0	0.0	0.0000	0.0000	0.0000

68

RESULTANT JOINT DISPLACEMENTS - FREE JOINTS

JOINT	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
1	-1.1799	0.0802				0.0004
2	-0.9080	0.0449				0.0004
3	-0.6487	0.0129				0.0003
4	-0.4166	-0.0076				0.0003
5	-0.2264	-0.0136				0.0002
6	-0.0843	-0.0075				0.0002
7	-0.0099	0.0013				0.0000

LOADING - 3 CONCENTRATED LOAD AT EL. 2140.

RESULTANT JOINT DISPLACEMENTS - SUPPORTS

JOINT	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
1	0.0190	0.0				-0.0000
2						
3						
4						
5						
6						
7						

LOADING - 4 GRAVE O. NOS

RESULTANT JOINT DISPLACEMENTS - SUPPORTS

JOINT	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
1	-0.0083	0.0				0.0000
2						

RESULTANT JOINT DISPLACEMENTS - FREE JOINTS

JOINT	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
1	0.5025	-0.0228				-0.0001
2	0.4168	-0.0117				-0.0001
3	0.3311	-0.0011				-0.0001
4	0.2494	0.0005				-0.0001
5	0.1967	0.0002				-0.0001
6	0.0748	0.0004				-0.0001
7	0.0192	-0.0011				-0.0000

69

	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
1	-0.3039			-0.1957		
2	-0.3245			-0.1854		
3	-0.3209			-0.1863		
4	-0.2742			-0.1411		
5	-0.1852			-0.1046		
6	-0.0796			-0.0328		
7	-0.0089			0.0013		

70

20

3	BLOCK NO 7			
CHANGES				
UNITS FEET				
CHANGE TO 'IDOKI' CANTILEVER BLOCK NO 7 & 8'				
JOI 1 COORD X 47.07 Y 371.25 Z EL 2415				
JOI 2 COORD X 42.24 Y 311.25				
JOI 3 COORD X 36.99 Y 251.25				
JOI 4 COORD X 34.83 Y 191.25				
JOI 5 COORD X 37.01 Y 131.25				
JOI 6 COORD X 39.98 Y 101.25				
JOI 7 COORD X 44.21 Y 71.25 Z EL 2115				
JOI 8 COORD X 30.63 Y 36.25 Z EL 2080				
UNITS INCHES				
JOI 9 REL REV 9.163 KMZ 3.12E11 Z UNITS IN INCHES				
DELETIONS				
MEMBER 1 2 3 4 5 6 7 PROPERTIES				
ADDITIONS				
UNITS FEET				
MEMBER PROP . PRIS				
1 AX 1444. 12 •1036E6				
2 AX 1889. 12 •2294E6				
3 AX 2310. 12 •4210E6				
4 AX 2648. 12 •6336E6				
5 AX 2944. 12 •9369E6				
6 AX 2944. 12 •9369E6				
7 AX 3035. 12 •1085E7				
LOADING 40 DEAD LOAD OF THE BLOCK				
MEMBER LOAD				

71

1	FORCE	Y	GLOBAL	UNIFORM	-215.
2	FORCE	Y	GLOBAL	UNIFORM	-215.
3	FORCE	Y	GLOBAL	UNIFORM	-347.
4	FORCE	Y	GLOBAL	UNIFORM	-400.
5	FORCE	Y	GLOBAL	UNIFORM	-442.

LOADING 3 EARTHQUAKE LOAD .05G

MEMBER LOAD

1	FORCE	X	GLOBAL	UNIFORM	-10.2
2	FORCE	X	GLOBAL	UNIFORM	-14.0
3	FORCE	X	GLOBAL	UNIFORM	-17.2
4	FORCE	X	GLOBAL	UNIFORM	-19.9
5	FORCE	X	GLOBAL	UNIFORM	-22.2
6	FORCE	X	GLOBAL	UNIFORM	-23.5
7	FORCE	X	GLOBAL	UNIFORM	-24.5

LOADING 3 CON LOAD AT EC 2140

MEMBER LOADS

6 FORCE X GLOBAL CON P 20000. L 25.

LOADING COMBINATION 7 TERRACE NO 71 CONCRETE G0 1.0.1 1.0.3 1.0.5 1.0.7 DEGREE

LOADING LIST 49 5 6 7

STIFFNESS ANALYSIS

LOADING LIST 6 7

LIST REACTIONS ALL

RESULTS OF LATEST ANALYSES

PROBLEM - 10011 TITLE - CANTILEVER, BLOCK NO 7 & 8

ACTIVE UNITS FEET KIP RAD DEGF SEC

ACTIVE STRUCTURE TYPE PLANE FRAME

ACTIVE COORDINATE AXES X Y

LOADING - 6 CON LOAD AT EL 2140

RESULTANT JOINT LOADS - SUPPORTS

JOINT	FORCE		MOMENT	
	X FORCE	Y FORCE	X MOMENT	Y MOMENT
	-10000.0039	-0.0001		597551.4975

LOADING - 7 GRAVES NO 7

RESULTANT JOINT LOADS - SUPPORTS

JOINT	FORCE		MOMENT	
	X FORCE	Y FORCE	X MOMENT	Y MOMENT
	50034609	884035625		-1796034.0000

S.N.C. COMPUTATION

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74

UNITS INCHES
LIST DISPLACEMENTS ALL

RESULTS OF LATEST ANALYSES

PROBLEM - 100TK1 TITLE - CANTILEVER BLOCK NO 7 & 8

ACTIVE UNITS INCH KIP RAD DEGF SEC

ACTIVE STRUCTURE TYPE PLANE FRAME

ACTIVE COORDINATE AXES X Y

LOADING - 6 CON LOAD AT EL 2140

RESULTANT JOINT DISPLACEMENTS - SUPPORTS

JOINT	DISPLACEMENT		
	X DISP.	Y DISP.	Z DISP.
	0.0109	0.0	-0.0000

RESULTANT JOINT DISPLACEMENTS - FREE JOINTS

JOINT	DISPLACEMENT		
	X DISP.	Y DISP.	Z DISP.
1	0.2534	0.0000	-0.0001
2	0.2092	0.0037	-0.0001
3	0.1661	0.0076	-0.0001
4	0.1179	0.0092	-0.0001
5	0.0728	0.0076	-0.0001
6	0.0502	0.0054	-0.0001
7	0.0283	0.0026	-0.0001

LOADING - 7 GRADEQ NO 7

RESULTANT JOINT DISPLACEMENTS - SUPPORTS

JOINT	DISPLACEMENT		
	X DISP.	Y DISP.	Z DISP.
	-0.0055	0.0	0.0001

75
0.0001

RESULTANT JOINT DISPLACEMENTS - FREE JOINTS

JOINT	DISPLACEMENT		ROTATION		2. ROT.
	X DISP.	Y DISP.	Z DISP.	X ROT.	
1	-0.9569	-0.1716			0.0002
2	-0.7916	-0.1773			0.0002
3	-0.6144	-0.1796			0.0003
4	-0.4174	-0.1513			0.0003
5	-0.2169	-0.1046			0.0003
6	-0.1268	-0.0726			0.0002
7	-0.0542	-0.0373			0.0002

76

JO'S TO INCLUDE BL 8							
CHANGES							
CHANGE 10 11001K1 CANTILEVER BLOCK NO 8							
JOI 8 KEL KFX 6.66E5 KNZ 2.64E11							
UNITS FEET							
JOI 8 COORD X 42.26 Y 36.25							
ADDITIONS							
LOADING LIST 4 5 6 7							
STIFFNESS ANALYSIS							
LOADING LIST 4 7							
UNITS INCHES							
LAST DISPLACEMENTS ALL							

20

S.N.C. COMPUTATION

RESULTS OF LATEST ANALYSES

PROBLEM - 100143 TITLE - CANTILEVER BLOCK NO 8

ACTIVE UNITS INCH KIP RAD DEG SEC

ACTIVE STRUCTURE TYPE PLANE FRAME

ACTIVE COORDINATE AXES X Y

LOADING # 6 CON LOAD AT EL 2140

RESULTANT JOINT DISPLACEMENTS - SUPPORTS

JOINT	DISPLACEMENT			ROTATION		
	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
8	0.0150	-0.0	-0.0000			

RESULTANT JOINT DISPLACEMENTS - FREE JOINTS

JOINT	DISPLACEMENT			ROTATION		
	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
1	0.1486	-0.0021				-0.0000
2	0.1220	0.0000				-0.0000
3	0.0966	0.0023				-0.0000
4	0.0706	0.0033				-0.0000
5	0.0446	0.0023				-0.0000
6	0.0316	0.0010				-0.0000
7	0.0193	-0.0004				-0.0000

LOADING - 7 GRAEAO NO 7

RESULTANT JOINT DISPLACEMENTS - SUPPORTS

JOINT	DISPLACEMENT			ROTATION		
	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
8	-0.0069	0.0	0.0			0.0001

78



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RESULTANT JOINT DISPLACEMENTS - FREE JOINTS

JOINT	DISPLACEMENT			ROTATION		
	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
1	-0.8419	-0.1680		0.0002	0.0002	
2	-0.6969	-0.1721		0.0002	0.0002	
3	-0.5399	-0.1666		0.0002	0.0002	
4	-0.3632	-0.1436		0.0003	0.0003	
5	-0.1826	-0.0977		0.0002	0.0002	
6	-0.1029	-0.0667		0.0001	0.0001	
7	-0.0405	-0.0327				

79

A-II Arch Analysis

81
S.N.C. COMPUTATIO

INTEGRATED CIVIL ENGINEERING SYSTEM

- ICES -

FEB 17 1970

TIME=15.45.09

MCDONNELL-ECI ICES EXECUTIVE SYSTEM

IMPLEMENTED 06 FEB 1970 AT

MCDONNELL AUTOMATION COMPANY

ST. LOUIS, MISSOURI

MAC REL. 1.0

STRUOL 'IDIKKI' ARCH DISPLACEMENTS'

ICES STRUDL-II
THE STRUCTURAL DESIGN LANGUAGE

CIVIL ENGINEERING SYSTEMS LABORATORY
MASSACHUSETTS INSTITUTE OF TECHNOLOGY
CAMBRIDGE, MASSACHUSETTS
V1 M1 NOVEMBER, 1969
15 45 26 2/17/70

PREPARED BY K.E. P

S CHARGE TO 2501-01-02-09

S RUN 9 FEB 1970

S RJE JOB NO. '2501 09 112R09501'

TYPE PLANE FRAME

UNITS FEET KIPS RADIAN

JOINT COORDINATES

1	0.0	0.0	S
2	35.8	29.7	
3	75.3	56.5	
4	119.2	78.9	
5	168.8	95.0	
6	225.2	131.3	
7	281.6	95.0	
8	331.2	78.9	
9	375.1	56.5	
10	414.6	29.7	
11	450.4	0.0	S
MEMBER INCIDENCES			
1	1	2	
2	2	3	
3	3	4	

[20]

S.N.C. COMPUTAT

4	4	5						
5	5	6						
6	6	7						
7	7	8						
8	8	9						
9	9	10						
10	10	11						
			THICKNESS	30 FEET ASSUMED				
				AREA OF REINFORCEMENT	600. SQ. INCHES			
MEMB	PROP	PRIS						
1 TO 10	6X	1592.	12	445000.				
UNITS	INCHES							
J01 1	REL	KFX	1.6E5	KFY	1.6E5	KM2	5.18E11	
J01 11	REL	KFX	1.9E5	KFY	1.8E5	KM2	6.26E11	
CONSTANTS								
E	3000.	ALL						
G	1250.	ALL						
UNITS	FEET							
LOADING 1 RADIAL LOAD ON ARCH TRIAL FROM SYSTEM OF EQUATIONS								
MEMBER	LOADS							
1	FORCE	Y	UNIFORM	-2197.0				
2	FORCE	Y	UNIFORM	320.0				
3	FORCE	Y	UNIFORM	65.5				
4	FORCE	Y	UNIFORM	-20.4				
5	FORCE	Y	UNIFORM	79.5				
6	FORCE	Y	UNIFORM	150.0				
7	FORCE	Y	UNIFORM	167.0				
8	FORCE	Y	UNIFORM	185.0				
9	FORCE	Y	UNIFORM	38.5				

10 FORCE Y UNIFORM -41.7

LOADING 2 'TANGENTIAL LOAD TRIAL FROM SYSTEM OF EQUATIONS'

MEMBER LOADS

1 FORCE X UNIFORM -24.8.

2 FORCE X UNIFORM -184.

3 FORCE X UNIFORM -62.

4 FORCE X UNIFORM -60.

5 FORCE X UNIFORM -24.8

6 FORCE X UNIFORM -35

7 FORCE X UNIFORM 2.18

8 FORCE X UNIFORM 24.5

9 FORCE X UNIFORM -93.6

10 FORCE X UNIFORM 44.6.

LOADING COMBINATION 3 'RADIAL+TANGENTIAL COMBINED' COMBINE 1 10, 2 10.

LOADING LIST 1 2 3

STIFFNESS ANALYSIS

LOADING LIST 1 2 3

LIST FORCES REACTIONS ALL



***** RESULTS OF LATEST ANALYSES *****

PROBLEM - IDIKKI TITLE - ARCH DISPLACEMENTS

ACTIVE UNITS FEET KIP RAD DEGF SEC

ACTIVE STRUCTURE TYPE PLANE FRAME

ACTIVE COORDINATE AXES X Y

LOADING - 1

RADIAL LOAD ON ARCH TRIAL FROM SYSTEM OF EQUATIONS

MEMBER FORCES

MEMBER	JOINT	FORCE			MOMENT		
		AXIAL	SHEAR Y	TORSIONAL	BENDING Y	BENDING Z	
1	1	-23349.5820512	-2104.7157959		-582303.9979000		
2	2	23349.5820512	11268.3437500		27274.0625000		
3	2	-22156.5898437	-1943.4140625		-271273.3750000		
4	3	22156.5898437	2181.2056641		-6823.9304437		
5	3	-22210.1875000	-950.1606445		6824.1796875		
6	4	22210.1875000	-2217.9768066		25896.207012		
7	4	-22292.0273437	-1231.8368672		-25695.0781250		
8	4	22292.0273437	2307.6477051		-66704.5625000		
9	5	-21371.5664062	-6746.5507812		66704.6875000		
10	5	21371.5664062	2734.8728027		-321556.0625000		
11	6	-20351.6914062	-6595.4492187		321556.0625000		
12	6	20351.6914062	-1617.1357422		-471329.6875000		
13	7	-20260.7656250	-2511.5526035		-375233.7500000		
14	7	20260.7656250	-6197.0859375		375233.4375000		
15	8	-20983.3242187	2933.0355645		-6024.5279437		
16	8	20983.3242187	-12650.621C937		-6024.5279437		
17	9	-22315.6445312	935.6757812		6034.243437		
18	9	22315.6445312	-11193.4179687		484435.8750000		
19	10	-23289.3867187	893.0355645		-484435.1975000		
20	10	23289.3867187	-7053.9062500		857669.4375000		
21	11						

RESULTANT JOINT LOADS - SUPPORTS

JOINT // X MOMENT // Y MOMENT

FORCE // X MOMENT // Y MOMENT

MEMBER

MOMENT

Y MOMENT

2 NOVENT

S.N.C. COMPUTATION

	X FORCE	Y FORCE	Z FORCE		
1	-16626.6718750	-16528.3593750	11	13420.3789062	-20298.9414052

-58230.4.0625000
85766.5.5625000

LOADING - 2 TANGENTIAL LOAD TRIAL FROM SYSTEM OF EQUATIONS

MEMBER FORCES

MEMBER	JOINT	FORCE			MOMENT		
		AXIAL	SHEAR X	SHEAR Y	TORSIONAL	SENDING X	SENDING Z
1	1	22226.6523437	1223.8508301			236824.8750000	
1	2	-10690.7148437	-1223.8508301			-179896.3125000	
2	2	10523.2812500	2247.1477051			179896.3125000	
2	3	-1740.3156738	-2247.1477051			-72632.0350000	
3	3	1448.2307129	2445.6C96191			72632.0350000	
3	4	1507.4085914	-2445.6C96191			47898.8281250	
4	4	-1972.0717773	21623366699			-47898.8281250	
4	5	5100.92218750	-2152.03566699			160659.4375000	
5	5	-5431.7187500	1391.5727539			-160659.4375000	
5	6	6839.1328125	-1391.5727539			222607.2525000	
6	6	-6911.4257812	-444.3928223			-222607.2525000	
6	7	6931.2890525	444.3928223			197397.5200000	
7	7	-6700.0546875	-1830.1587012			-197397.5200000	
7	8	6586.3710537	1830.1687012			101948.5525000	
8	8	-6216.5078125	-2863.387653			-101948.5525000	
8	9	3009.0429687	2963.387653			-38185.5562500	
8	10	-4617.8046875	-3442.5627441			38185.7627187	
9	9	149.9514150	3462.5527441			-202511.3750000	
9	10	182.0836029	-3441.0126953			202511.4515000	
10	10	-20928.4875000	3441.0126953			-352573.3750000	

RESULTANT JOINT LOADS - SUPPORTS

JOINT	FORCE			MOMENT		
	X FORCE	Y FORCE	Z FORCE	X MOMENT	Y MOMENT	Z MOMENT
1	16324.8554687	15133.4335937				236824.8750000
11	-13909.9218750	16010.7460937				-362573.4375000

LOADING - 3 RADIAL+TANGENTIAL COMBINED

MEMBER FORCES

86
-58230.4.0625000
85766.5.5625000

MEMBER	JOINT	FORCE			MOMENT		
		AXIAL	SHEAR Y	BENDING Z	TORSIONAL	BENDING Y	BENDING Z
1	1	-1122.9277344	-880.3657227			-345479.0562500	
1	2	12692.8632812	10044.4921875			91377.6525000	
2	2	-11633.3045875	-11216.2656250			-91377.5525000	
2	3	20416.2655312	-4058.4535133			-79455.9375000	
2	3	-20761.492187	1495.4487905			79456.1250000	
3	3	23817.5976562	-4723.5859375			73795.0500000	
3	4	-24264.0937500	918.4978027			-73794.8750000	
4	4	27352.9424962	145.3109894			93954.8750000	
4	5	-26803.773437	-5654.9804687			-93954.7500000	
5	5	28210.6952187	1143.2998047			-9894.8.5375000	
5	6	-27263.1717375	-7339.8398437			9594.8.9375000	
6	6	27292.9726562	-1172.7426758			-27394.2.1250000	
6	7	-26960.8125000	-4341.7343750			27394.2.0525000	
7	7	26847.1328125	-4366.9179687			-273284.3750000	
7	8	-27199.3203125	39.6.779797			-44190.1757912	
8	8	25992.3632812	-9207.234687			44199.9563631	
8	9	-26939.4375000	5913.1132812			281624.4375000	
9	9	22465.59320312	-7750.8515625			-281924.6875000	
9	10	-23107.2929687	5552.604697			495096.1250000	
10	10	23616.1997075	-23612.825547				
10	11						

RESULTANT JOINT LOADS - SUPPORTS

JOINT	FORCE			MOMENT		
	X FORCE	Y FORCE	Z FORCE	X MOMENT	Y MOMENT	Z MOMENT
1	-301.8188477	-2394.9257812		-345479.1250000		
11	-489.5439453	-4288.1914362		495096.1250000		

UNIT INCHES
OUTPUT DECIMAL 4

LIST DISPLACEMENTS ALL

88

20

S.N.C. COMPUTATION

RESULTS OF LATEST ANALYSES

PROBLEM - 101021 TITLE - ARCH DISPLACEMENTS

ACTIVE UNITS INCH KIP RAD DEG SEC

ACTIVE STRUCTURE TYPE - PLANE FRAME

ACTIVE COORDINATE AXES X Y

LOADING - 1

RADIAL LOAD ON ARCH TRIAL FROM SYSTEM OF EQUATIONS

RESULTANT JOINT DISPLACEMENTS - SUPPORTS

JOINT	X DISP.	DISPLACEMENT Y DISP.	Z DISP.	X ROT.	ROTATION Y ROT.	Z ROT.
1	0.1039	0.1033				
11	-0.0703	0.1128				

RESULTANT JOINT DISPLACEMENTS - FREE JOINTS

JOINT	X DISP.	DISPLACEMENT Y DISP.	Z DISP.	X ROT.	ROTATION Y ROT.	Z ROT.
2	0.0907	0.1472				
3	0.0600	0.2233				
4	0.0379	0.3062				
5	0.0286	0.3967				
6	0.0399	0.4749				
7	0.0597	0.4810				
8	0.0532	0.4056				
9	0.0100	0.2836				
10	-0.0462	0.1697				

LOADING - 2 TANGENTIAL LOAD TRIAL FROM SYSTEM OF EQUATIONS

RESULTANT JOINT DISPLACEMENTS - SUPPORTS

JOINT	X DISP.	DISPLACEMENT Y DISP.	Z DISP.	X ROT.	ROTATION Y ROT.	Z ROT.

RESULTANT JOINT DISPLACEMENTS - FREE JOINTS

JOINT	DISPLACEMENT			ROTATION		
	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
1	-0.1020	-0.0946				-0.0000
11	0.0728	-0.0889				0.0000
2	-0.1004	-0.1163				-0.0001
3	-0.0806	-0.1540				-0.0001
4	-0.0560	-0.2021				-0.0001
5	-0.0379	-0.2481				-0.0001
6	-0.0295	-0.2714				-0.0000
7	-0.0211	-0.2545				0.0001
8	-0.0005	-0.2093				0.0001
9	0.0315	-0.1565				0.0001
10	0.0632	-0.1132				0.0001

LOADING - 3 RADIAL+TANGENTIAL COMBINED

RESULTANT JOINT DISPLACEMENTS - SUPPORTS

JOINT	DISPLACEMENT			ROTATION		
	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
1	0.0019	0.0087			0.0000	
11	0.0026	0.0238			0.0000	-0.3300

RESULTANT JOINT DISPLACEMENTS - FREE JOINTS

JOINT	DISPLACEMENT			ROTATION		
	X DISP.	Y DISP.	Z DISP.	X ROT.	Y ROT.	Z ROT.
2	-0.0097	0.0309			0.0001	
3	-0.0206	0.0693			0.0001	
4	-0.0181	0.1041			0.0001	
5	-0.0093	0.1436			0.0001	
6	0.0104	0.2036			0.0001	
7	0.0386	0.2275			0.0000	
8	0.0527	0.1953			0.0001	
9	0.0415	0.1271			-0.0001	
10	0.0171	0.0565			-0.0001	

FINISH

A-III. PLATE DIAGRAMS

PLATE 1A Maximum Compressive and Tensile Stress
Under Own Weight - Cantilever Analysis



PLATE 1A

PLATE 1B Maximum Compressive and Tensile Stress
Under Earthquake - Cantilever Analysis

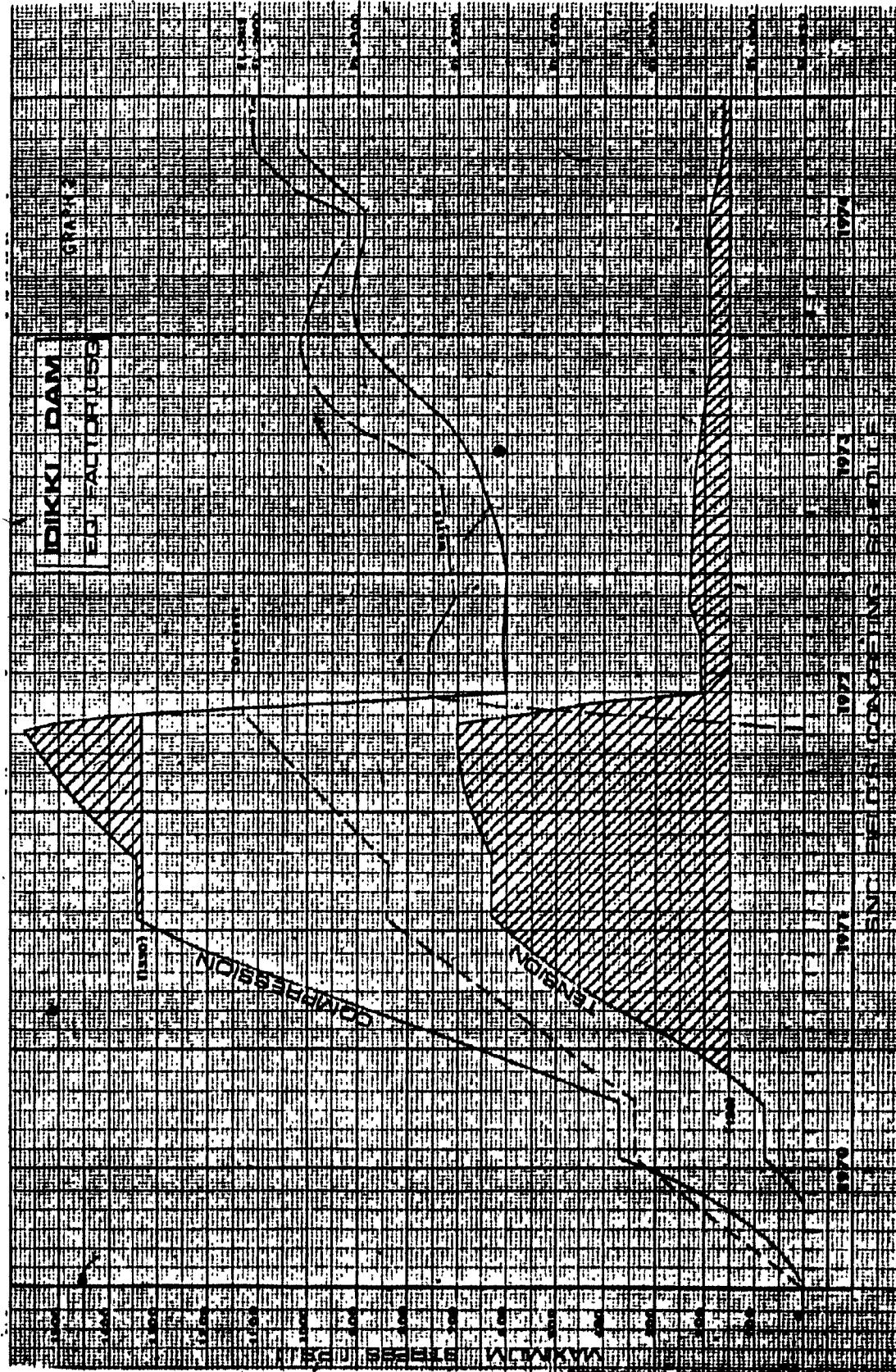


PLATE 1B

PLATE 1C Maximum Cracked Compressive Stress at
Any Point in the Dam

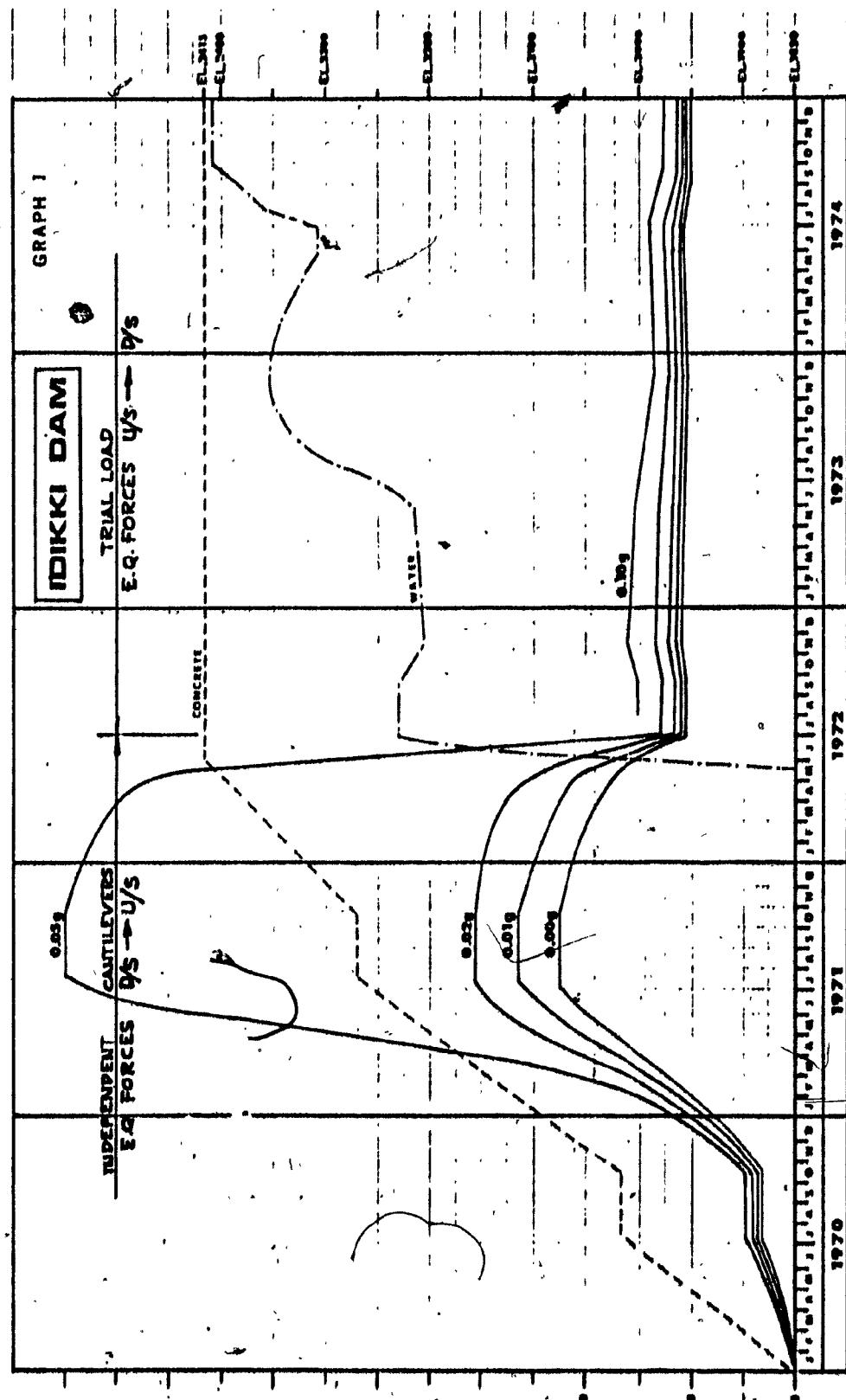
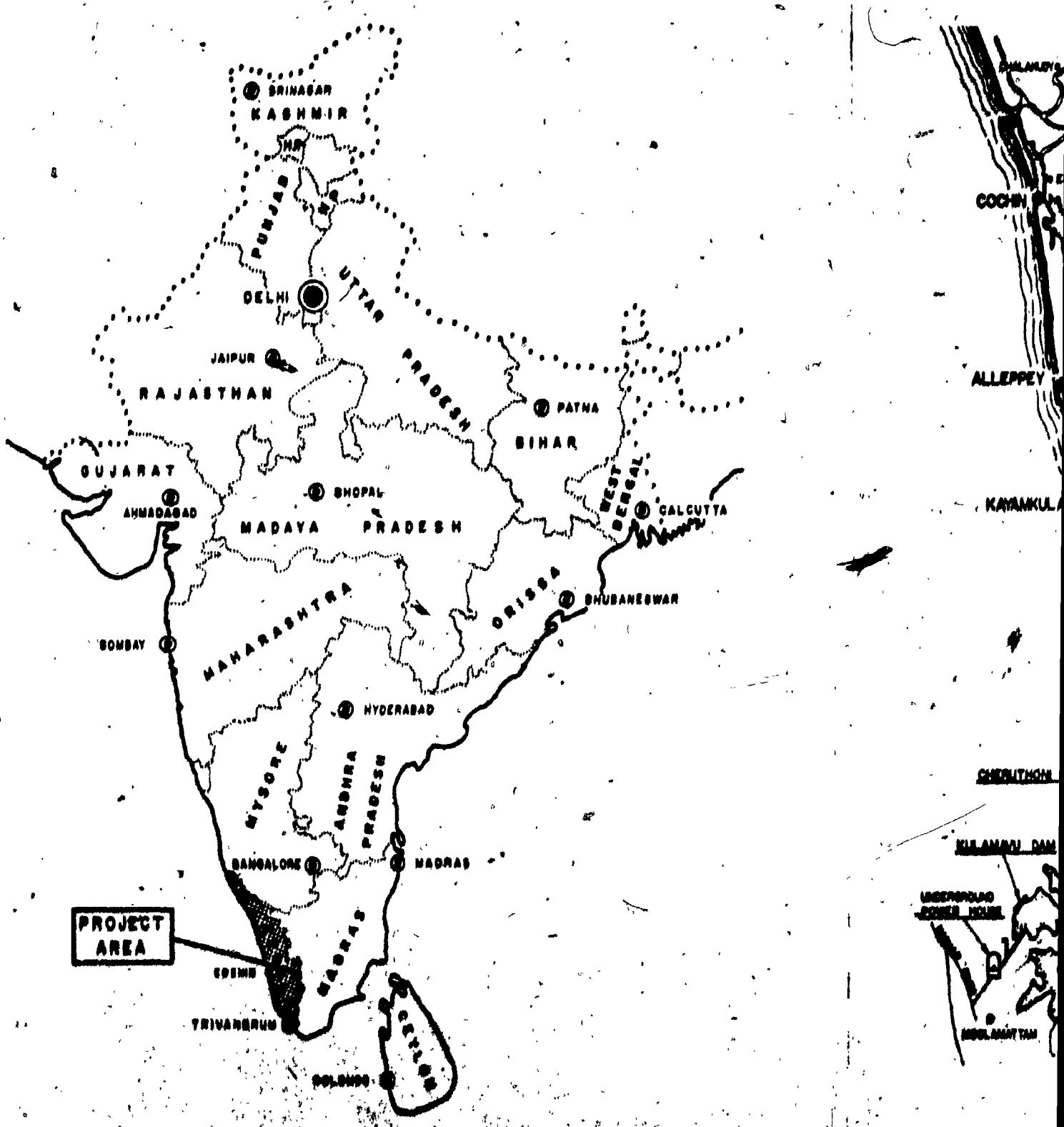
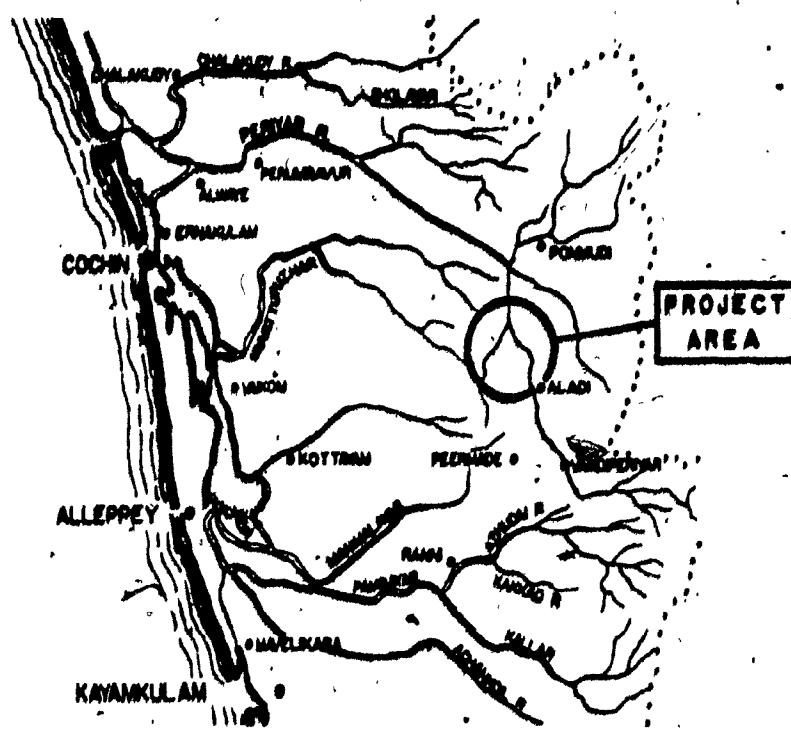


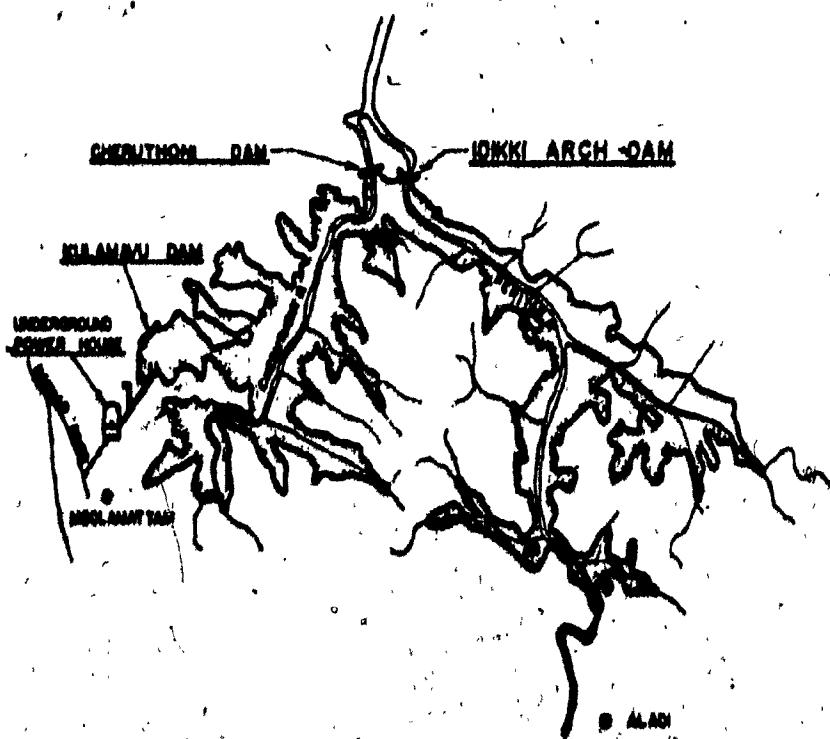
PLATE 2 Idikkki Arch Dam - plan view and
location of the dam



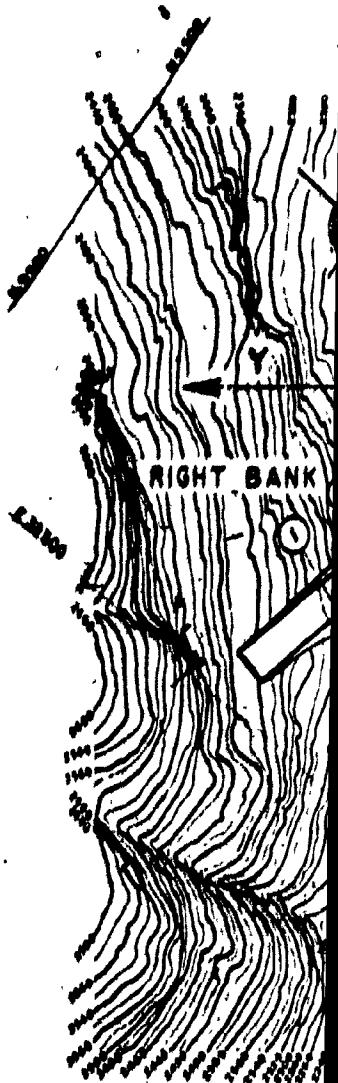
INDIA

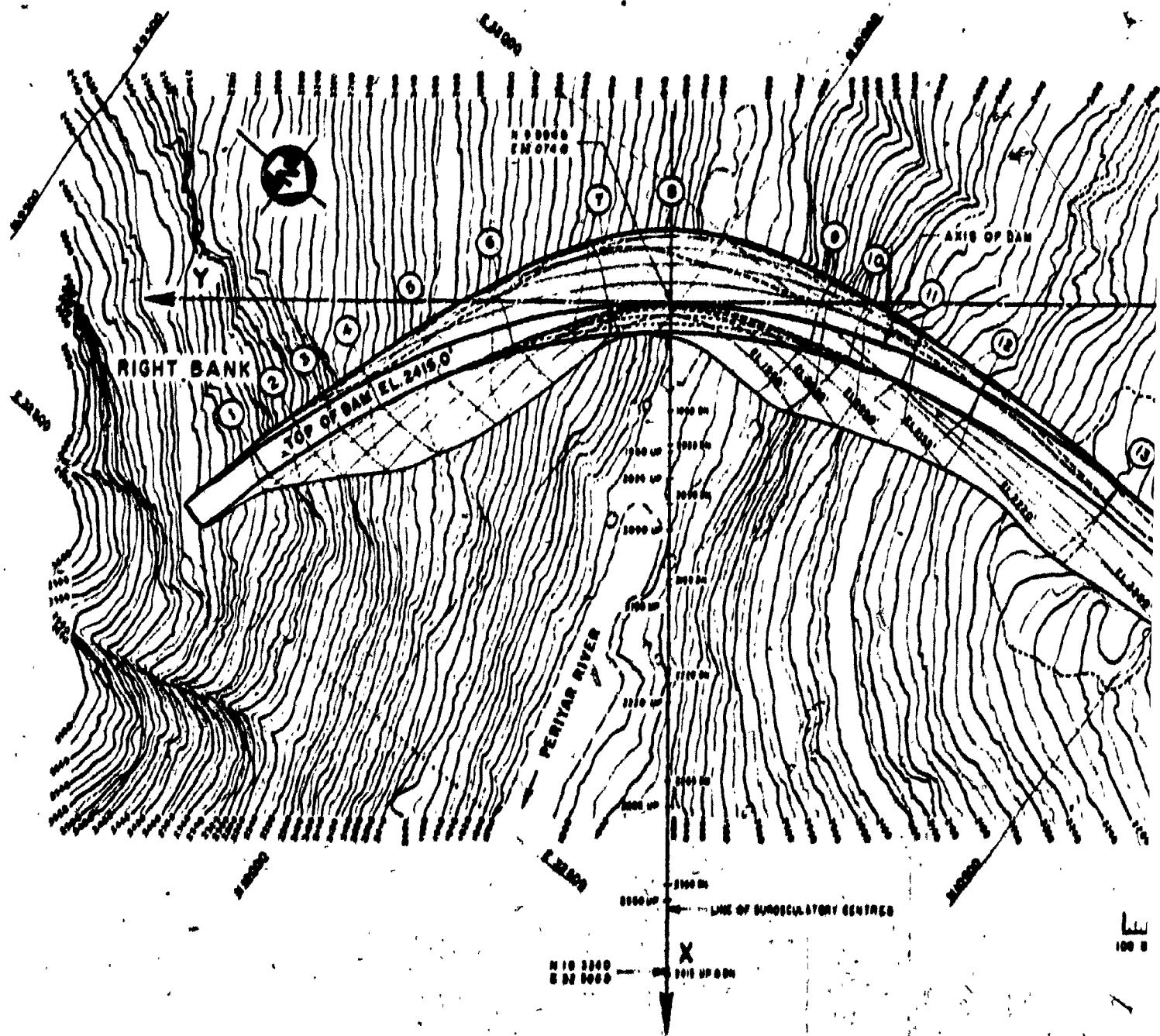


KERALA STATE

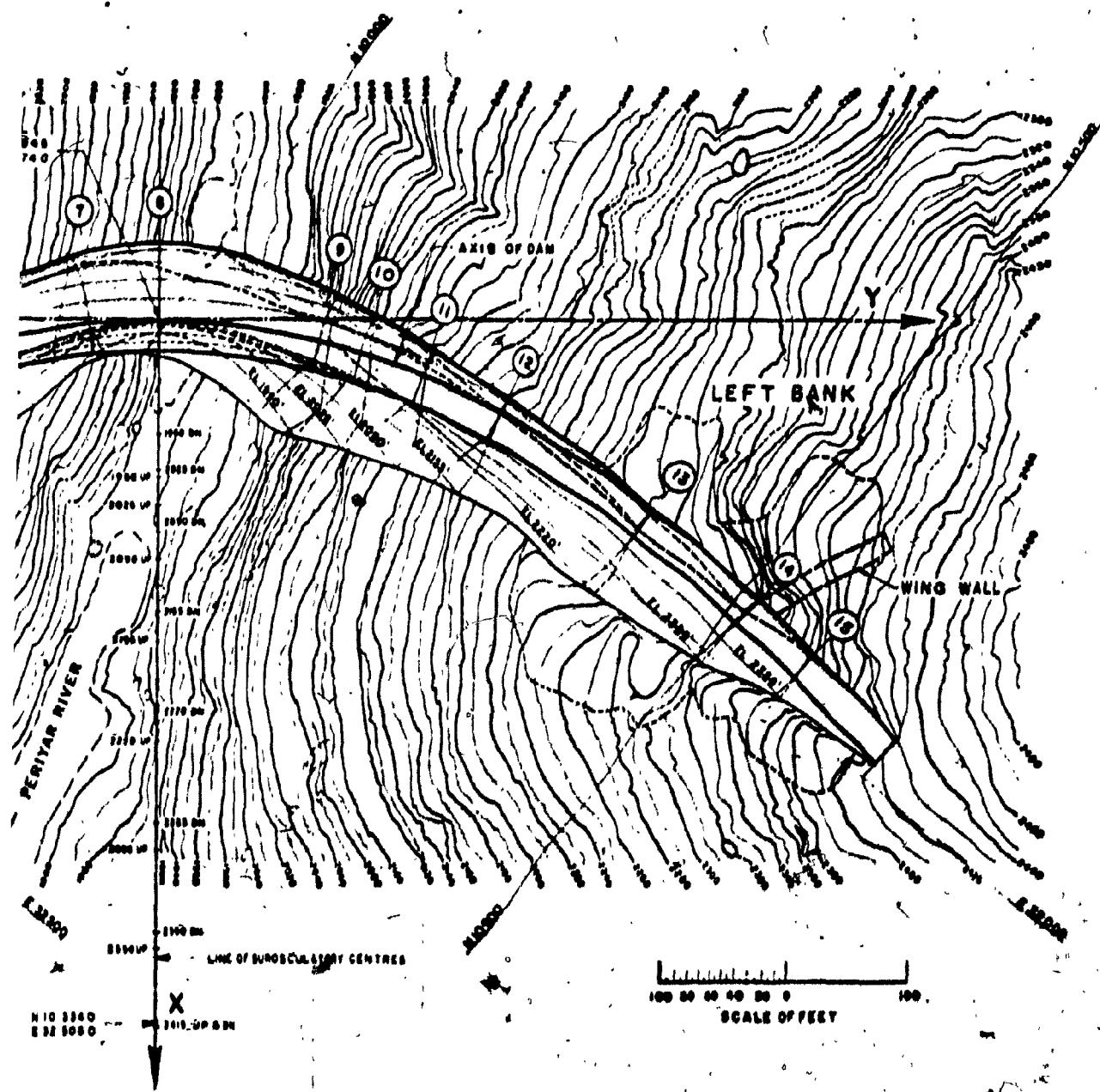


PROJECT AREA





IDIKKI ARCH DAM



IDIKKI ARCH DAM

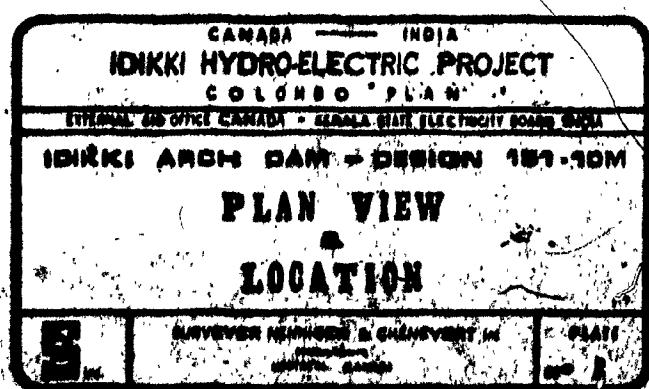
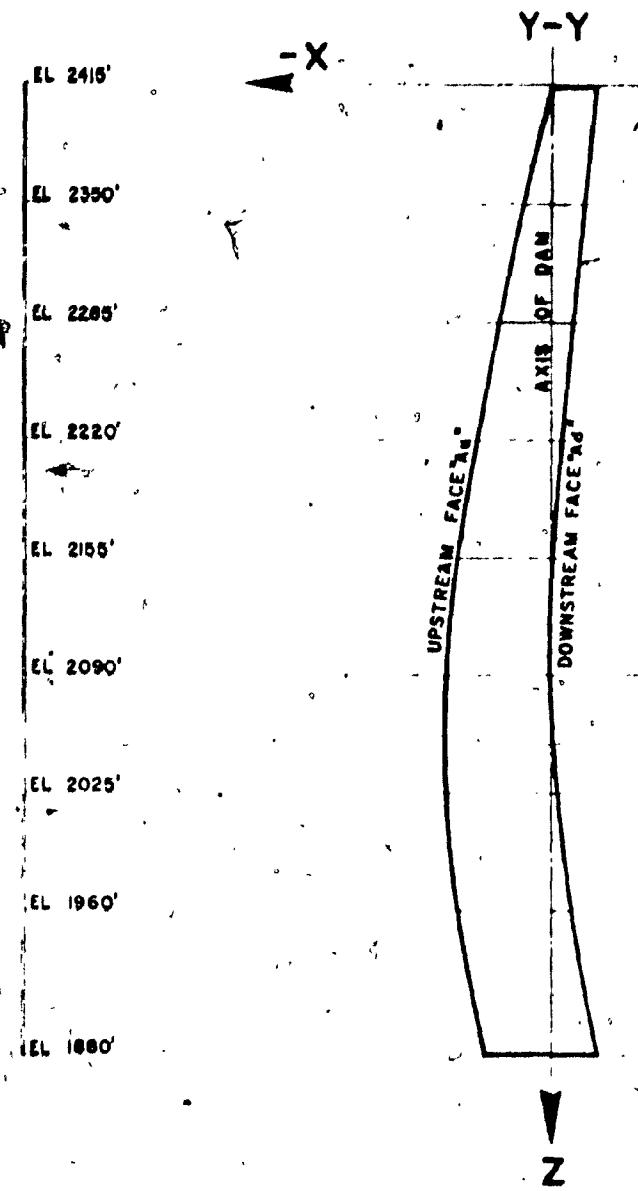


PLATE 3 Idikki Arch Dam - Geometrical
Definition



ANALYTICAL DEFINITION

UPSTREAM FACE "A_u(z)"

$$-1.123 \times 10^{-10} \cdot z^6 - 1.376 \times 10^{-10} \cdot z^5 \\ + 2.164 \times 10^{-9} \cdot z^4 - 0.240 \cdot z^3$$

DOWNSTREAM FACE "A_d(z)"

$$-2.442 \times 10^{-10} \cdot z^6 + 1.484 \times 10^{-10} \cdot z^5 \\ + 6271 \times 10^{-10} \cdot z^4 - 0.06 \cdot z^3 + 25.00$$

THICKNESS "T(z)"

$$-1319 \times 10^{-10} \cdot z^6 + 2.860 \times 10^{-10} \cdot z^5 \\ -1.537 \times 10^{-9} \cdot z^4 + 0.134 \cdot z^3 + 25.00$$

SUROSCULATORY CENTRES

UPSTREAM "S_u(z)"

$$6.842 \times 10^{-10} \cdot z^6 - 7.342 \times 10^{-10} \cdot z^5 \\ + 4.316 \times 10^{-9} \cdot z^4 - 3.695 \times 10^{-8} \cdot z^3 \\ - 2.399 \times 10^{-7} \cdot z^2 - 0.750 \cdot z + 5500$$

DOWNSTREAM "S_d(z)"

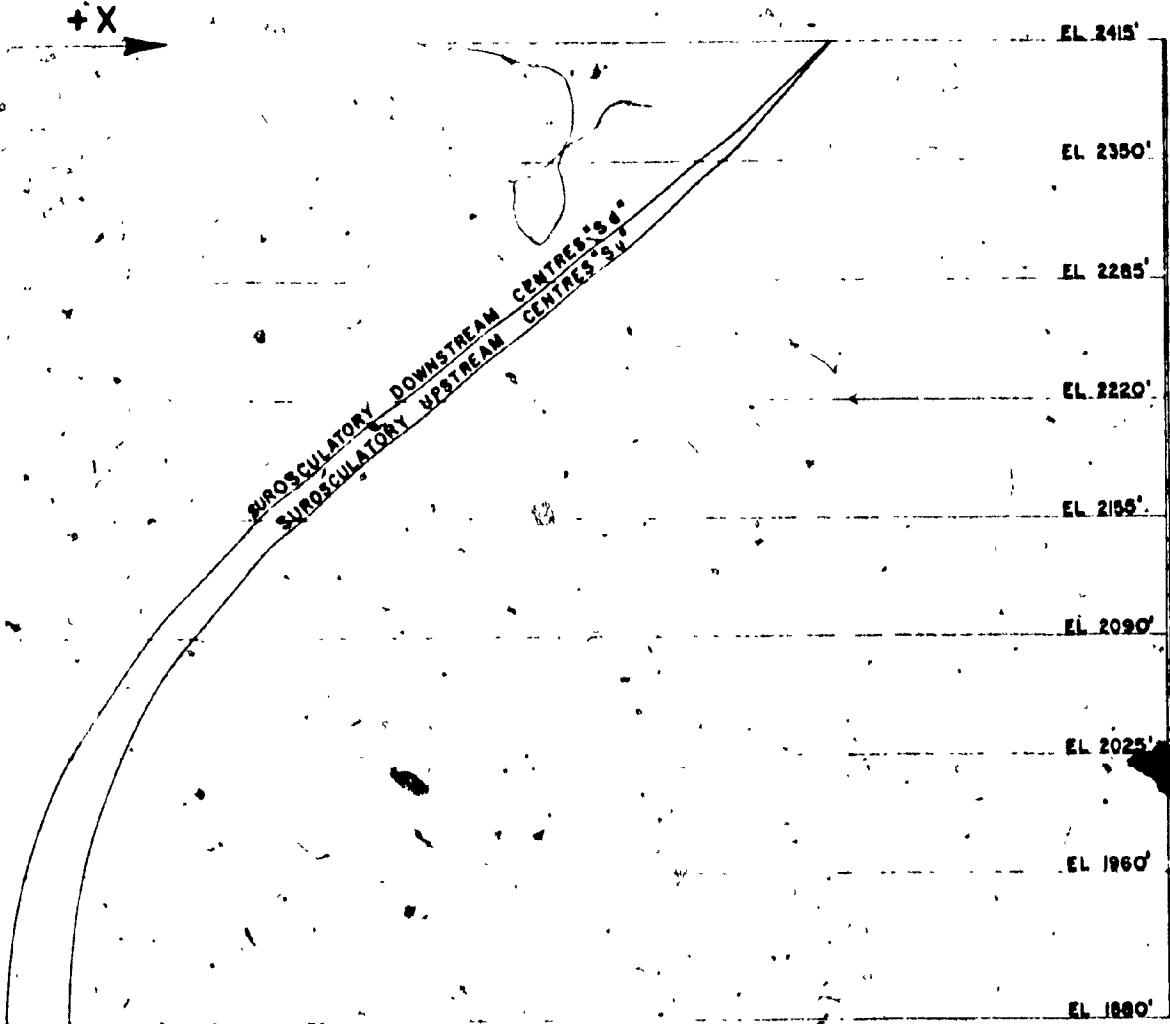
$$-1.229 \times 10^{-10} \cdot z^6 + 2.040 \times 10^{-10} \cdot z^5 \\ - 1.965 \times 10^{-9} \cdot z^4 + 1.398 \times 10^{-8} \cdot z^3 \\ - 3.954 \times 10^{-7} \cdot z^2 - 0.890 \cdot z + 5500$$

NOTE

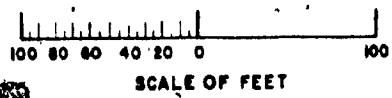
HORIZONTAL PLANS ARE DEFINE
BY PARABOLAS: $y = P(z)(x - A(z))$

UPSTREAM PARAMETER
 $P_u(z) = S_u(z) - A_u(z)$

DOWNSTREAM PARAMETER
 $P_d(z) = S_d(z) - A_d(z)$



CROWN SECTION



DEFINITION

UPSTREAM FACE "Ad(z)"

$$= 1.484 \times 10^{-12} \cdot z^3$$

$$- 0.06 \cdot z + 25.00$$

THICKNESS "T(z)"

$$= -1319 \times 10^{-12} \cdot z^6 + 2.860 \times 10^{-12} \cdot z^3$$

$$- 1537 \times 10^{-9} \cdot z^6 + 0.134 \cdot z + 25.00$$

NOTE

HORIZONTAL PLANS ARE DEFINED BY PARABOLAS: $y^2 = P(z)(x - A(z))$

UPSTREAM PARAMETER

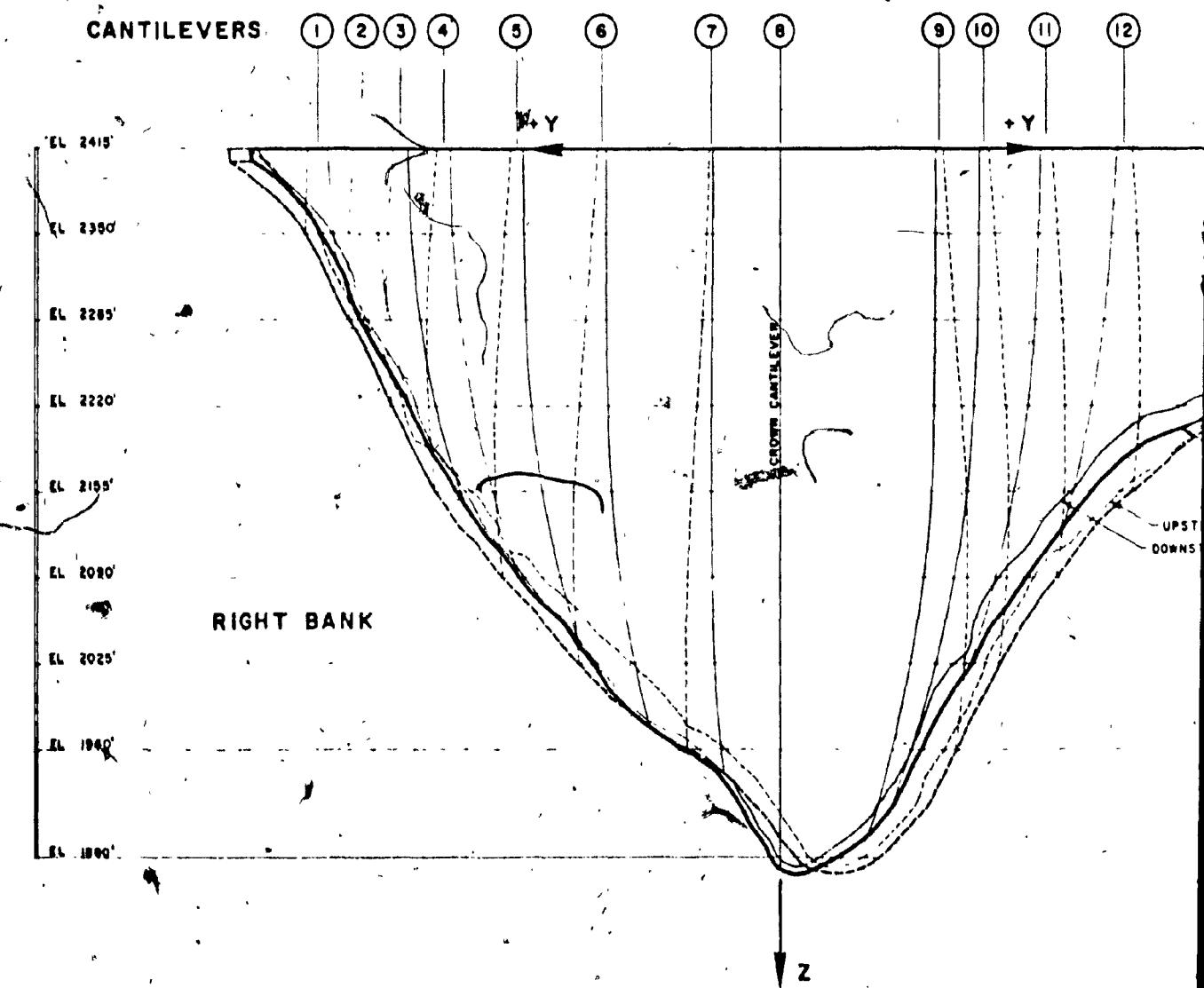
$$P_u(z) = S_u(z) - A_u(z)$$

DOWNSURFACE PARAMETER

$$P_d(z) = S_d(z) - A_d(z)$$

CANADA — INDIA	
IDIKKI HYDRO-ELECTRIC PROJECT	
COLOMBO PLAN	
EXTERNAL AID OFFICE CANADA — KERALA STATE ELECTRICITY BOARD INDIA	
IDIKKI ARCH DAM - DESIGN 151.10M	
GEOMETRICAL DEFINITION	
 INC.	SURVEYOR MENNIGER & CHENEVERT INC. <small>CONTRACTORS MONTREAL, CANADA</small>
PLATE No. 3	

PLATE 4 Idikki Arch Dam - Elevation and
Projection on y-y axis



10f

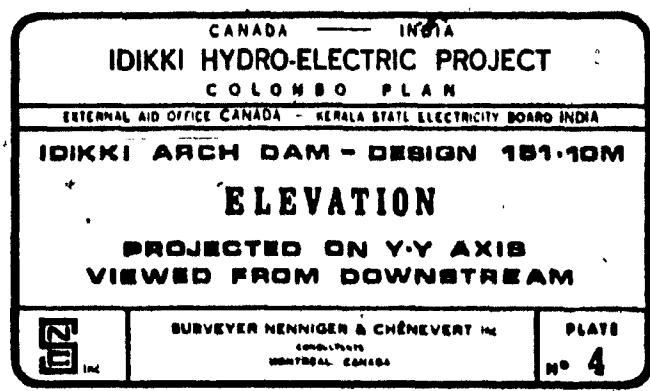
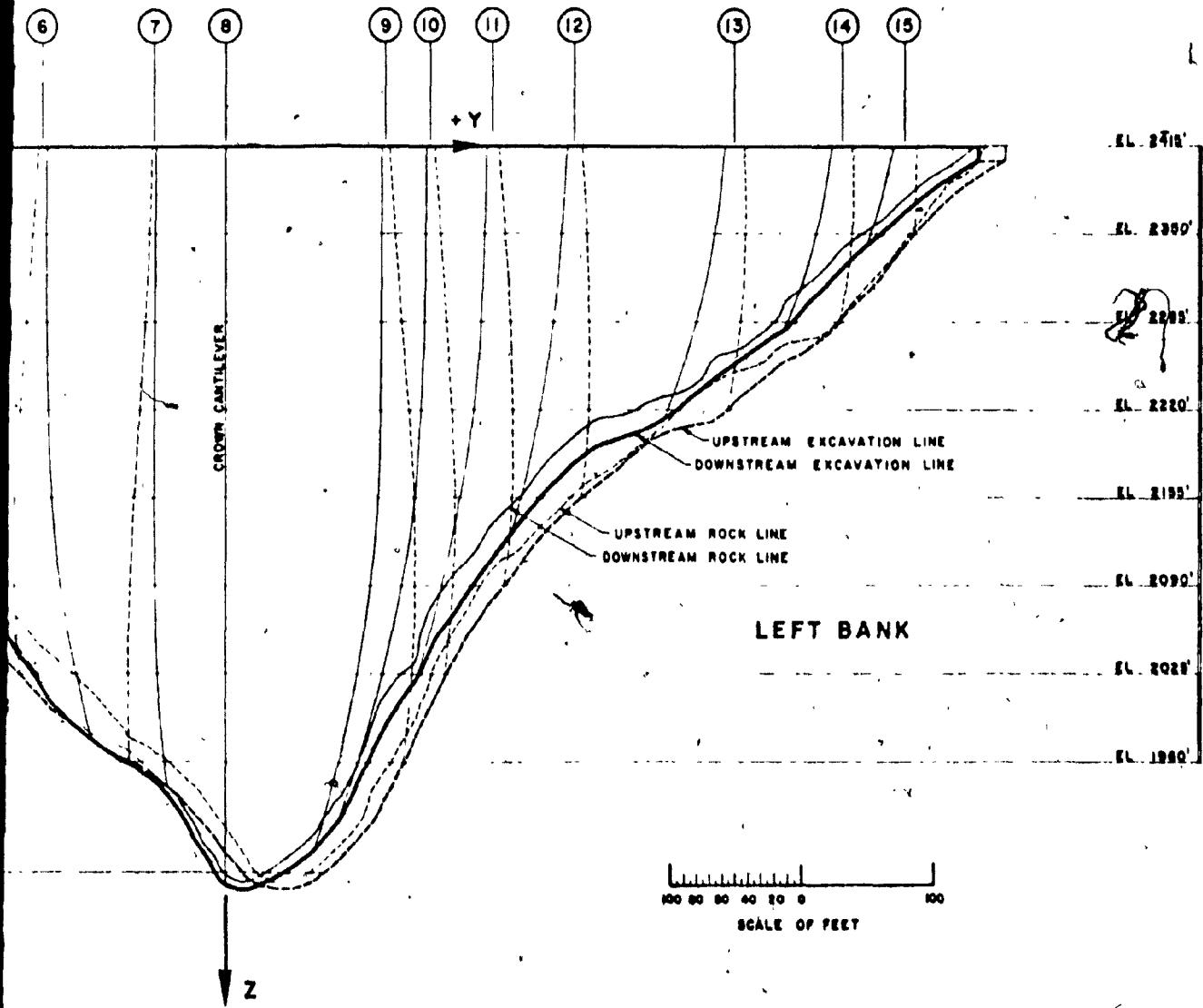
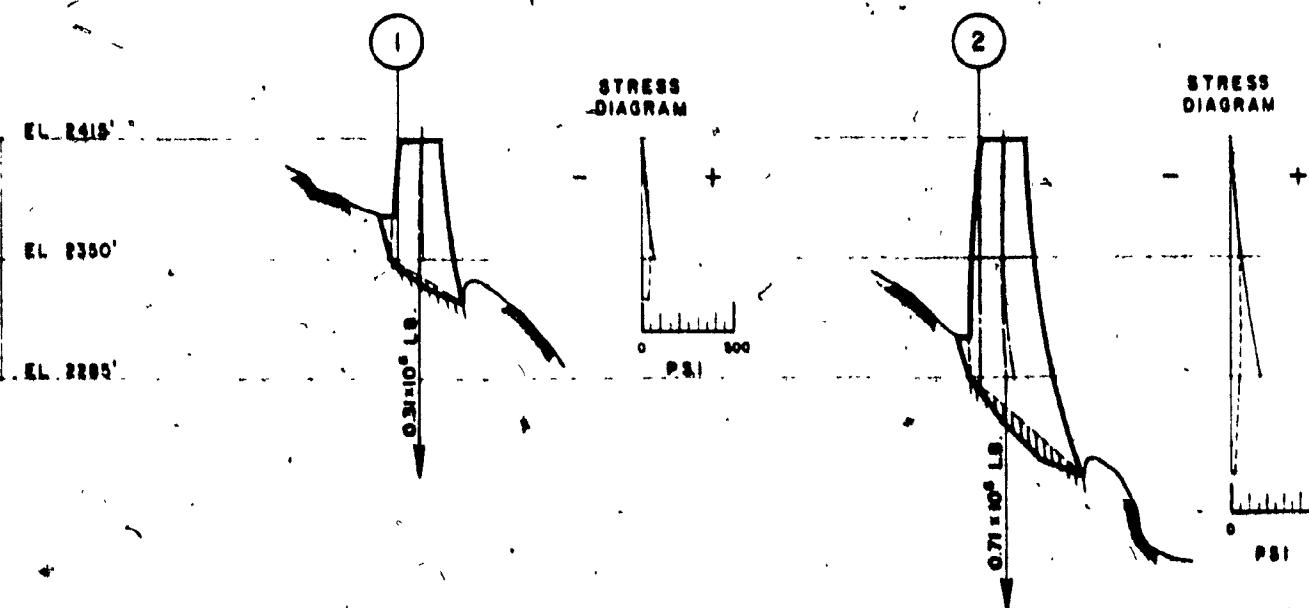
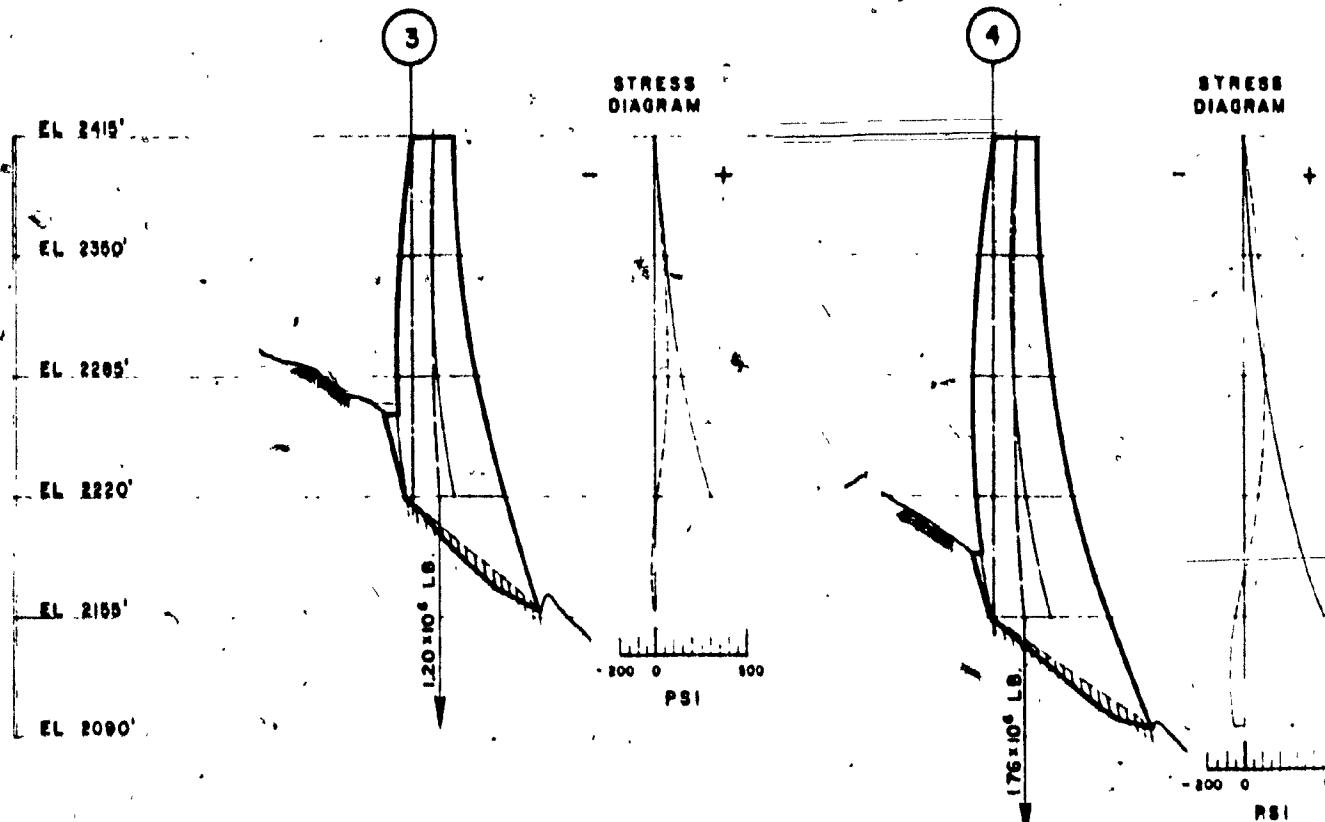
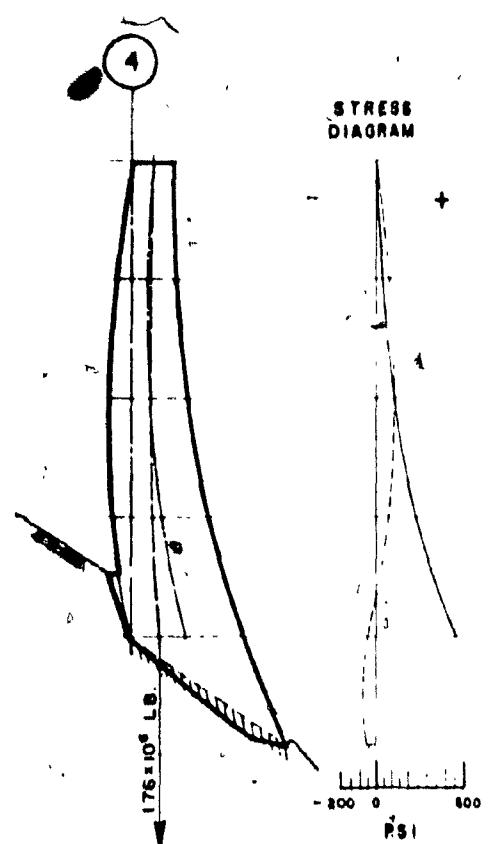


PLATE 5A Idikki Arch Dam - Cantilevers
1 to 8

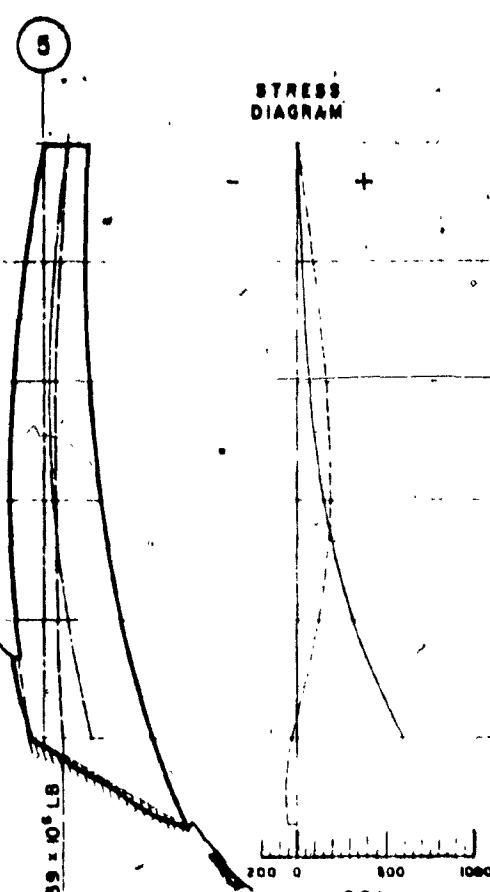


1 of 1

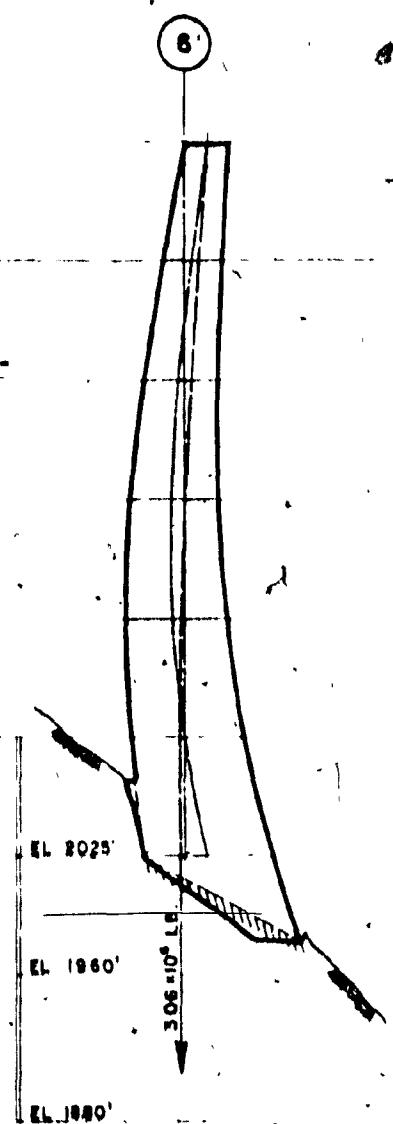
STRESS
DIAGRAM



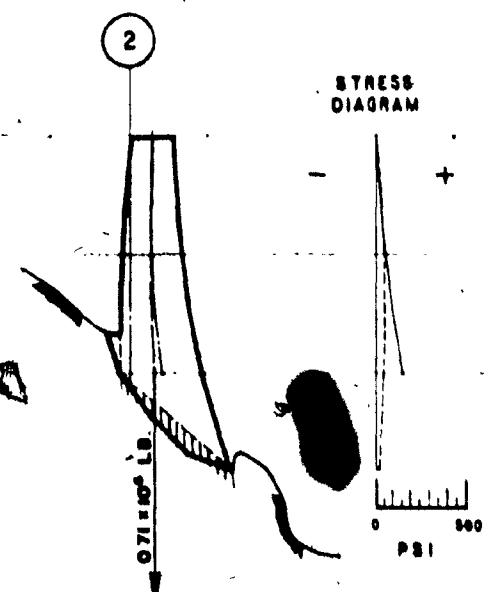
STRESS
DIAGRAM



6



STRESS
DIAGRAM

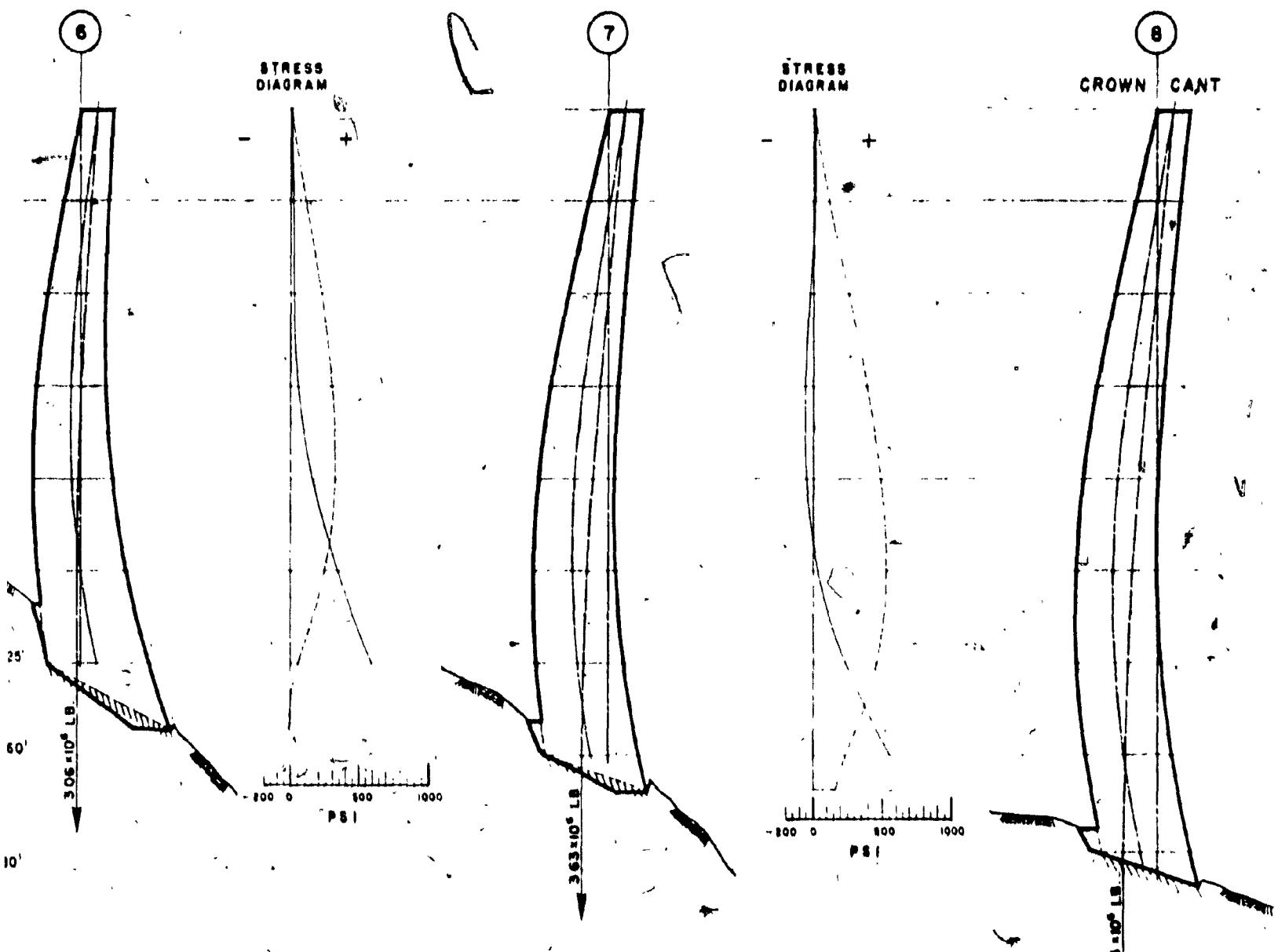


EL 2415'

EL 2380'

EL 2285'

LOADING C

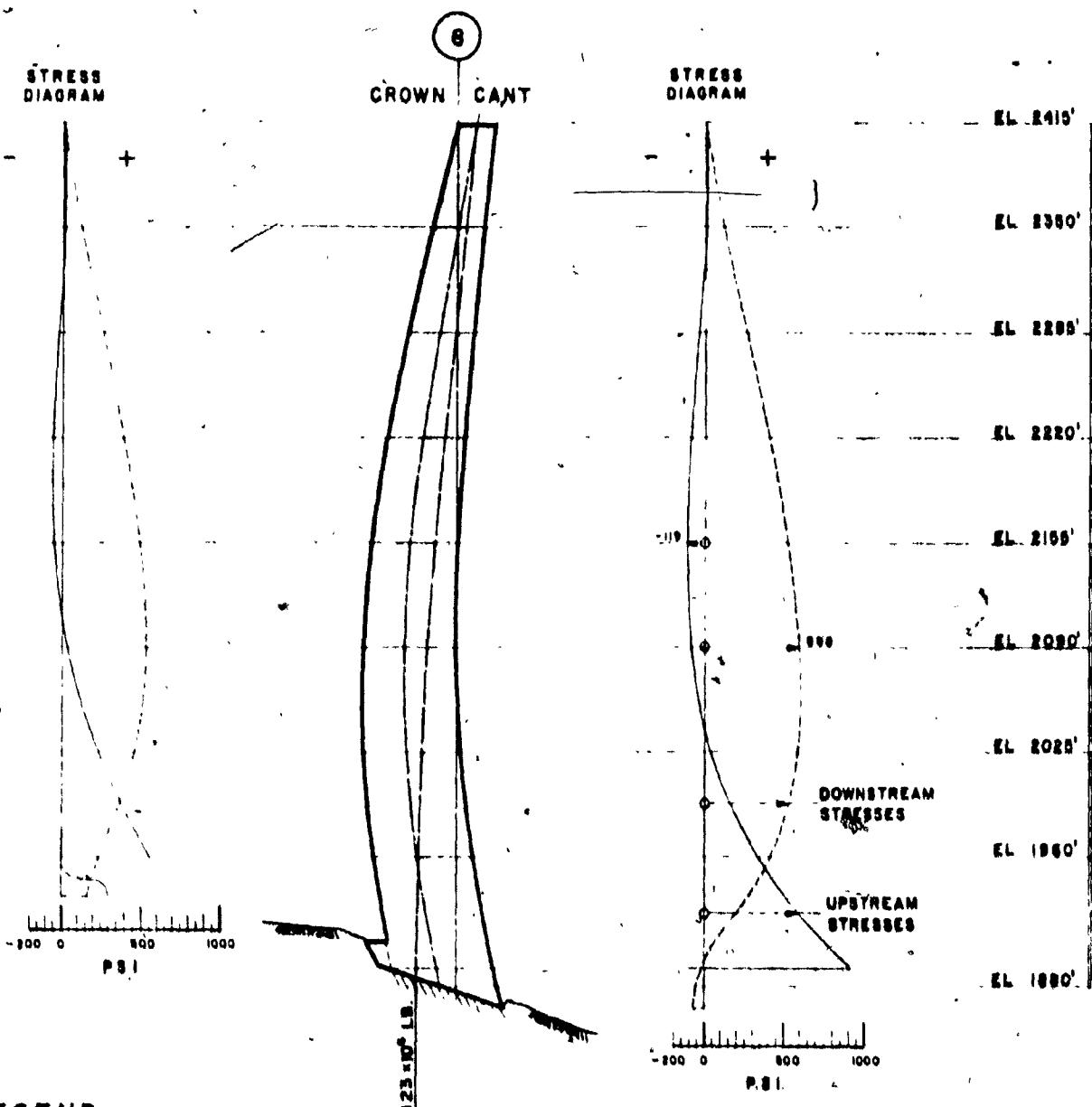


LEGEND

- (1) CANTILEVER NUMBER
- AXIS OF DAM
- GRAVITY CENTER LINE
- POSITION OF RESULTANT FORCE
- ROCK
- FOUNDATION FOR COMPUTATIONS
- THEORETICAL CANTILEVER AT FOUNDATION
- RESULTANT AT FOUNDATION
- STRESS DIAGRAM
- UPSTREAM FACE
- DOWNSTREAM FACE
- TENSILE STRESSES
- + COMPRESSIVE STRESSES

LOADING CONDITION 3.5c

3 of

**LEGEND**

- CANTILEVER NUMBER
- AXIS OF DAM
- GRAVITY CENTER LINE
- POSITION OF RESULTANT FORCE
- ROCK
- FOUNDATION FOR COMPUTATIONS
- THEORETICAL CANTILEVER AT FOUNDATION
- RESULTANT AT FOUNDATION
- UPSTREAM FACE
- - - DOWNSTREAM FACE
- - TENSILE STRESSES
- + COMPRESSIVE STRESSES

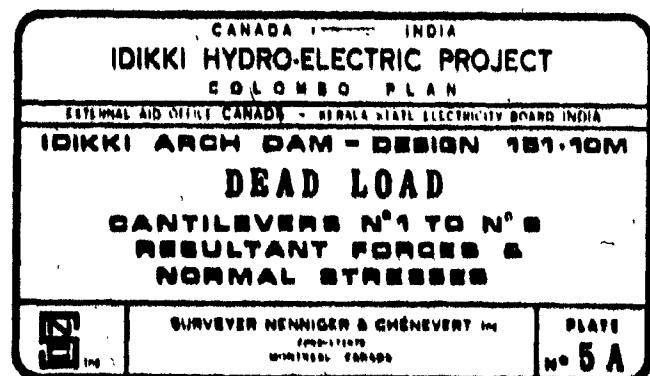


PLATE 5B Idikki ARch Dam - Cantilevers
9 to 15

CANTILEVERS

EL. 2410'

EL. 2350'

EL. 2280'

EL. 2160'

EL. 2090'

EL. 2025'

EL. 1960'

9

STRESS
DIAGRAM



CANTILEVERS

14

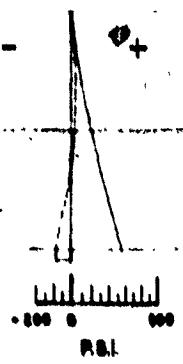
STRESS
DIAGRAM

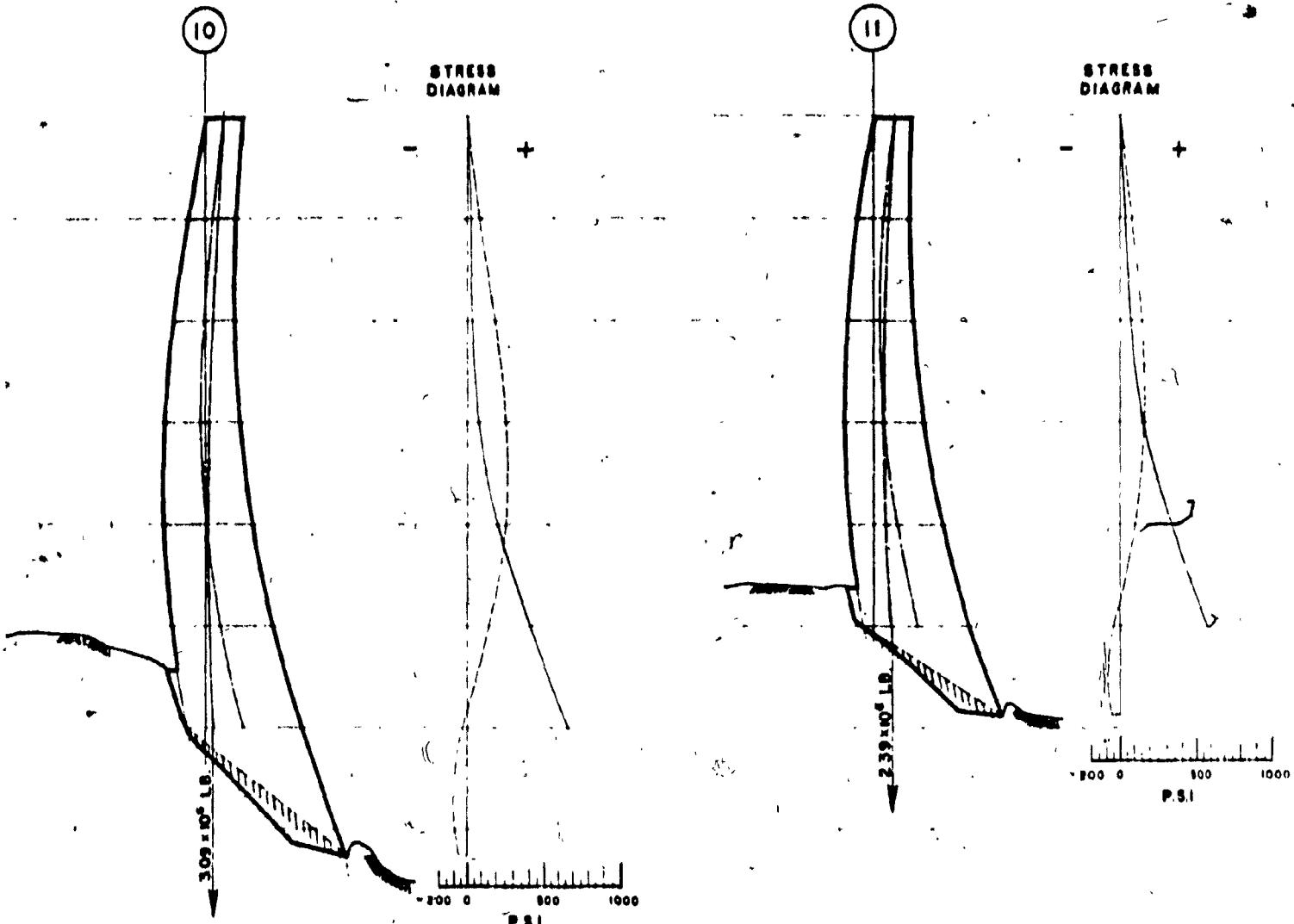
EL. 2410'

EL. 2350'

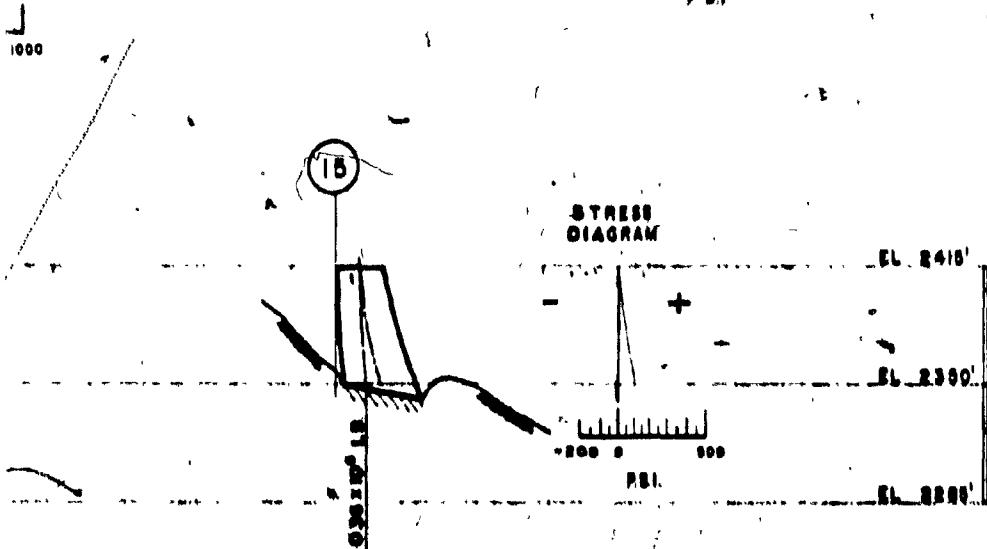
EL. 2280'

10f



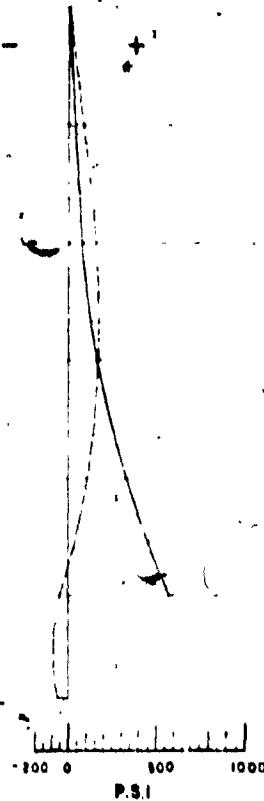


SCALE OF FEET.



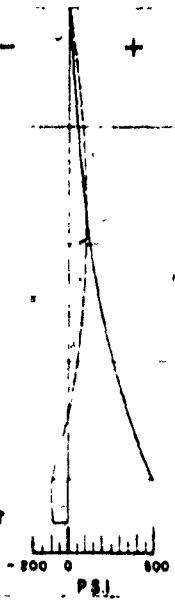
LOADING CONDITION 3

STRESS
DIAGRAM

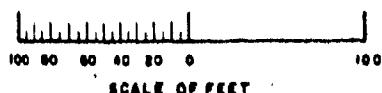


12

STRESS
DIAGRAM



13



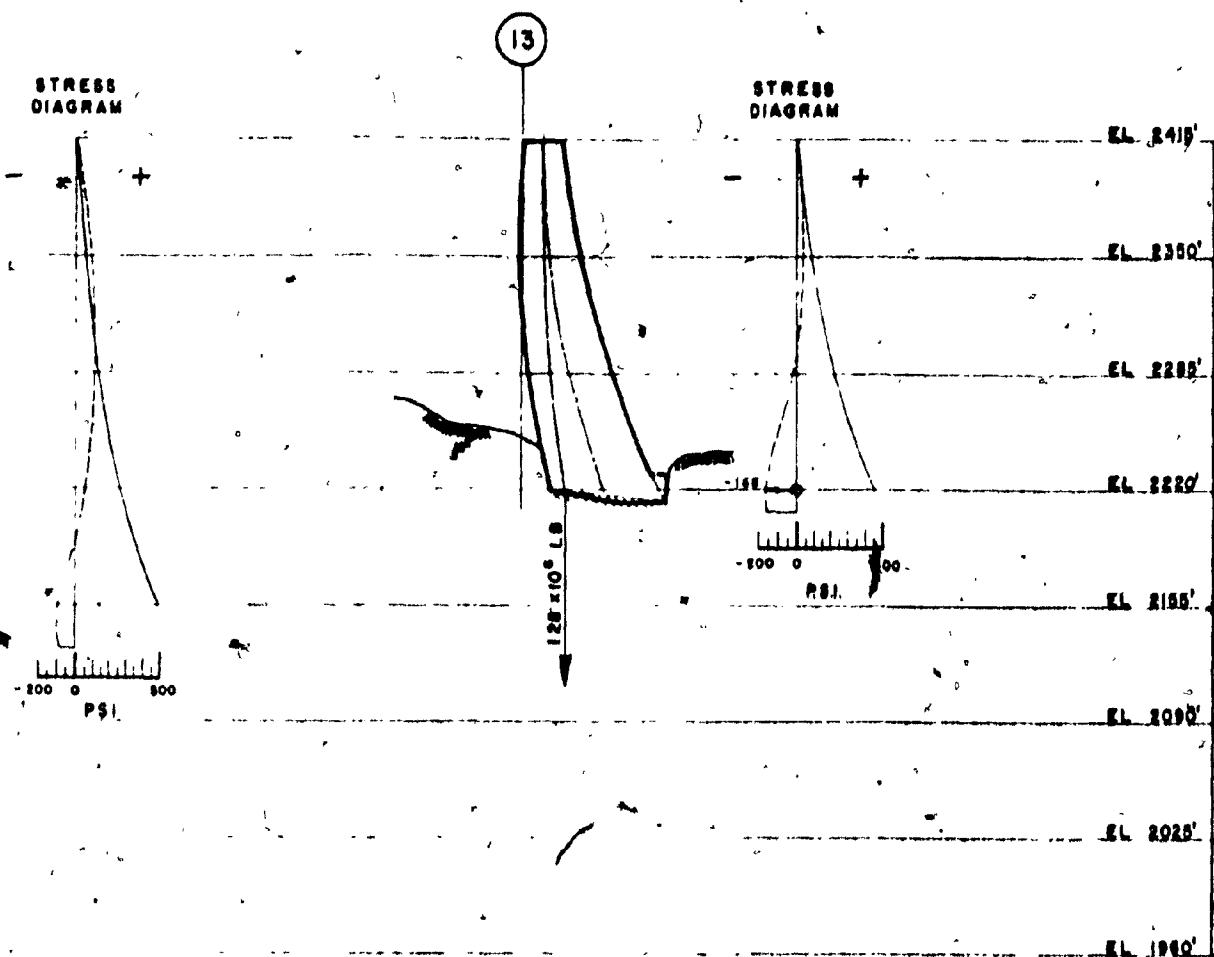
LEGEND

(1)

- CANTILEVER NUMBER
 - AXIS OF DAM
 - GRAVITY CENTER LINE
 - POSITION OF RESULTANT FORCE
 - ROCK
 - FOUNDATION FOR COMPUTATIONS
 - THEORETICAL CANTILEVER AT FOUNDATION
 - RESULTANT AT FOUNDATION
- STRESS
DIAGRAM
- - - UPSTREAM FACE
 - - - DOWNSTREAM FACE
 - TENSILE STRESSES
 - + COMPRESSIVE STRESSES

LOADING CONDITION 3.5a

3 of 1



LEGEND

- CANTILEVER NUMBER
- AXIS OF DAM
- GRAVITY CENTER LINE
- POSITION OF RESULTANT FORCE
- ROCK
- FOUNDATION FOR COMPUTATIONS
- THEORETICAL CANTILEVER AT FOUNDATION
- RESULTANT AT FOUNDATION
- UPSTREAM FACE
- DOWNTSTREAM FACE
- TENSILE STRESSES
- + COMPRESSIVE STRESSES

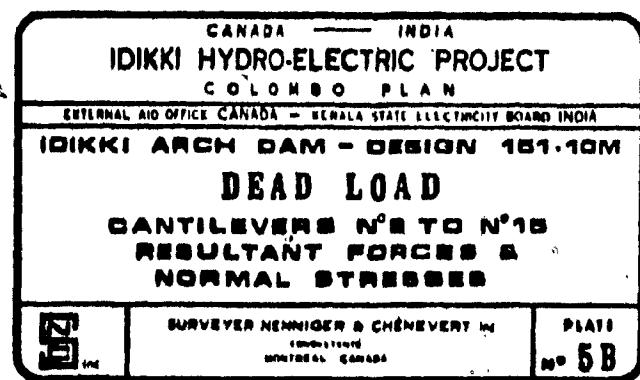
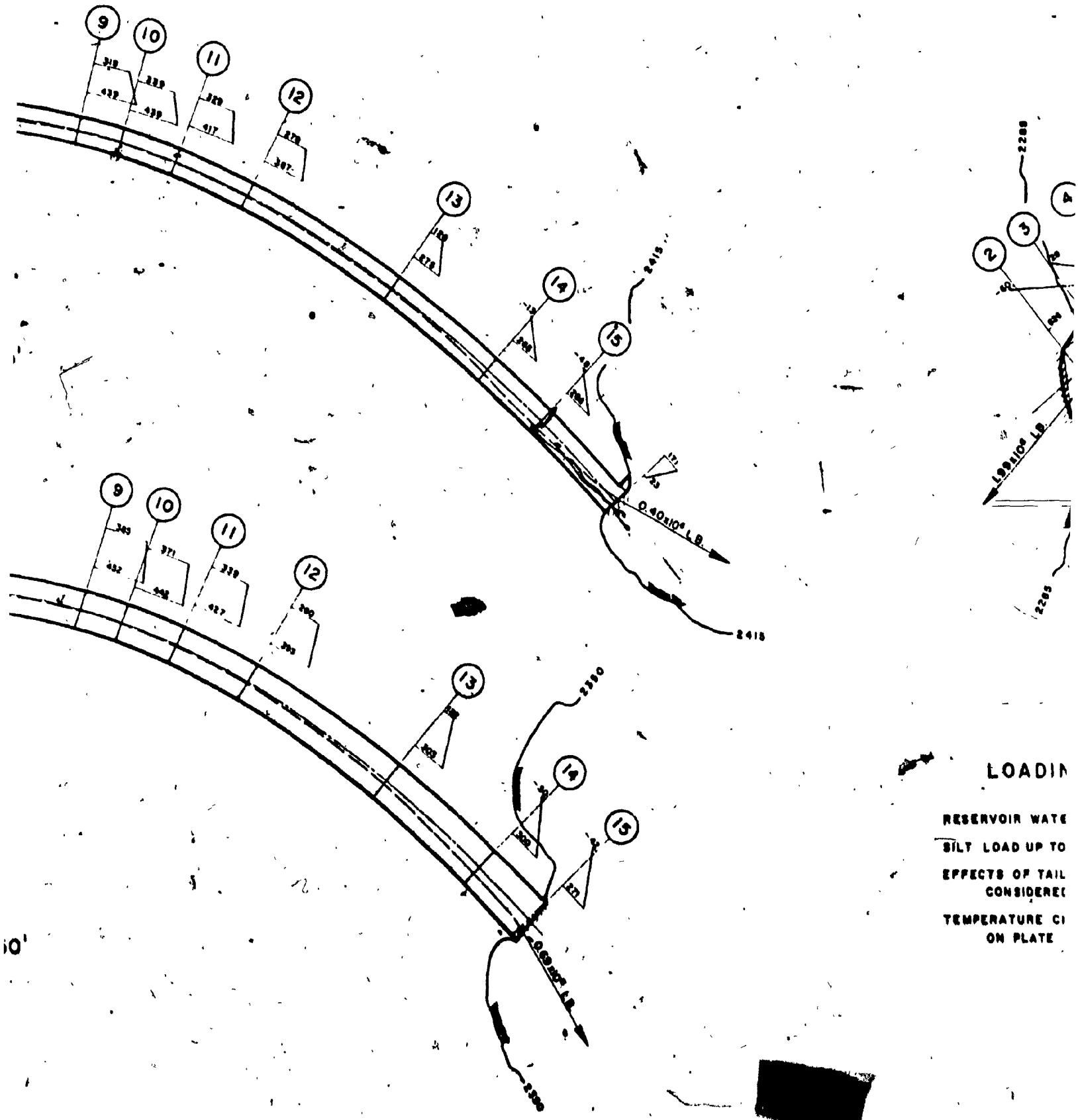


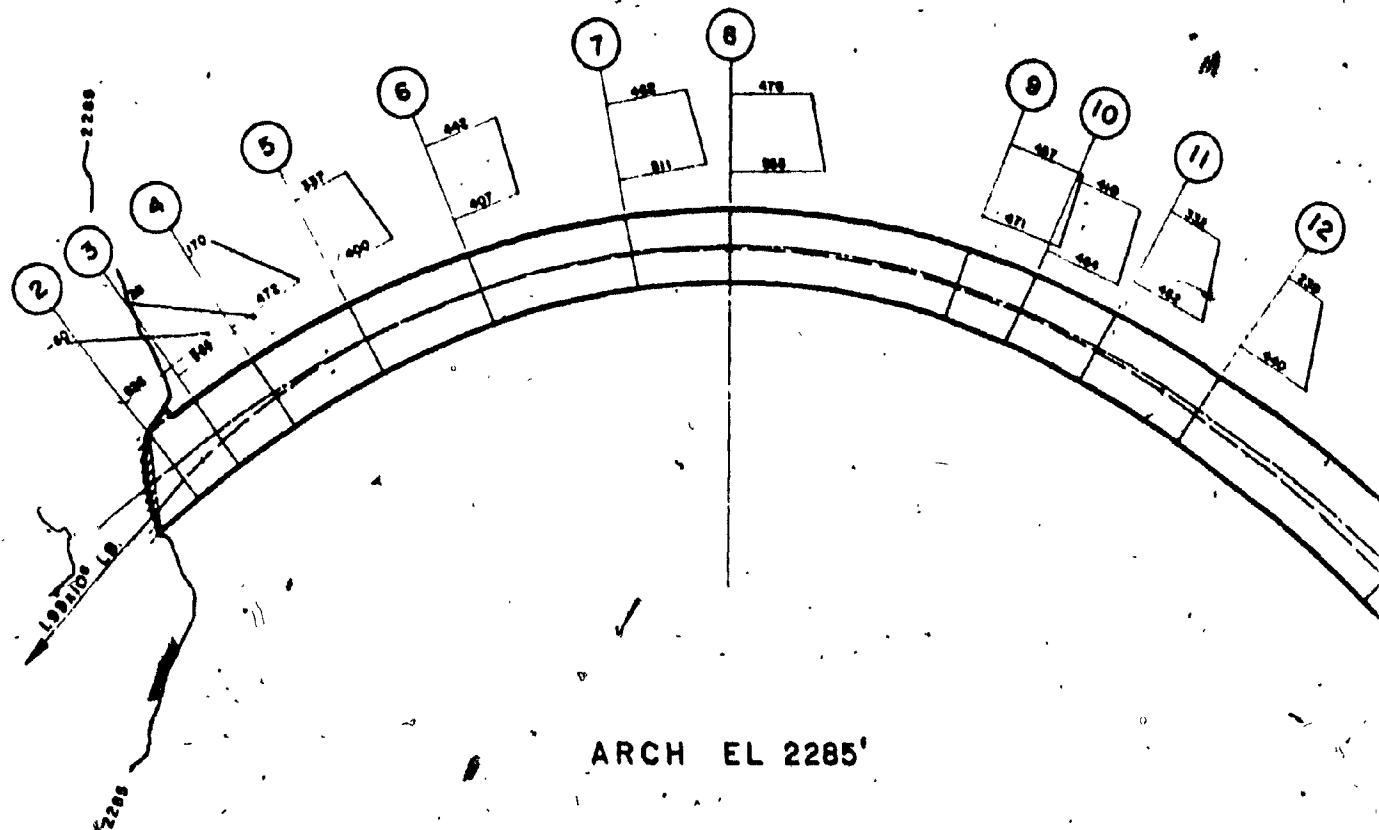
PLATE 6A Idikki Arch Dam - Arches El.2285
to El.2415

ARCH EL 2415'

ARCH EL 2350'

182





ARCH EL 2285'

LEGEND

LOADING CONDITION 3.5 b

RESERVOIR WATER SURFACE EL 2408.5 FT.

SILT LOAD UP TO EL 2100 FT

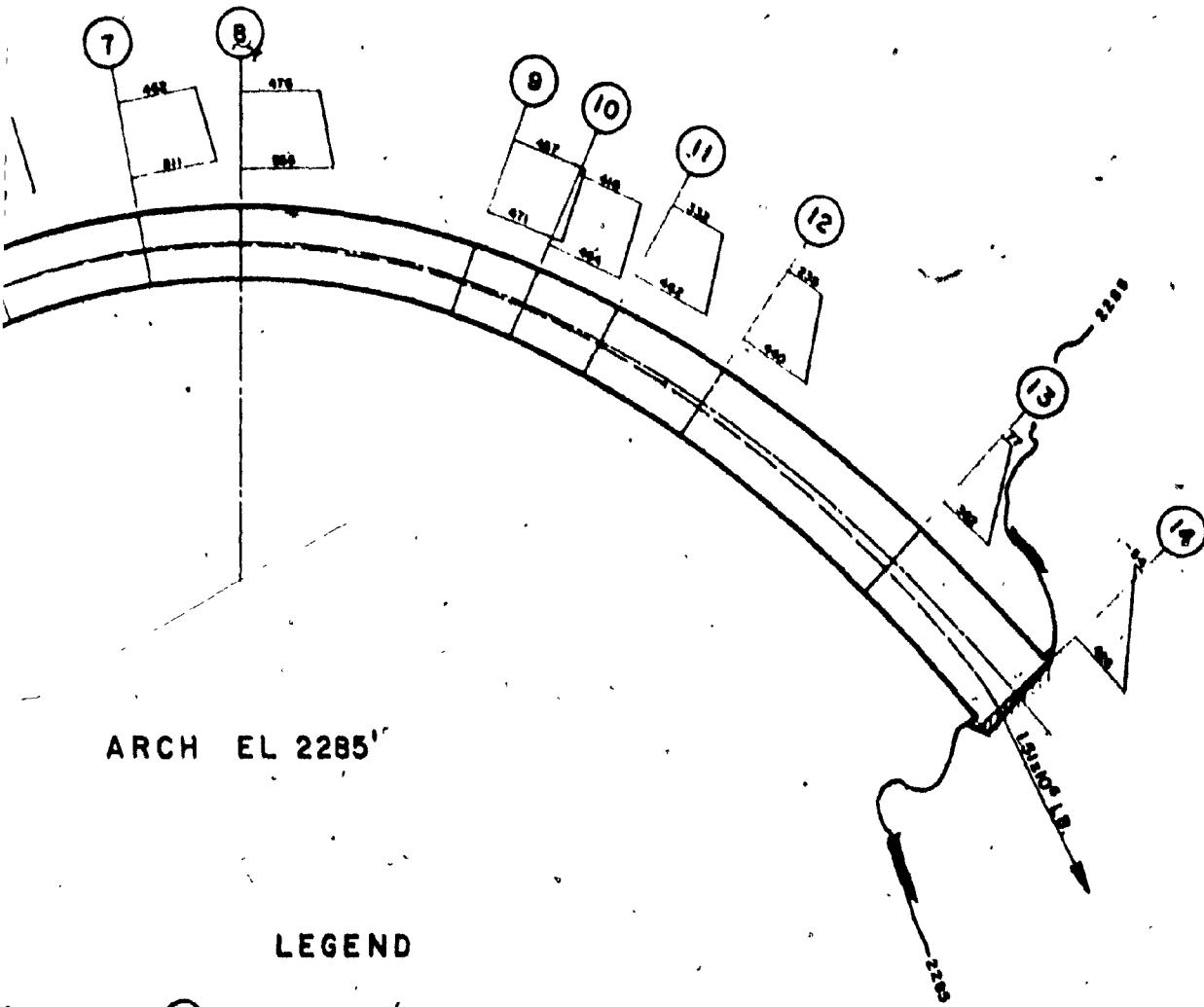
EFFECTS OF TAILWATER AND UPLIFT ARE NOT
CONSIDERED

TEMPERATURE CHANGES IN CONCRETE AS SHOWN
ON PLATE N°14

- (1) CANTILEVER NUMBER
- PLANE OF CENTERS
- — ARCH CENTER LINE
- — — POSITION OF RESULTANT FORCE
- — — ROCK
- — — FOUNDATION FOR COMPUTATIONS
- — — THEORETICAL ARCH ABUTMENT
- STRESS DIAGRAM
- RESULTANT AT ABUTMENT

3 of 1

10



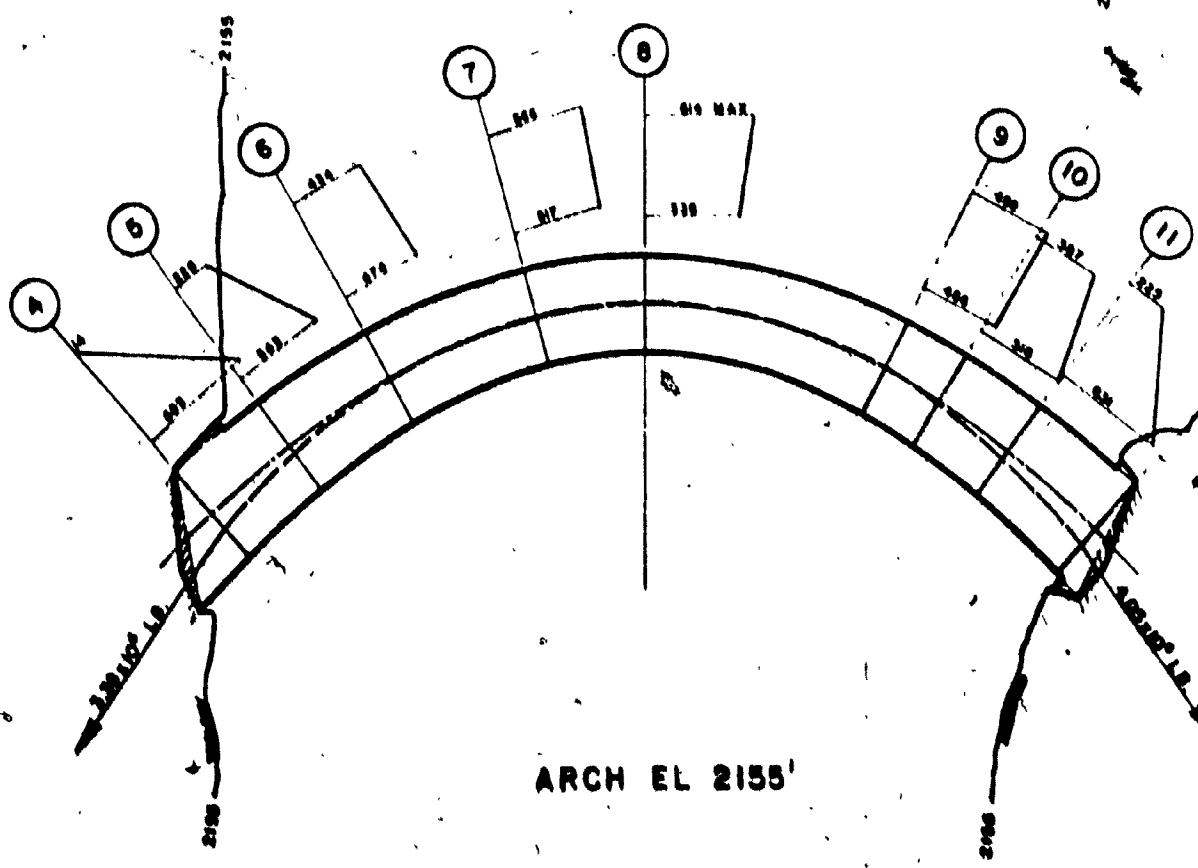
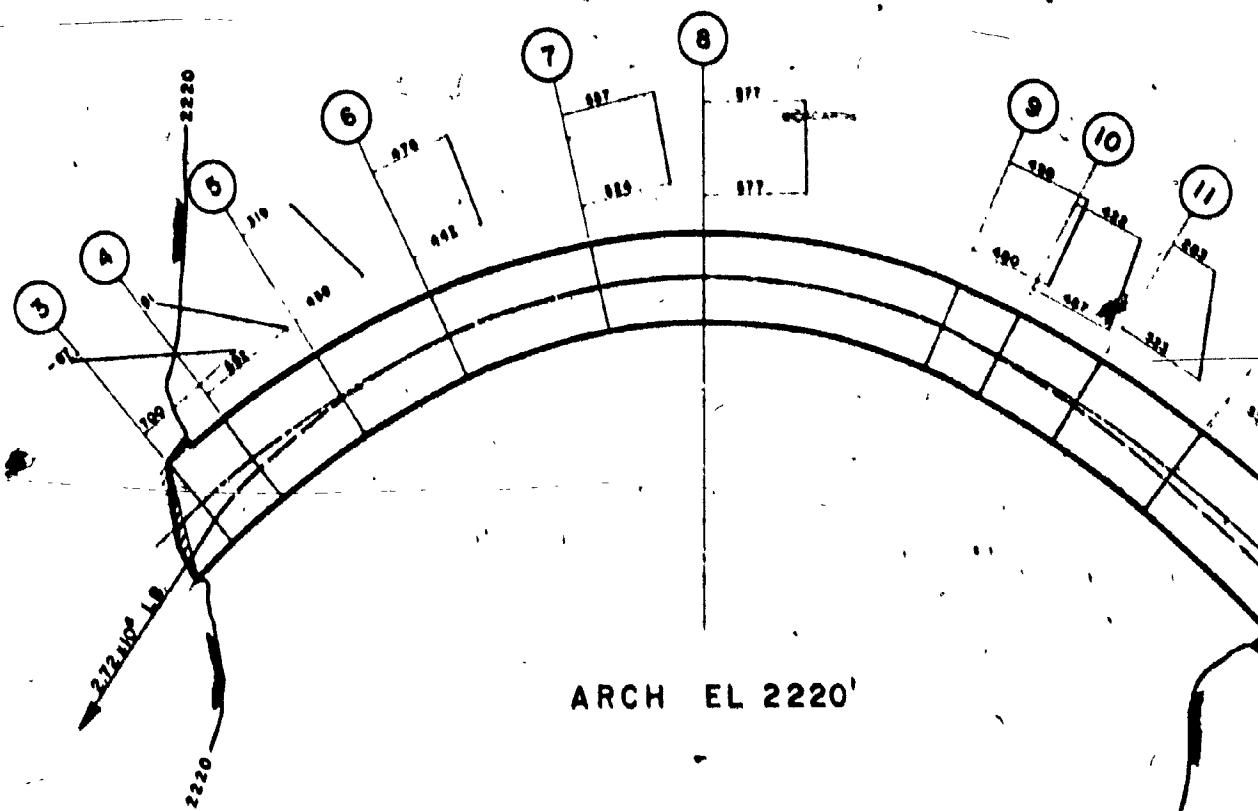
LEGEND

- 1 CANTILEVER NUMBER
- PLANE OF CENTERS
- — ARCH CENTER LINE
- — — POSITION OF RESULTANT FORCE
- — — — ROCK
- — — — — FOUNDATION FOR COMPUTATIONS
- — — — — — THEORETICAL ARCH ABUTMENT
- — — — — — — STRESS DIAGRAM
- — — — — — — — RESULTANT AT ABUTMENT

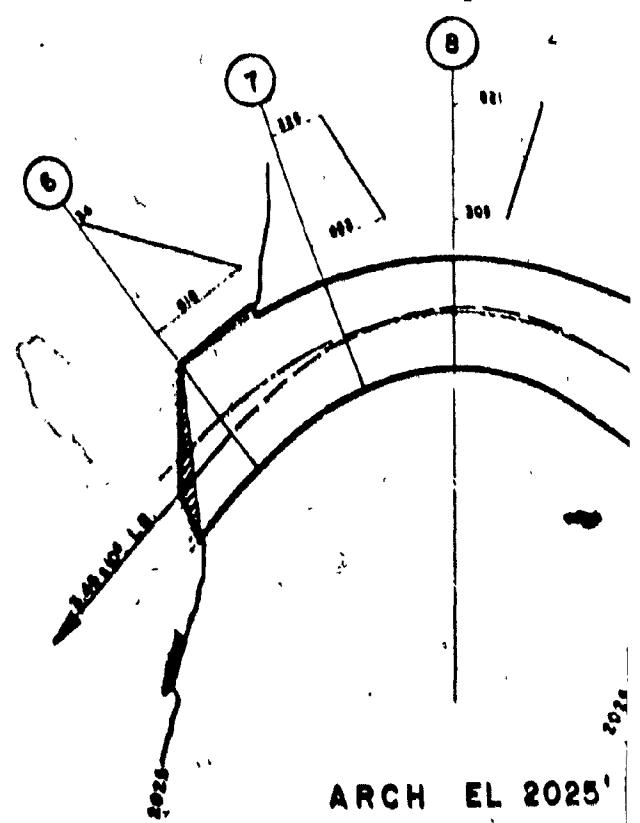
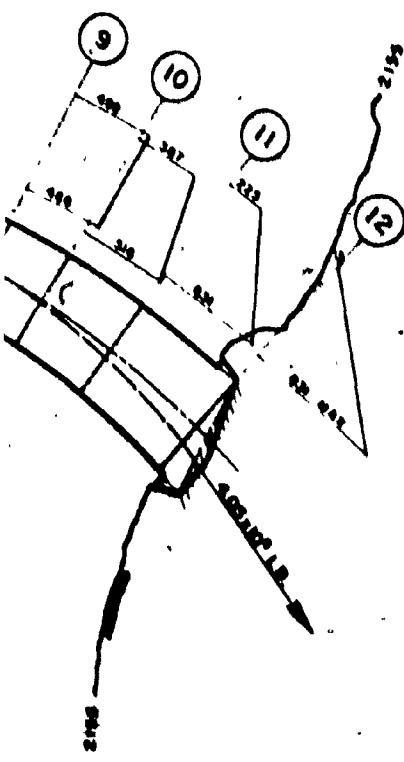
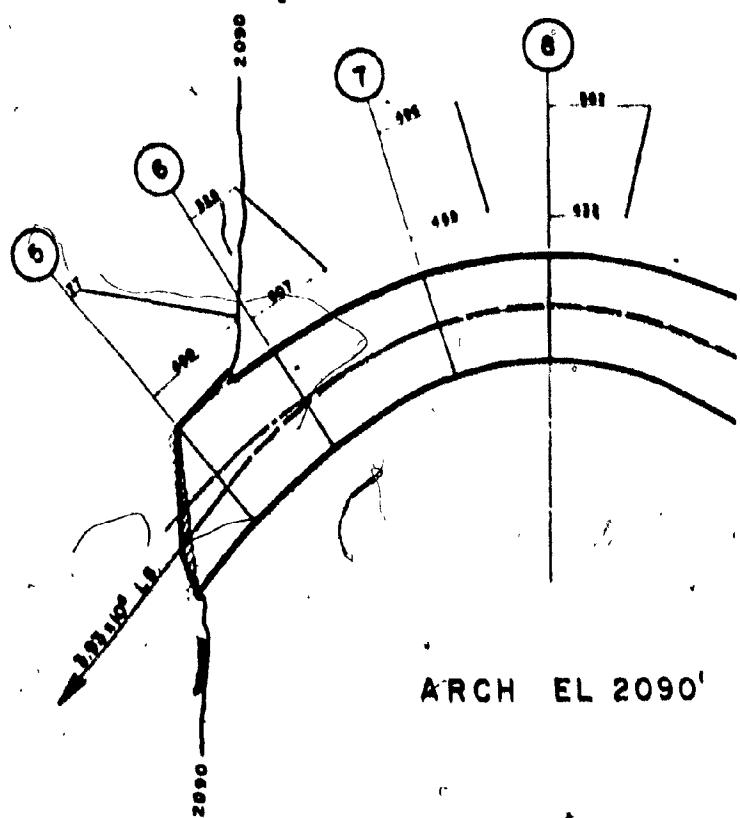
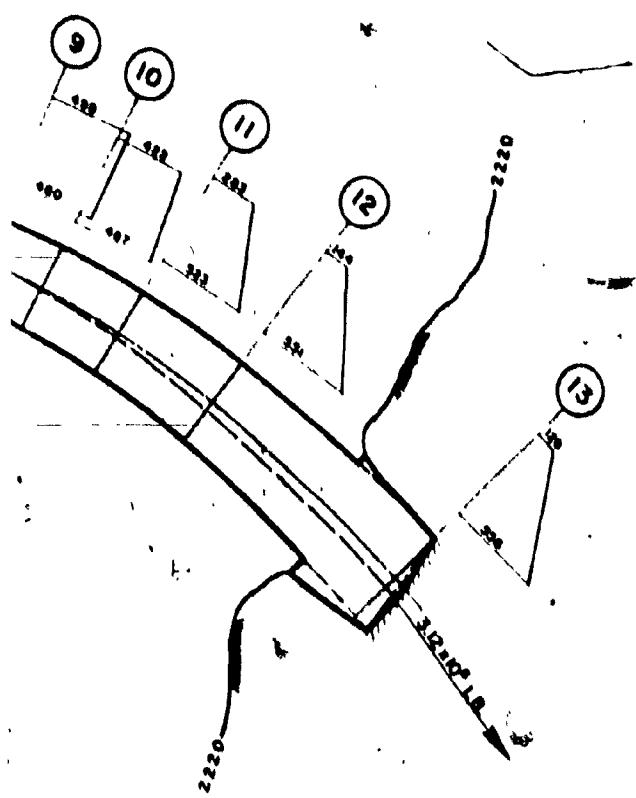
SCALE OF FEET

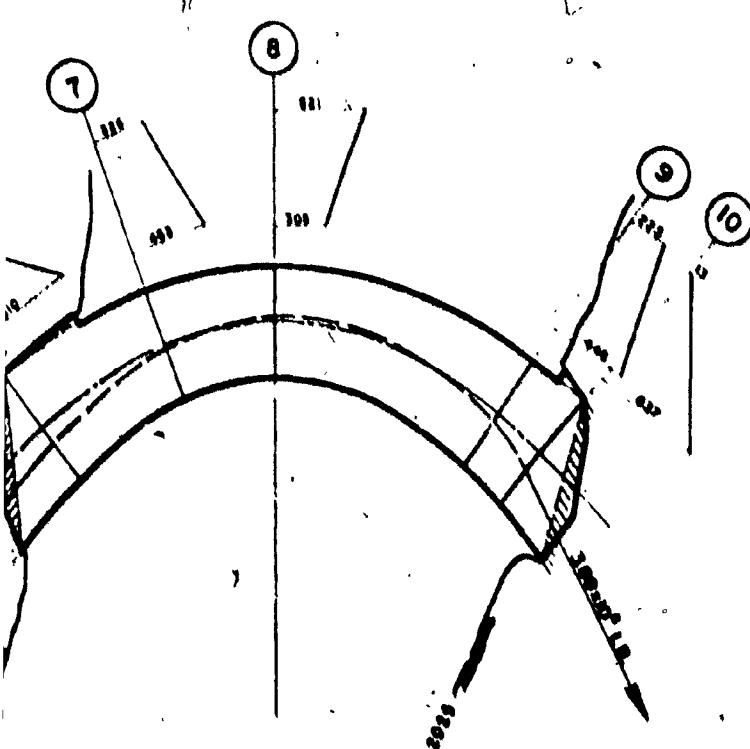
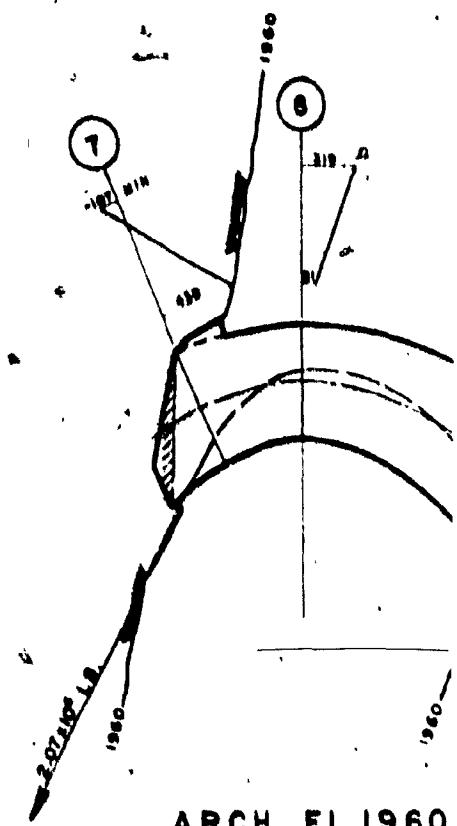
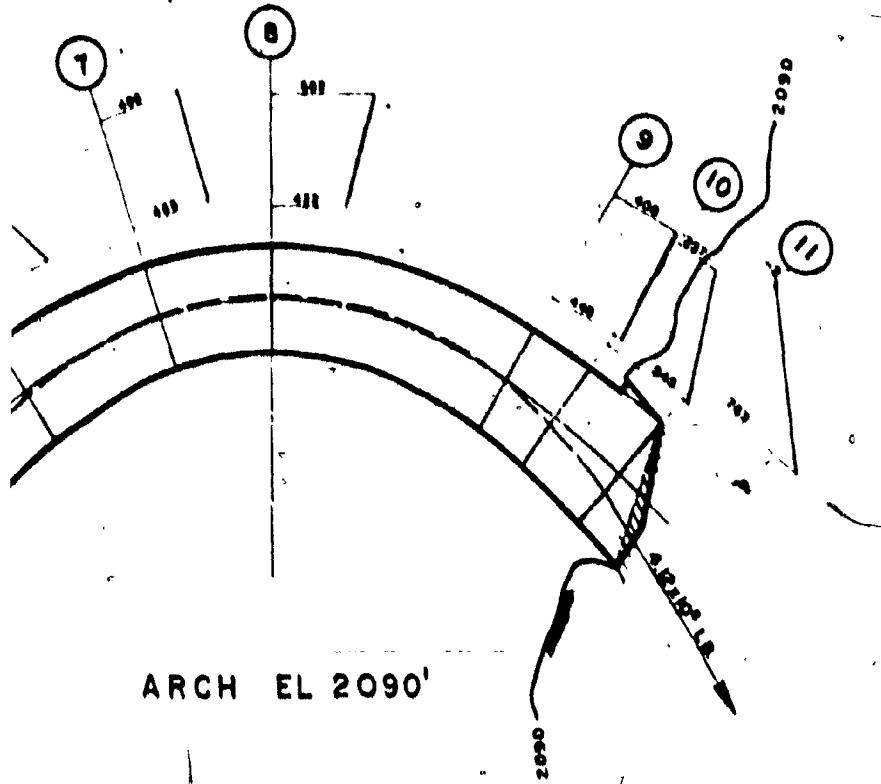
CANADA — INDIA	
IDIKKI HYDRO-ELECTRIC PROJECT	
COLOMBO PLAN	
EXTERNAL AID OFFICE CANADA - KERALA STATE ELECTRICITY BOARD INDIA	
IDIKKI ARCH DAM - DESIGN 181.10M	
NORMAL STRESSES & RESULTANT FORCES	
ARCS EL 2285' TO EL 2415'	
FULL ADJUSTMENT	
	SURVEYOR MENNIGER & CHENEVERT INC Montreal, Quebec
	PLATE NO. 8A

PLATE 6B Idikki Arch Dam - Arches El.1960
to El.2220



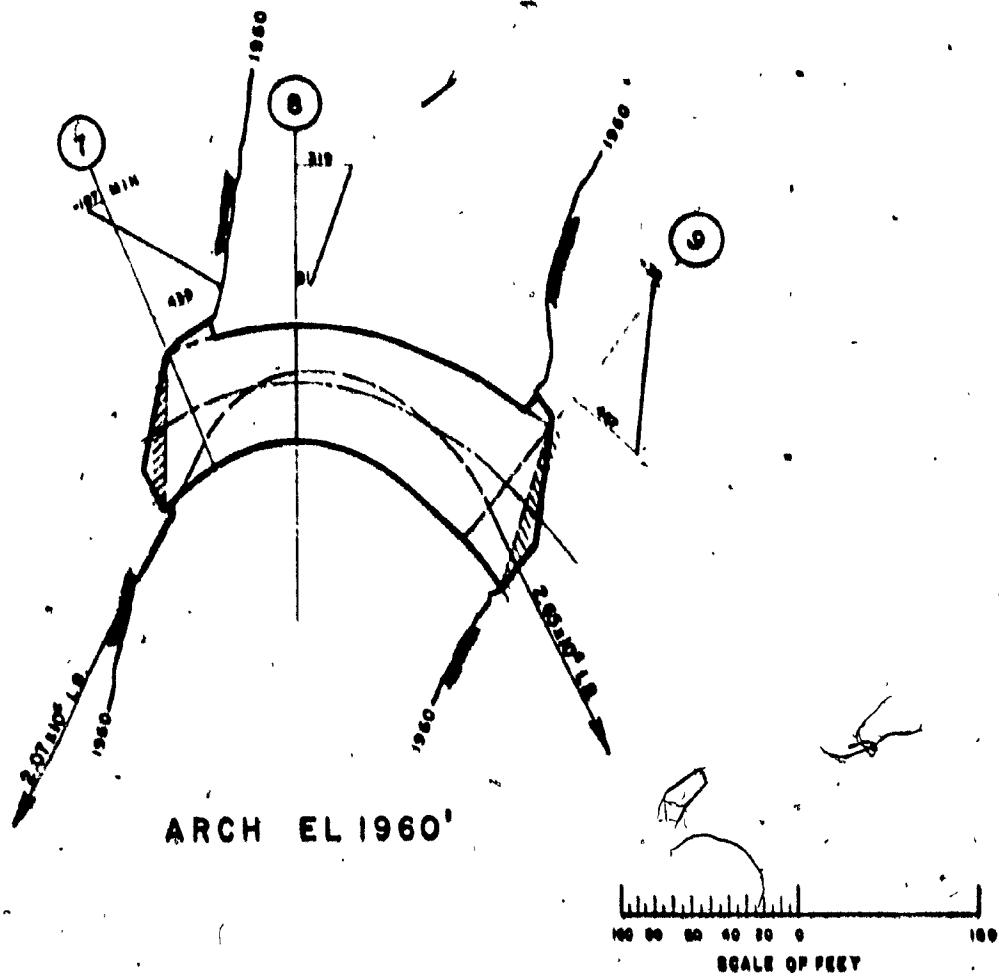
100





LEGEND

- (1) CANTILEVER NUMBER
- PLANE OF CENTERS
- ARCH CENTER LINE
- POSITION OF RESULTANT FORCE
- ROCK
- FOUNDATION FOR COMPUTATIONS
- THEORETICAL ARCH ABUTMENT
-  STRESS DIAGRAM
-  RESULTANT AT ABUTMENT

**LEGEND**

- (1) CANTILEVER NUMBER
- PLANE OF CENTER
- ARCH CENTER LINE
- POSITION OF RESULTANT FORCE
- ROCK
- FOUNDATION FOR COMPUTATIONS
- THEORETICAL ARCH ABUTMENT
- STRESS DIAGRAM
- RESULTANT AT ABUTMENT

LOADING CONDITION 3.5

RESERVOIR WATER SURFACE EL 2408 FT
 SILT LOAD UP TO EL 2100 FT
 EFFECTS OF TAILWATER AND UPLIFT ARE NOT
 CONSIDERED
 TEMPERATURE CHANGES IN CONCRETE AS SHOWN
 ON PLATE N°14

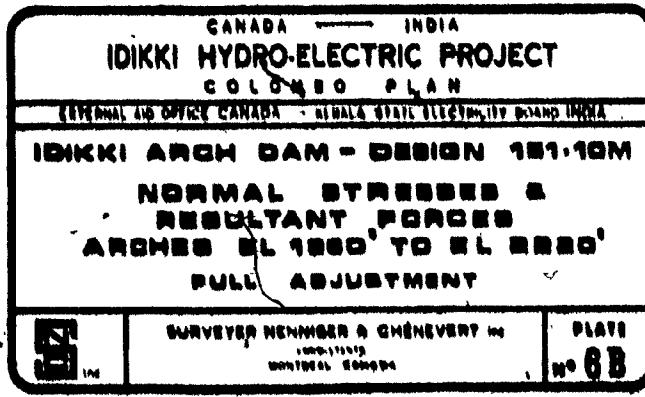
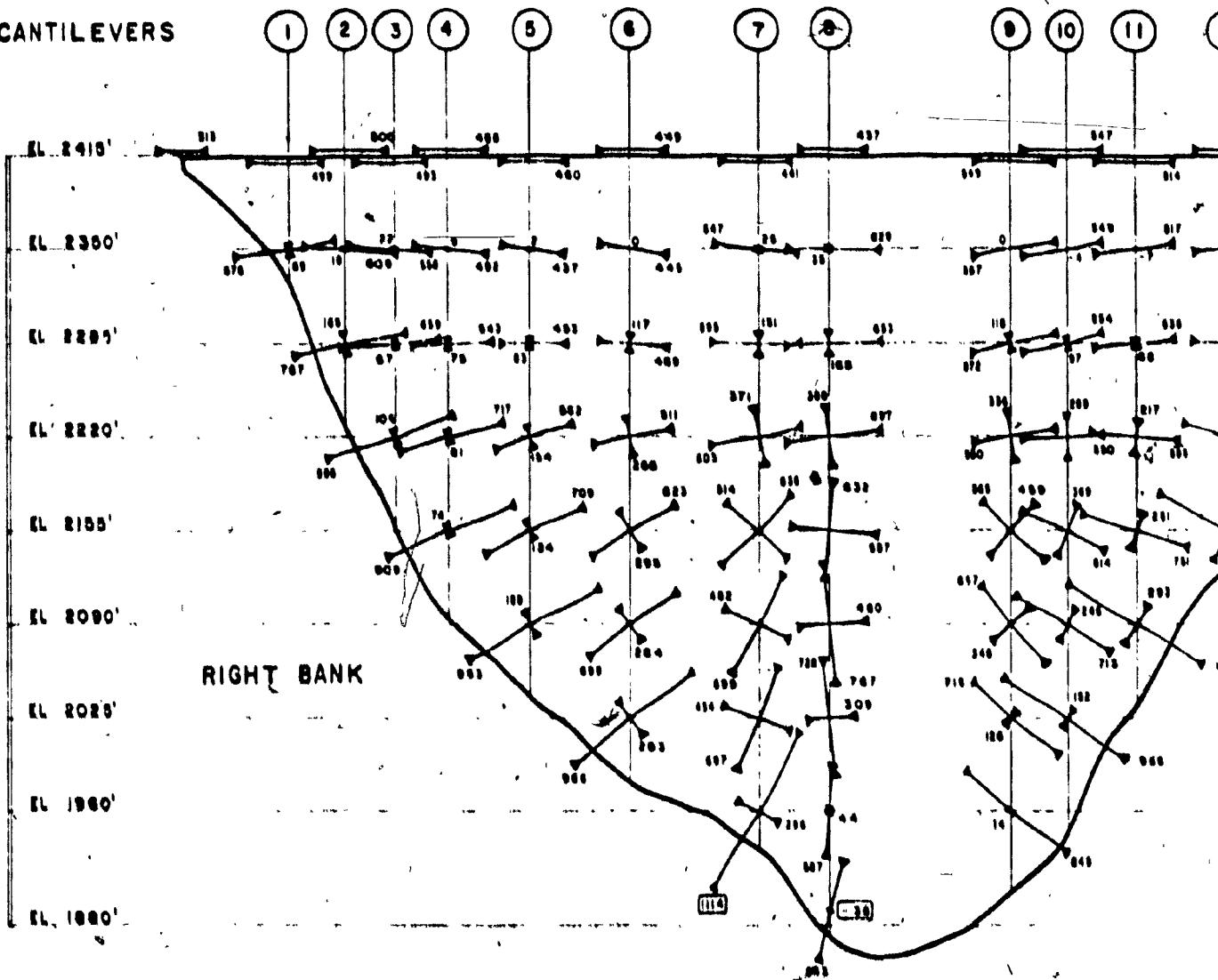


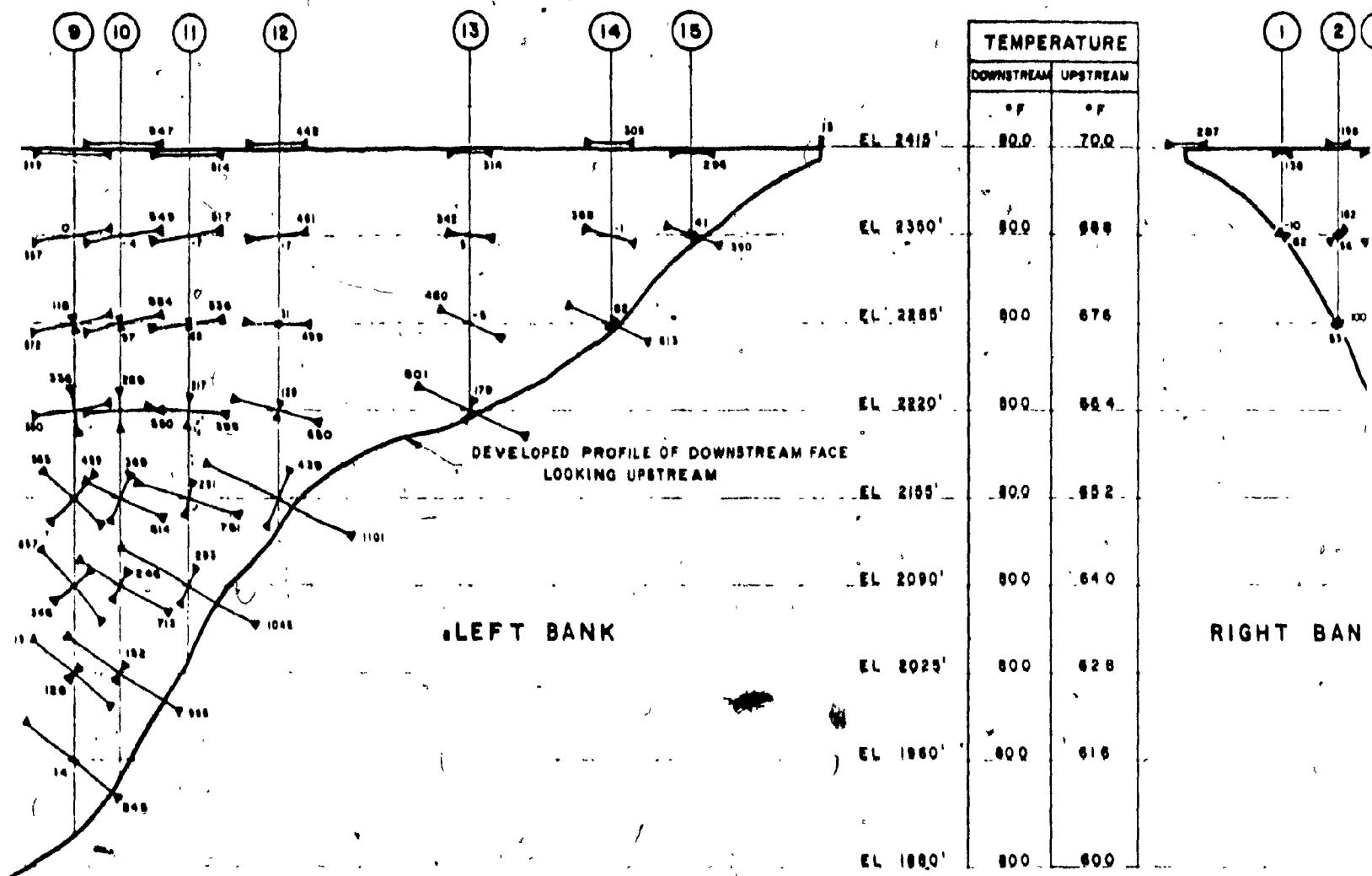
PLATE 7 Principal stresses in the dam -
Hydrostatic load, dead load,
temperature and 0.10 G. earthquake

CANTILEVERS



11 of 1

DOWNTREAM STRESSES



ESSES

LOADING CONDITION 3.5d

RESERVOIR WATER SURFACE EL 24030 FT

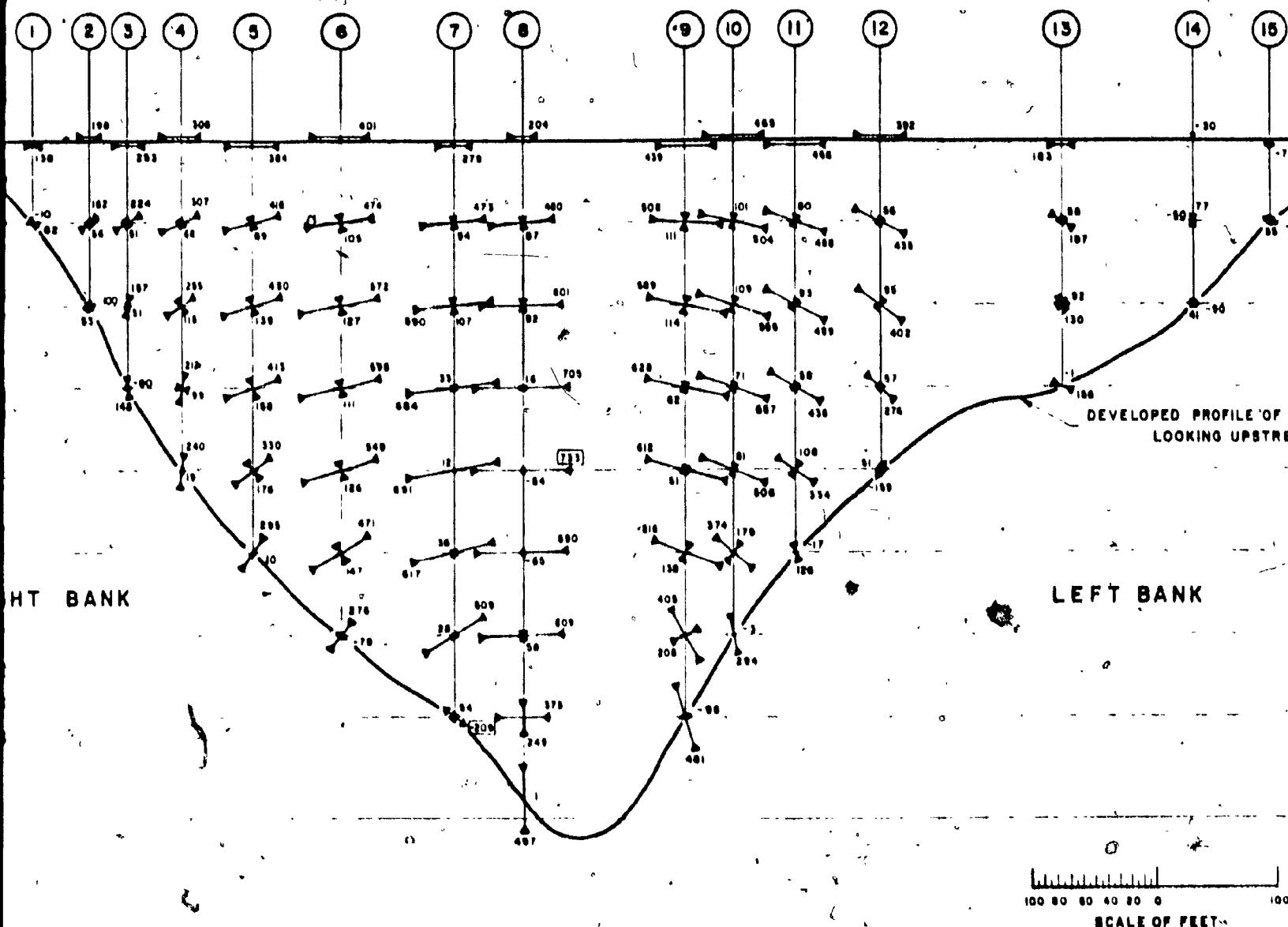
SILT LOAD UP TO EL 2100 FT

EFFECTS OF TAILWATER AND UPLIFT ARE NOT
CONSIDERED

EARTHQUAKE ASSUMPTIONS:

UPSTREAM-DOWNSTREAM ACCELERATION
IN THE DIRECTION OF THE LINE OF CENTERS
0.1 G. PERIOD OF VIBRATION - 1.0 SECOND

TEMPERATURE CHANGES IN CONCRETE AS INDICATED

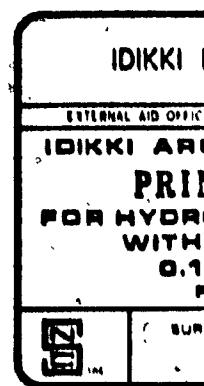


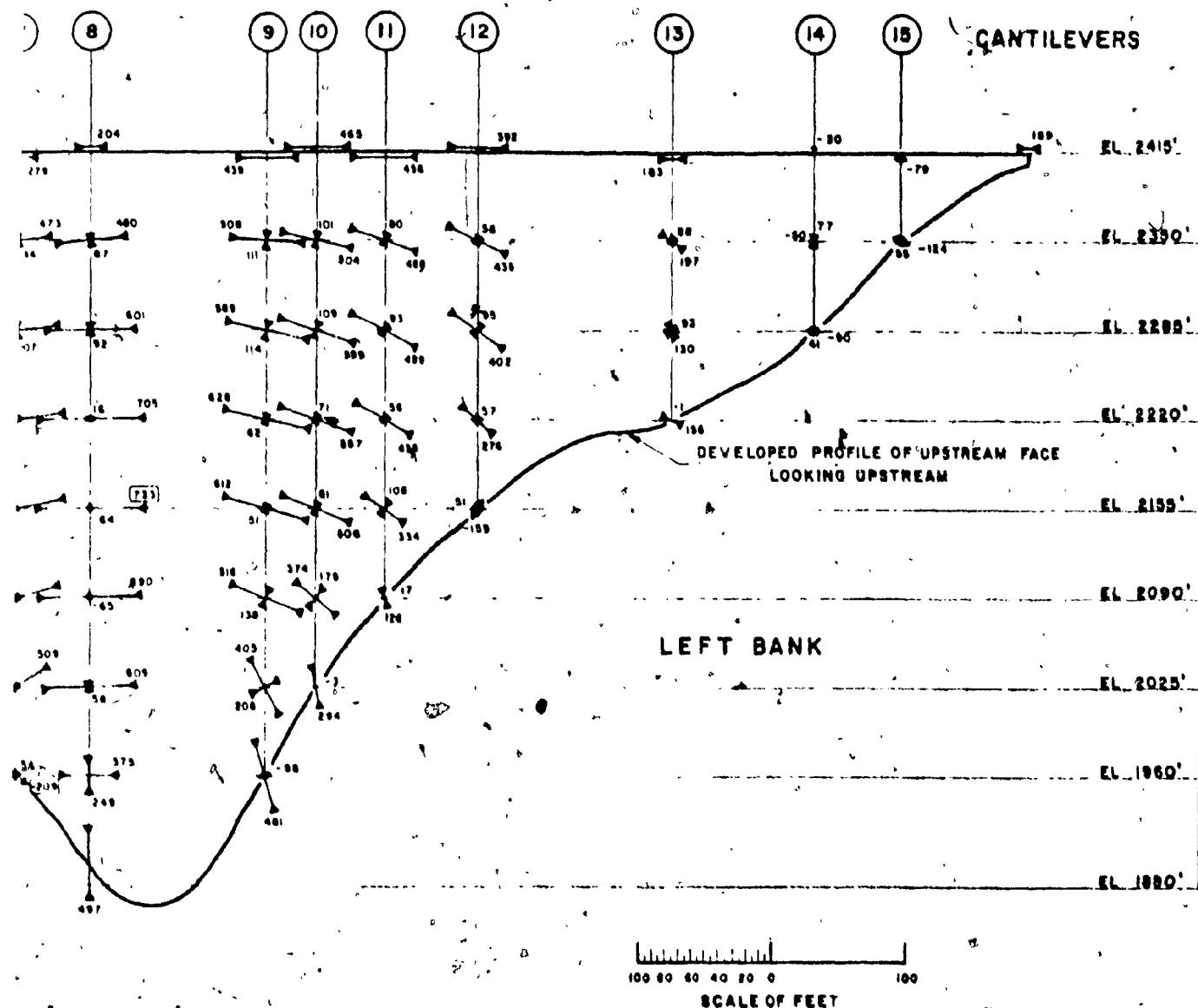
SCALE OF STRESSES

TENSION

COMPRESSION

301



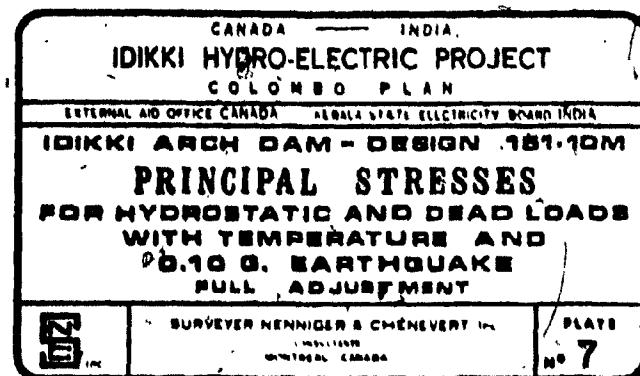


REAM STRESSES

CALE OF STRESSES

TENSION

COMPRESSION



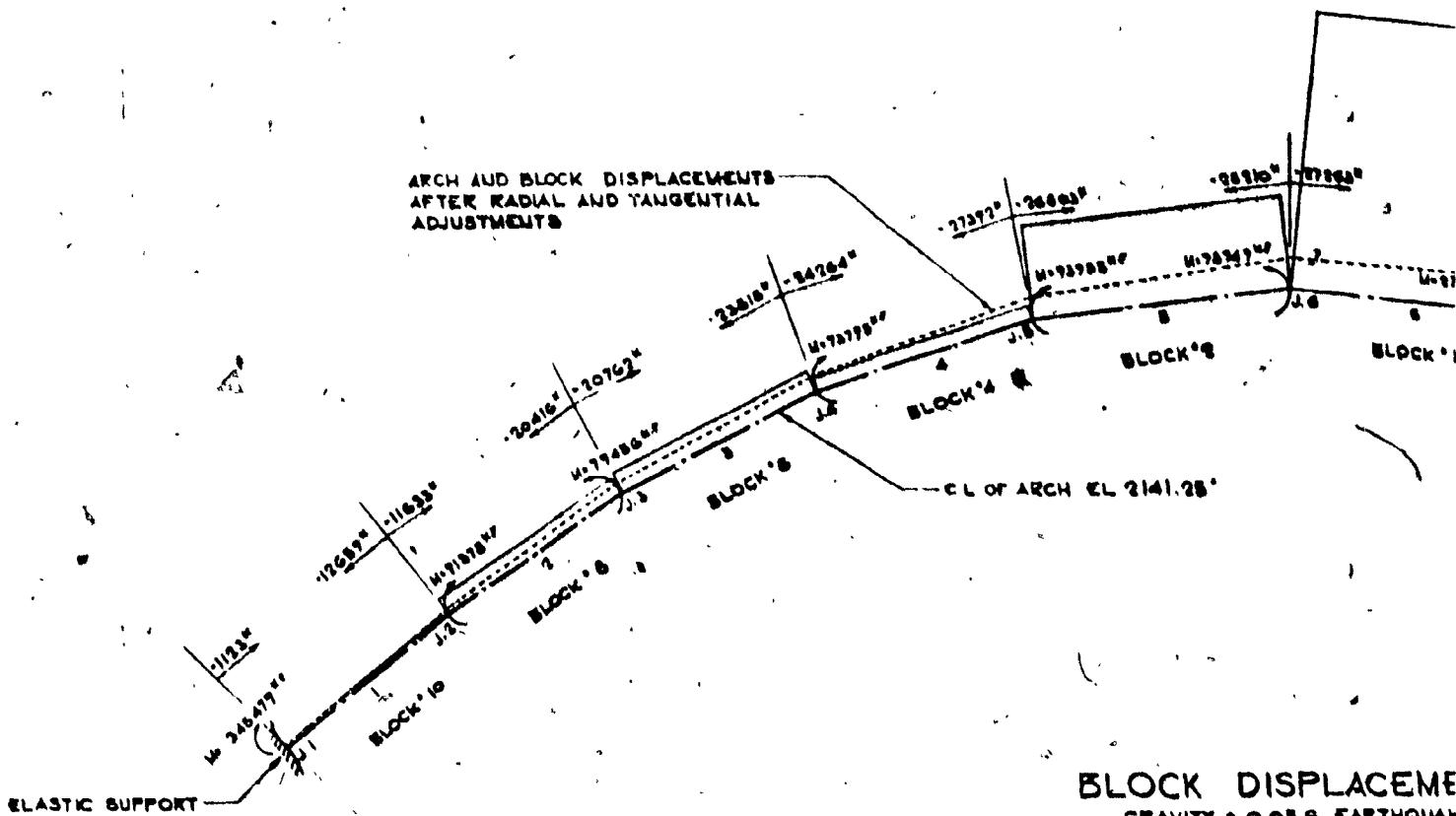
A-IV DRAWINGS

DRAWING 1 Idikki Dam - Seismic Bars,
forces and displacements at
elevation 2140'

0.00776 0.01467 0.01880 0.01937 0.01487
 5' 10" 5' 8" 5' 6" 5' 4" 5' 2"

ARCH TANGENTIAL DISPLAC
(IN INCHES)

ARCH AND BLOCK DISPLACEMENTS
AFTER RADIAL AND TANGENTIAL
ADJUSTMENTS



BLOCK DISPLACEMENT
GRAVITY = 0.08 G EARTHQUAKE

COLOURED

1 of 1

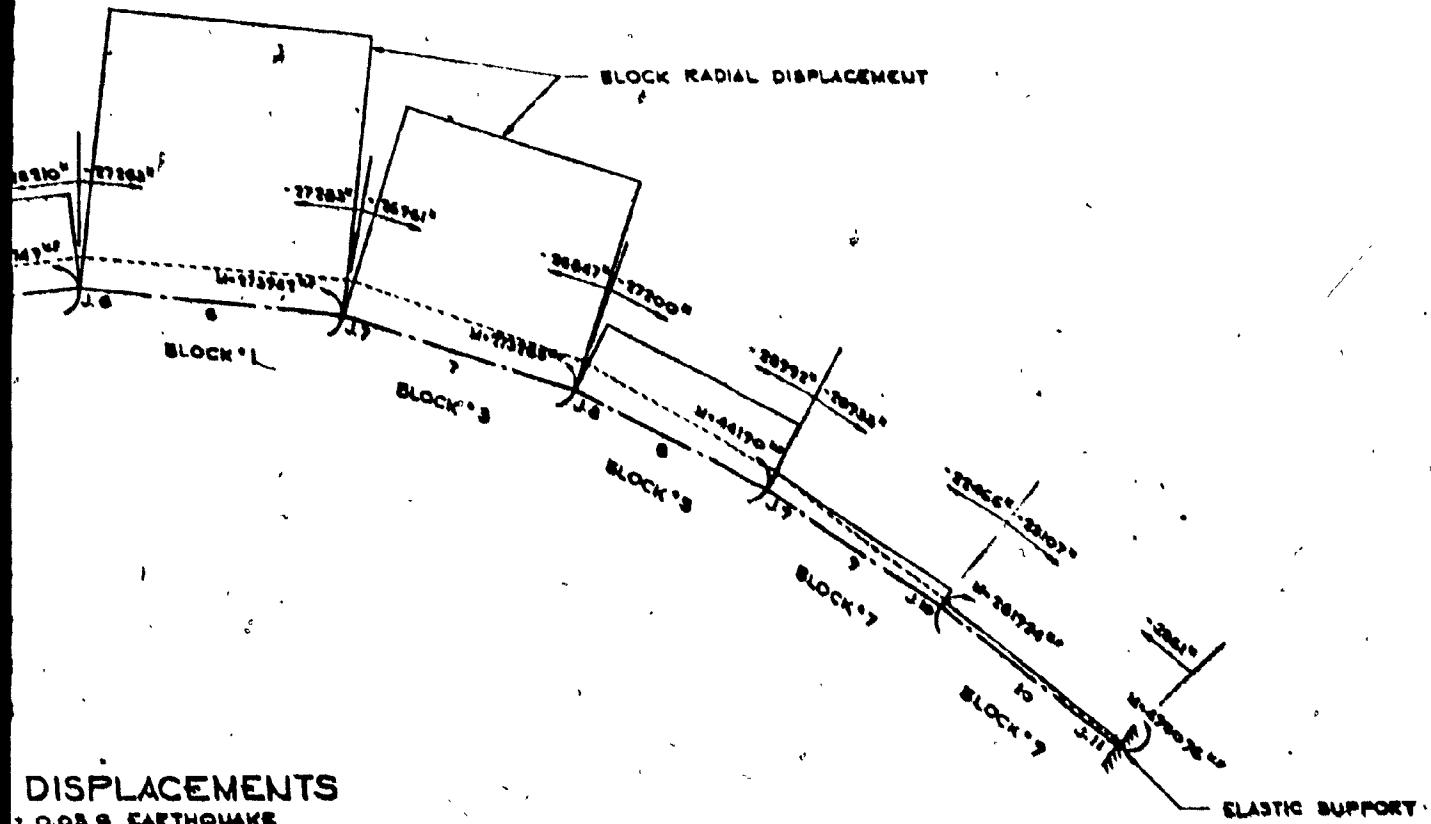
BLOCK	NUMBER	DISPLACEMENTS (IN INCHES)		UNIT LOAD
		RADIAL DUE TO EQ. 844H	TANG.	
10	1	0.00776	0.01467	0.0100
9	2	0.0977	0.3040	0.01787
8	3	0.1938	0.1161	0.00860
7	4	0.0783	0.3010	0.00000
6	5	0.0319	1.0168	0.83300
5	6	1.8182	1.8700	1.88400
4	7	1.4060	1.3910	1.18100
3	8	0.4933	0.3198	0.33400
2	9	0.1708	0.0684	0.07000
1	10	0.0342	0.01683	0.010700

BLOCK
10
9
8
7
6
5
4
3
2
1

1 UNIT LOAD = 10,000 KIPS
 2 ARCH THICKNESS 30 FT. (ASSUMED)
 3 EARTHQUAKE ACTION UPSTREAM
 TO DOWNSTREAM

0.00278 -0.01988 -0.03431 -0.02568 -0.01937
8' 1" 8' 3" 8' 5" 8' 7" 8' 7"

RADIAL DISPLACEMENTS (INCHES)



DISPLACEMENTS 0.05 G EARTHQUAKE.

BLOCK	NUMBER	ANCH LOAD (W.E.P.)			
		RADIAL		TANGENTIAL	
		TOTAL	UNIFORM	TOTAL	UNIFORM
10	1	-7682	-177.0	-11499	-348.2
9	2	18700	390.0	-9700	-184.0
6	3	3700	68.0	-3900	-68.0
4	4	-1960	-39.4	-3100	-60.0
2	5	4800	79.8	-1400	-24.0
1	6	9400	189.0	-17	-0.32
3	7	8700	167.0	10	1.16
9	8	7000	160.0	1000	34.0
7	9	1800	38.0	4000	72.0
7	10	-1900	-41.7	20000	440.0

FOUNDATION CONSTANTS

JOINT I
KFX $1.6 \cdot 10^6$ KNU/mm
KFY $1.6 \cdot 10^6$ KNU/mm
KHZ $8.16 \cdot 10^6$ KNU/mm
JOINT II
KFX $1.91 \cdot 10^6$ KNU/mm
KFY $1.8 \cdot 10^6$ KNU/mm
KHZ $9.76 \cdot 10^6$ KNU/mm

COLOURED



SURVEYOR MANAGER & CHENEVERT INC.
Proprietary
Manufactured in Canada

H. P. PHILIPPOE
P. SARGA
P. SARSON
R. C. STANNETT

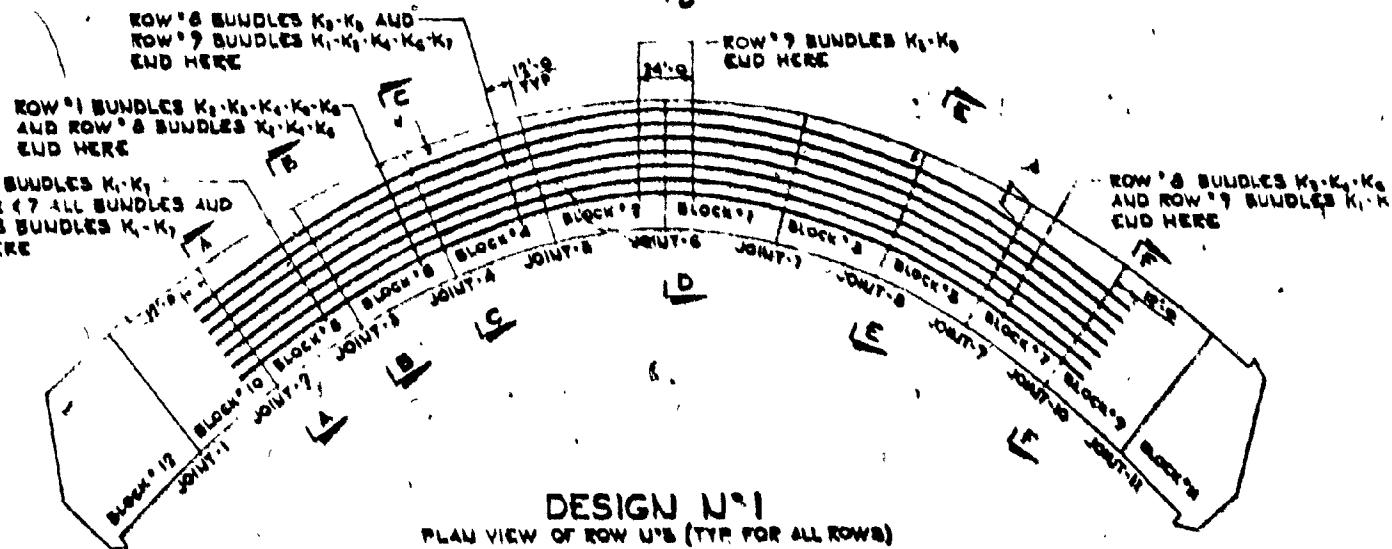
E.I.B.

1
1
1

DRAWING 2 Idikki Dam - Seismic Bars,
reinforcement layout

COLOURED

10



E

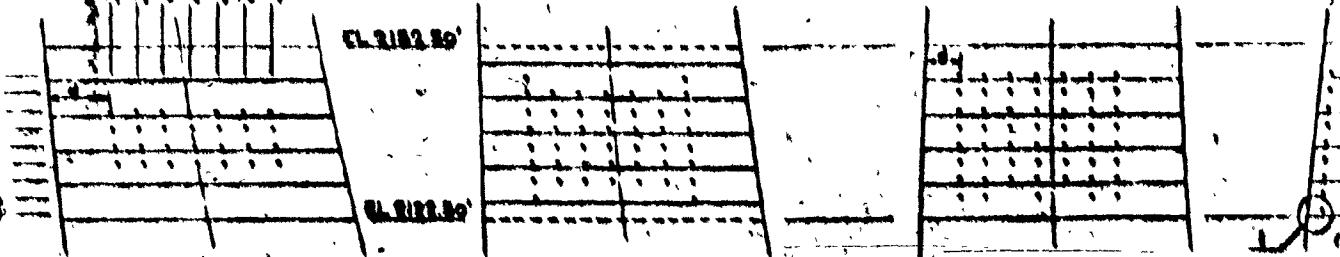
D

C

B

A

ROW N°



SE

PLAN VIEW TYPICAL FOR ROW N°8
ROWS N°1, 2, 3, 4, 5, 6, 7, 8, 9
AND 10 SAME AS IN SECTION A-A
ROWS N°11, 12, 13, 14, 15, 16, 17, 18, 19
AND 20 SAME AS IN SECTION B-B
ROWS N°21, 22, 23, 24, 25, 26, 27, 28, 29
AND 30 SAME AS IN SECTION C-C

COLOURED

292

119

9 BUNDLES K₁, K₂, K₃
DW 9 BUNDLES K₁, K₂, K₃, K₄, K₅, K₆
ERE

ROW N°	DESIGN W ¹						TOTAL LENGTH	WEIGHT IN TONS
	K ₁	K ₂	K ₃	K ₄	K ₅	K ₆		
1	411	383	347	341	374	379	376	38.511
2	411	408	398	392	384	378	376	40.000
3	400	433	443	438	430	426	416	44.000
4	460	483	448	438	410	408	416	44.000
5	460	483	448	438	420	418	416	44.000
6	460	483	448	438	420	418	416	44.000
7	411	408	398	392	382	378	376	40.000
8	411	304	378	372	364	370	370	38.511
9	287	283	24	243	24	372	370	38.511
							TOTAL	388.511

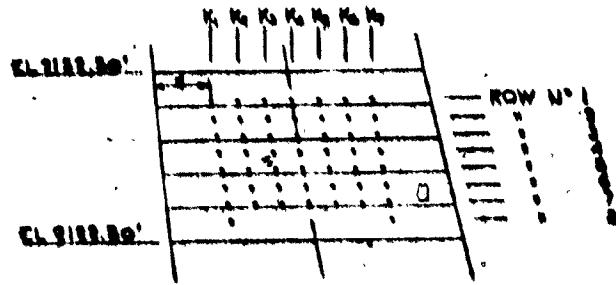
ROW N°	DESIGN W ²						TOTAL LENGTH	WEIGHT IN TONS
	K ₁	K ₂	K ₃	K ₄	K ₅	K ₆		
1	411	393	373	373	373	373	373	37.511
2	411	416	373	373	373	373	373	37.511
3	411	416	373	373	373	373	373	37.511
4	411	416	373	373	373	373	373	37.511
5	411	416	373	373	373	373	373	37.511
6	411	416	373	373	373	373	373	37.511
7	411	416	373	373	373	373	373	37.511
8	411	416	373	373	373	373	373	37.511
9	411	416	373	373	373	373	373	37.511
							TOTAL	343.511

ROW N°	VALUE OF V ¹						TOTAL LENGTH	WEIGHT IN TONS
	K ₁	K ₂	K ₃	K ₄	K ₅	K ₆		
1	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
2	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
3	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
4	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
5	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
6	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
7	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
8	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
9	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
							TOTAL	9.000

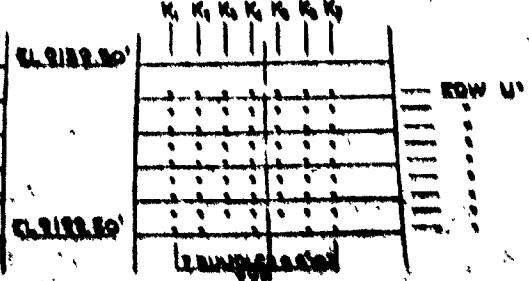
VARIABLE
BUNDLE OF 6-36 mm DIA BARS
PROPERLY TIED TOGETHER

LIFT JOINT

DETAIL 1 (TYP)
N° 140

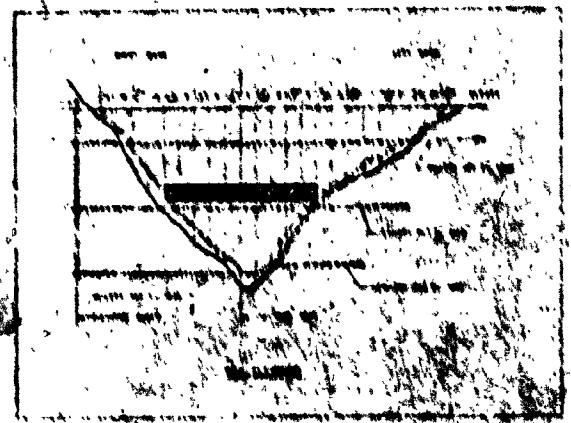


SECTION F-F
11' 8 1/2"



SECTION D-D
11' 8 1/2"

SECTION E-E
TYP FOR JOINTS 2, 7, 8 & 9



CANADA IDIKKI HYDRO-ELECTRIC PROJECT INDIA
1962-1974
IDIKKI DAM - TURBINE BASE REINFORCEMENT LAYOUT
2

SURVEY NUMBER & SURVEYOR		SHEET NO.
NAME	DESIGNATION	
		1
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