Paper No. 338

Stress and Stability Analysis of Warsak Dam

By

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Synopsis :

This paper deals with the stability and stress analysis of the main dam for various conditions of loading and the assumptions made. The height of the dam places it in the medium height class and justifies careful consideration of stability and stresses.

Three sections are considered.

(1) The main dam spillway section.

(2) The bulkhead section.

(3) The north bulkhead-stairwell section.

A summary of the results for all the sections and detailed calculations for one typical section of the main dam spillway along-with the design criteria are given at the end (APP : A) to serve as a guide to the reader.

A comparison of the stresses and assumptions made in the design of Warsak Dam and some very prominent dams in U.S.A. is also given in a tabular form, (TABLE A).

2. *The Project.*

The 235′ high Warsak Dam is a major part of the Multipurpose Warsak Hydro-Electric Project located on Kabul River about 18 miles from Peshawar in West Pakistan. The dam is a mass concrete gravity structure with a central overflow section consisting of 9 bays of 40 feet each and side bulkhead sections. Plate 1. shows the plan and the sections of the dam.

The power station is located above ground on the right bank just below the dam. The main object of the project is to produce 2,40,000 KW of the much needed power and also to bring under irrigation nearly 1,20,000 acres of virgin land.

There will be six generating sets of 40,000 KW each, of which only four will be installed in the first phase scheduled for completion by middle of 1960.

Two canals, one taking off on the right bank and the other on the left bank through two tunnels, will irrigate about 1,20,000 acres.

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The right bank tunnel is nearly 3½ miles long and will have a 10′ finished diameter after lining with concrete, and the left bank tunnel will be 8600 feet long with a section of 5′ × 6′. The total discharge for irrigation will be 500 cusecs in Kharif and 350 cusecs in Rabi.

3. *The Design* :

(1) *Forces acting on the dam* :

The first consideration in designing a dam is the determination of the nature. of forces acting on the structure the following forces are considered as acting on the dam :—

(i) Water pressure, external water pressure & internal water pressure or uplift.

(ii) Earth pressure.

(iii) Earthquake forces.

(iv) Weight of the dam.

(v) Wind pressure.

(vi) Wave pressure and heights.

(vii) Ice pressnre.

The wind pressure, ice pressure and wave pressure are considered negligible for Warsak Dam.

(2) *General requirements for stability of gravity dams*

There are two direct ways in which a gravity dam may fail :

(i) By sliding (a) on a horizontal or nearly horizontal joint above the foundation, (b) on the foundation, or (c) on a horizontal or nearly horizontal Seam in the foundation rock.

(ii) By overturning on a horizontal joint (a) within the dam, (b) at the base, or (c) at a plane below the base.

The direct cause of the sliding is the presence of horizontal forces greater than the combined shearing resistance of the joint or base and the static friction induced by the vertical forces.

The direct cause of pure overturning, not preceded by some other type of failure, if tension is ignored, is the presence of horizontal forces great enough in comparison with the vertical forces to cause the resultant of all forces acting on the dam above any horizontal plane, including uplift, to pass outside the limits of the dam.

As the resultant approaches the face, the compressive stresses increase reapidly, hence overturning would be preceded and accelerated by a compression failure. In fact, a dam with the resultant well inside the joint may overturn if the toe of the dam fails by crushing or other causes so as to

reduce the effective lenght of the joint or base sufficiently to cause the resultant to pass outside.

A dam may start to overturn but finally fail by sliding. If the resultant passes appreciably outside the middle third (or kern, if the base is irregular form), a horizontal tension crack may occur, which reduces the shearing strength of the joint or base. Also, the admission of headwater pressure to the fissure increases the uplift, reducing the net reaction, and the frictional resistance to horizontal motion. As a result, sliding may occur.

The area of the foundation being of great extent, stability below the base is usually assured. Failures have resulted, however, from erosion of the foundation by water spilling over the crest. In this event, the dam may become undermined and eventually overturn, or the erosion may expose a horizontal seam filled with clay or other practically frictionless material, in which event, the supporting downstream ledge having been removed, the dam and that portion of the foundation above the exposed seam may slide.

The gradual disintegration of the dam by weathering and other causes will, of course, finally result in its failure. Modern masonry dams are practically indestructible if well built and composed of proper ingredient. However, if proper precautions are not taken in the choice of materials and methods of construction, rapid deterioration may occur.

Rules governing the Stability of Gravity Dams :

(**Rule 1**) *Governing the Location of the Resultant* :

> "Tension shall not exist in any joint of the dam, under any conditions of loading without earthquake. For dams with rectangular joints, this requirement is met if the resultant of all forces including uplift, acting on the dam above any horizontal joint, for full or empty reservoir, intersects the joint within the middle third."

(**Rule 2a**) *Resistance to Sliding Shear Neglected :*

> "The tangent of o, the angle between the vertical and the resultant of all forces, including uplift, acting on the dam above any horizontal plane, shall be less than the allowable coefficient of friction at that plane."

$$\text{i.e. } \frac{\Sigma (P)}{\Sigma (W)} = \tan \theta = \angle f$$

where f is the coefficient of friction of the materials on each side of the joint or at the base, as indicated by tests on well-dressed specimens of the same materials."

Values of f for masonry on masonry and masonry on good rock foundations have been assumed variously between 0.6 and 0.75. In general, and for careful work, a value of 0.75 is not excessive. A value of f = C = 0.65 has been adopted in this design.

(Rule 2b) *Resistance to Sliding, Shear Included* :

> "The total frictional resistance to sliding on any joint, plus the ultimate shearing strength of the joint, must exceed the total horizontal force above the joint for all conditions of loading, by a safe margin."

This relationship may be stated algebraically thus ;

$$\Sigma P = \frac{C. \Sigma W + B_1 R_s}{f_s}$$

$$\text{of}_s = \frac{\Sigma W + C + B1\ Rs}{\Sigma P}$$

Where (B1) is the x-sectional area of the base, R_s is the average unit shear resistance and fs is the shear friction factor of safety.

Shear strength may be determined by tests if published data for the material involved is not available. It is necessary to know the shear strength of both the foundation and the masonry, the small value being. used Houk states that,

"The principal uncertainty involved in evaluating the shear friction factor of safety is the determination of the average shearing strength of the material. Values of shearing stress used in calculating (f_s) for dam built by the Bureau of Reclamation have varied from about 300 psi to 700 psi, depending upon the characteristics of concrete and rock specimens as determined by laboratory tests." A value of R_s = 400 psi has been adopted in this design.

(Rule 3) *Governing Compress ive Stress* :

> "The unit inclined compressive stresses in the dam and the foundation shall not exceed certain prescribed values, (as laid down on page 10)."

(Rule 4) *Governing Internal Tension.*

> "The dam shall be designed and contsructed in such a manner as to avoid or adequately provide for tension on planes, inclined interior vertical, or horizontal."

The application of this rule is complicated and cannot readily be carried along in the step-by step application of the other rules for the design of a dam. Ordinarily, it is not a determining factor in design. Hence, the normal procedure is to complete the design without regard to Rule 4 and then to test for internal tension.

Rule—5. *Governing the Margin of safety :—*

"All assumptions of forces acting on the dam shall be unquestionably on the safe side, all unit stresses adopted in design shall provide an ample margin against rupture, and the adopted safe sliding factor or shear-friction safety factor shall be conservative."

The term of "factor of safety" as used in structural design is directly applicable to hollow dams and arch dams through the stress equations, but the term is less directly applicable to gravity dams. It is sometimes said that a gravity dam, with the resultant, reservoir full, at the downstream middle third point has a factor of safety of two against overturning. This statement comes from the fact that doubling the active overturning moment in such a dam of triangular from with vertical upstream face will move the resultant out to the downstream face. Actually, failure by crushing would occur before the resultant reached the face. On the other hand, the possibility of doubling the moment is non-existant, hence a numerical factor of safety against overturning is meaningless. The same is true for sliding. The shear-friction safety factor and the safety factor against a compression failure have the ordinary structural significance.

Rule—6. *Governing Details of Design and methods of Construction :*

"All details shall support and conform to the assumptions used in the design ; the masonry shall be of a quality suited to the working stresses adopted, and shall be practically watertight and durable ; protection against overflowing water shall be ample."

Comparison of Stresses and Assumptions, existing dams.

In Table 1 are given the pertinent assumptions used in the design and calculated stresses in a number of prominent dams, as well as those for the examples of design included hereinafter.

The descriptive notes following and referred to in the table contain essential information required for a correct interpretation of the data. Some of these notes were required by those furnishing the data.

3. Assumptions made :

In analysing the structure, the following design assumptions are made.

(1)—The transverse contraction joints in the dam are neither keyed nor grouted.

(2)—All loads are carried by gravity action or weight of vertical parallel side cantilevers which receive no support from adjacent elements.

(3)—Unit vertical pressures or normal stresses and shear stresses on horizontal planes are assumed to vary linearly from upstream face to downstream face.

(4)—Unit horizontal pressures or normal stresses on vertical planes and vertical shear stresses are assumed to vary linearly between sections.

Theoretically, the variation in shear stresses on both horizontal and vertical planes is of parabolic from, but the approximation made in assuming a straight line variation is so small as not to affect the calculated stresses appreciably. The only case where this assumption is wrong is when both the upstream and the downstream faces of the dam are vertical. As this case only occurs near the top of the bulkhead section, it does not influence the stresses computed here.

The sections were analysed by considering horizontal sections at 20′ intervals. At each level the forces and moments are tabulated together with.

(1)—The factor of safety against overturning, which is the total resisting moment divided by the total overturning moment.

(2)—Tan θ the sliding coefficient being the total horizontal force divided by the total vertical force at any section.

(3)—The factor of safety against sliding, which is the sum of the frictional resistance and the shear resistance divided by the sum of the horizontal forces Σ P. The frictional resistance is the sum of the vertical forces ΣW multiplied by the coefficient of friction, C, taken as O. 65. The total shear resistance is the cross-sectional area multiplied by the considered unit shear resistance taken to be 400 psi.

The vertical stresses given in the tables include an allowance for uplifts, to show the greatest possible tensions occuring.

The principal stresses given on the tables and those shown on the plates are for the no-uplist condition to give maximum values of compression and shear.

4. *Conditions of Loading :*

(*i*) *Water Levels :* The regulated headwater is at EL. 1269 feet with the tailwater at EL. 1128 feet.

In times of maximum flood the headwater level is taken as EL. 1285 and the tailwater as the that of 1167′. For the spillway section under these conditions, the hydraulic jump reduces the tailwater elevation to 1128′. The reservoir empty condition is also considered.

(ii) *Uplift*: Water pressure acts not only at the exposed faces but also within the concrete and at the base. Its effect is important as it causes a reduction of the stabilising forces.

No means are available to permit an exact evaluation of uplift.

However, an upstream grout curtain such as proposed for Warsak and aduquate drainage would help to reduce it to a considerable extent.

Tests on existing dams with a grout curtain and drainage indicate values not exceeding two-thirds of the full hydrostatic load. In some instances it has been as low as one-half or even less, considering the nature of rock and other conditions at Warsak, it has been assumed to act over two-thirds of the area of the cection and varies linearly between the full headwater pressure at the heel and the full tailwater pressure at the toe.

(iii) *Silt Pressure*: It is assumed that silt will accumulate to elevation 1230 and will give an additional pressure which will depend on its density and angle of internal friction.

Values used are :—

Dry.wt. of silt = 90 lbs/ cuft

Specific gravity = 2.7

$\therefore$ Bourant WT. (W_s) $= 90 \times \frac{1.7}{2 \cdot 7}$

of (W_s) $= 56 \cdot 5$ lbs/cuft.

Angle of internal friction (ϕ) $= 33^0$

Then

$$\text{Silt pressure} = \frac{W_{sh}{}^2}{2} \times \frac{(1-\sin)}{(1+\sin)}$$

$$= 8.25h^2 \text{ lbs/ft width.}$$

and it is assumed that it will act at h/2.5 from the base.

(1) Normally the centroid of silt pressure and earth pressure is assumed to act at a distance of 1/3 of the height of the fill above the considered base. This results from the assumption that the pressure increases linearly with the depth. However, experience has shown that the linear distribution does not apply in some cases as arch action between the particles may occur which results in a different pressure distribution. On account of this variation a more conservative location of the centroid of the silt pressure has been used in the stability and stress calculations taking a value of 0.4h rather than 0.33h as would be for a linear pressure distribution.

(iv) *Earthquake* : Warsak Dam is located in a Seismically active region therefore safety demands full consideration of the earthquake forces in the design analysis. Thc earthquake forces are of course combined with the other forces acting on the dam. Because of their short duration and infrequent occurrence, Some concession in the factor of safety is permissible for the conditions of maximum earthquake and most adverse combination of other conditions.

(*a*) *Earthquake force on the body of the dam* : This force is calculated from the equation for the force required to accelerate a given mass ;

$$P_e = Mxa = \frac{W}{g} \times \alpha g = \alpha W$$

where P_e =Horizontal Earthquake force.

M=Mass of the dam or any portion under consideration.
a =earthquake acceleration

$$\alpha = \frac{a}{g} = \frac{\text{Earthquake acceleration}}{\text{Acceleratioa due to gravity}}$$

The force P_e acts through the centre of gravity of the blocks in opposite direction to the earthquake acceleration. For Warsak a Value of $\alpha = 0{\cdot}15$ has been assumed, having due regard to the saismic record of the area.

(*b*) *Increased water pressure :* The inertia of water in the reservoir also produces a force on the dam, which is determined by the following education developed by Von Karman, (Ref. Davis Hydraulics).

P	$= 0.555\ \alpha\ Wh^2$
P	= additional unit water pressure caused by earthquake at depth h.
α	$= 0.15$
W	= 62.5 lbs/cuft
P	$= 0.555 \times 0.15 \times 62.5\ h^2$
	$= 5.25h^2$ lbs/ft width.

This force is to act at $\left(\frac{4}{3\pi} h = 0.424h\right)$ from the bottom of the section.

Earthquake movement may take place in any direction. For gravity dam reservoir full, most unfavourable direction is upstream (normal of the axis). The corresponding force acts downstream. For reservoiar empty, a downstream acceleration is more unfavourable. A vertical acceleration changes the weight of the masonry and the water in the same ratio Considering these elements alone, the resultant is not displaced from the position it would occupy if there were no earthquake. However, the stresses are changed It the acceleration is upward, the stress is equal to the no-earthquake stress multiplied by $(1 + \alpha)$ which is generally less than the stress for an equal horizontal acceleration. If the acceleration is downward, the multiplier is $(1-\alpha)$. For a direction of acceleration intermediate between horizontal and vertical, the situtation is more complex. For small deviations from the horizontal, the maximum stress may be slightly greater than for a horizontal

acceleration of equal value, but the difference is smaller than the uncertainties in the value of α : hence, deviation from the horizontal may be ignored for dams with straight vertical faces.

Uplift is usually assumed to be unaffected by earthquake.

5. *Combination of Loads*:

Case A—Headwater at elevation 1269 feet.
Tailwater at Elevation 1128 feet.
No silt
No earthquake
With uplift.

Case B—Headwater at elevation 1269 feet.
Tailwater at elevation 1128 feet.
Silt level at elevation 1230 feet.
No earthquake
With uplift.

Case C—Headwater at elevation 1269 feet.
No tailwater
Silt level at elevation 1230 feet.
No earthquake
With uplift.

Case D—Headwater at elevation 1285 feet.
Tailwater at elevation 1167 feet.
Due to hydraulic jump this is considered at elevation 1128 feet for stability calculations of the spillway sections.
Silt level at elevation 1230 feet.
No earthquake
With upilft.

Case E—Reservoir empty
No tailwater
No silt
No earthquake
With uplift,

All cases were also analysed with earthquake, this condition being given the sufficx 1″.

The results of analysing maximum sections with bases at E1. 1050 for spillway, E1.1130, for bulkhead and E1. 1132 for stairwell section, for these canditions of loading are given in tables 6 toS
and shown graphically on Plates 3 to 8

6. Consideration of Results :

Stability: The lowest value of the factor of safety against overturning is 1.41, which occurs in the spillway section under condition D_1. This is a condition including all known forces, some of a transitory nature, so that the dam is cosidered safe against this form of failure.

The lowest value of the factor of sefety against sliding also occurs under this condition D_1 and is 5, which is above the figure of 4 used by the United States Department of the Interior, Bureau of Reclamation reffered to as USBR.

For the rock formation the cohesion is reduced to 200 psi and the foundation of the spillway section is given a rise of 1 in 13 in the downstream direction, to resist the thrust, giving safety factory against sliding of 4 for the head condition without earthquake ; which is accepted by the USBR.

The bulkhead foundation is given a rise of in 20, which results in factors of between 4 and 5 with earthquake.

Stresses : For the two grades of concrete to be used in construction, the allowable stresses are:-

4000 psi, used for the 10 ft skin of dam

compression	—1000 psi
Shear	— 500 psi
Diagonal tension	— 80 psi

2000 psi, used for hearting of dam

Compression	— 500 psi
Shear	— 250 psi
Diagonal tension	— 40 psi

From the tables and plates the maximum stresses are:—

For earthquake condition :

Spillway section: Case D_1 at elevation 1050 feet,

388 psi compression at the downstream face,

183 psi shear at the downstream face,

38 psi tension at the upstream face with uplift allowance.

Standard Bulkhead Section: Case D_r at elevation 1130 feet.

221 psi compression at the downstream face,

102 psi shear at the downstream face,

No tension.

Stairwell Section: Case D_1 at elevation 1132 feet.

218 psi compression at the downstream face,

109 psi shear at the downstream face,
—Case E1 at elevation 1150 feet.

45 psi tension at the upstream face.

For the No Earthquake condition:

Spillway Section: Case D at elevation 1050 feet.

267 psi compression at the downstream face,

117 psi shear at the downstream face
No tension.

Standard Bulkhead Section: Case E at elevation 1130 feet.

155 psi compression at the upstream face,

77 psi shear at the upstream face,
No tension.

Stairwell Section: Case E at elevation 1132 feet.

147 psi compression at the upstream face,
73 psi shear at the upstream face,

No tension.

It can be seen that tension only develops with earthquake.

Stresses due to differential deflection:

The steep banks of this dam give differences in the depths at each end of a 40—feet block of the bulkhead section in the order of 35 feet. This results in a difference in the deffections at the ends of the block giving stresses due to the twisting action induced.

These stresses were investigated and the maximum found to be 63 psi in shear.

Inspection and Access Calleries:

The inspection gallery in the main dam is located near the upstream face between elevation 1135 feet and 1148.5 feet, and is illustrated in sketch No. 1—

The stresses are obtained from the results given in the Engineering Monograph No.12 published by the USBR,

Considering th two cases, D_1, El :

Direct Stresses;

		Case D_1		*Case* El
At apex	.....	215 psi	—	111 psi
At centre of base	.....	137 psi	—	112 psi
At mid-side	.....	4 psi	X +	251 psi
At bottom corner	.....	298 psi	+	372 psi

Compression shown — positive. Tension shown — negative

These give a maximum tension of 8, 680 pounds per foot run of gallery, and 3/4-inch bars at 12 inches are required at top and bottom with nominal steel at sides.

The access gallery is in an area which has a maximum principal stress of 110 psi parallel to the downstream face. By a similar analysis to that of the inspection gallery, the force per foot run is found to be 8,450 pounds which also requires 3/4—inch bars at 12—inch centres.

Temperature Stresses ;

The basic formula for these stresses is

$$f = CE_c R (t_p + t_r - t_f)$$

where f is the stress produced

C = Cofficient of thermal expansion 0·000005

E = Modulus of elasticity of concrete $2X\ 10^6$ psi

R = restraint factor unity at the foundation dimishing to zero at a height of one-half the block length.

T_p = placing temperature.

t_r = Temperature rise of concrete after placing due to heat of hydration of concrete.

t_f = final temperature of structure.

W. M. Price and C. L. Townsend, in their report of January 19, 1956, indicated that with a placing temperature of 55 to 60 degrees Fahrenheit, a rise of 15 to 20 degrees Fahrenheit due to hydration, the drop to the mean annual air temperature of 72 degrees Fahrenheit will be a maximum of 10 degrees Fahrenheit.

This wil result in a stress = 10R (10F) = 100 R
at the foundation R is 1 and stress will then be 100 psi tension, which is permissible.

This figure is sensitive to rises in the placing temperature and it is important that all precautions be taken to keep this down and reduce any rise in the concrete temporature after placing. This is being done by adopting the following measures to control the temperatures in mass concrete :—

1. Cement content of Concrete mix for the mass concrete is kept minimum possible compatible with the workability and strength requirements. The Cement used is 300 pounds (0·8 barrels) per cubic yard of concrete. This Compares most favourably with 0·75 to 1·2 barrels used on various Dams of T. V. A. U. S. A.

2. The concrete is placed in 2½feet lifts within the 30 feet height of the main dam immediately above the rock foundation, and in 5 feet lifts in portion of the main dam other than the lower 30 feet.

3. Cooling Coils are used in both 2½ and 5 feet lifts, and cold river water circulated.

4. 72 hours minimum exposure between 2½ feet lifts is given where no coils are used, and 48 hours where cooling coil are used.

5. In summer effctive means such as shading of water tanks, water pipings, spraying of aggregates, aluminum painting of Transit Mixers, Cement Silos, buckets, etc ; are employed. The aggregate bins belt conveyors are also shaded to avoid direct sun.

The maximum placing temperatures of concrete placed within the 30 feet height of the main dam immediately above the rock foundation; remained below 65 F⁰ ; as this portion of the Dam was completed in the winter months. The placing temperatures in the summer varied from 80⁰ to 20⁰F.

Mean annual temperature of the Day : 75° F. This is the temperature which the mass concrete would eventually attain.

7. *Conclusions* :

The dam will be safe under all known possible conditions of loading, as the following conditions have been maintained in the field.

(1)—The spillway base does not extend below elevation 1050 feet and the bulkhead base below elevation 1132 feet.

(2)—A good sound rock formation was established for the base, which was cleaned and has all seams and fissures plugged or grouted.

(3)—The formation was given a rise to resist the thrust as calculated or have an equivalent deformity in its profile.

(4)—All horizontal construction joints are sloped upwards in the downstream direction and treated in accordance with the concrete pecifications.

(5)—The concrete is of minimum strength, as specified.

(6)—Adequate precaution have been taken to avoid tension due to temperature variation and shrinkage of the concrete.

8. *Acknowledgements :*

The author is indebted to Messrs. H. G. Acres & Co Ltd., Niagara Falls, Canada, for their kind permission to draw material extensively from their design report of October, 1957 on the above subject. Thanks are also due to Mr. Rafiq Ahmad, Assistant Engineer (Designs) Warsak Dam Project Organisation, for his able assistance in the preparation of the manuscript.

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Appendix. A

Design Calculations.

To check the stresses and stability of the dam at all elevations, the spillway and bulkhead sections have been tested at 20′ intervals starting from the top considering horizontal sections and adding the new section to previous one, going down right upto the base at El 1050. Computations have been made for the various combination of loads as described above at every section. However, to present a clear picture of design, cases B and B_1 have been dealt here in detail for spilway section with base at elevation 1050. The results for these two cases B & B_1 and also for all the other cases have been tabulated at the end.

Design of Spillway Section at El. 1050

Case B : Headwater at Elevation 1269
Tailwater at ,, 1128
Silt level at ,, 1230
With uplift.
No Earthquake.

Case B1 : As case B above plus Earthquake.

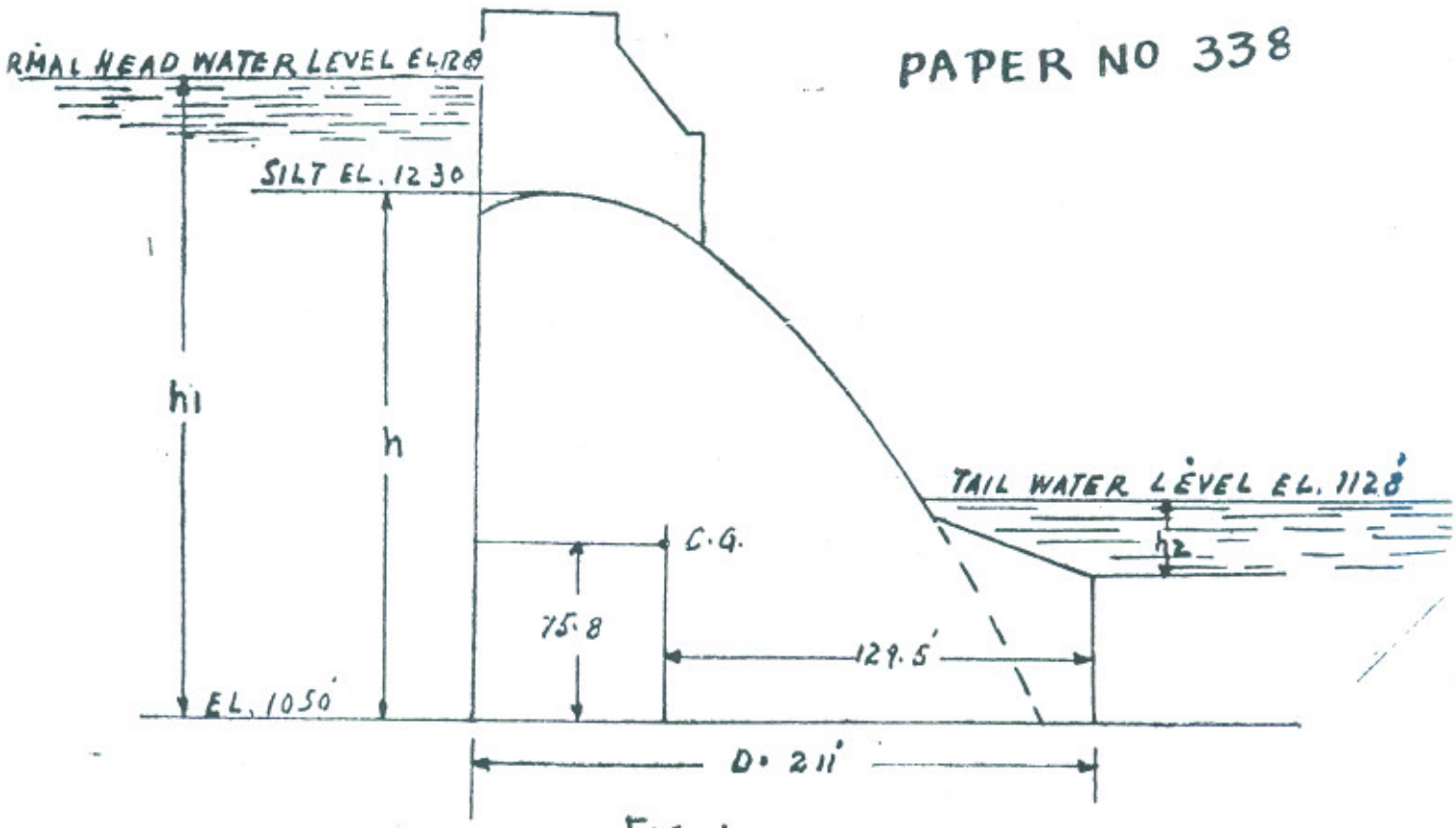


FIG. 1.

Design date :

1. Head of water at u/s face (h_1 = 1269 - 1050
   = 219 ft.
2. Head of water at d/s face (h_2 = 1128 - 1102
   = 26ft.
3. Width at base (D) 211 ft.
4. Length of Section (B) = 50ft.
5. Specific wt. of water (W) = 62·5 lbs/cuft.
6. Silt head (h) = 1230 - 105
   = 180 ft.
7. Wt. of the section (W) = 1,98,597 Kips.
8. Specific wt. of silt (ws) = 56·4 lbs/ cuft. (see page-7).
9. Specific wt. of concrete. = 145 lbs/cuft.
10. XSectional Area of base at E.L. = B × D
    = 50 × 211 □′
11. × = 0·15 (see Page 8)
12. C = 0·65 (see page 4)
13. Rs = 400 psi (See page 4)

Calculations:

1. *Weight of the dam*:

× —sectional Area = 2,70,000 (as measured)
Length of section (B) = 50 ft,
Volume of concrete = 50 × 2,70,000 cuft.
Specific wt. of concrete = 145 lbs/cuft. Assumed,
Weight of concrete. $= \frac{50 \times 2,70,000 \times 145}{1,000}$ kips
= 1,98,597 kips.

L.arm. = 129·5 ft. (as measured).

Weight Kips(W)	*L.Arm (ft).*	*Resisting moment(ft. Kips).*
1,98,597	129. 5	2,57,18,045.

2. *Horizontal water pressure upstream :*

h_1 = 1269—1050
= 219 ft.

Pressure $= 1/2\ wh_1^2 \times 50$ lbs.

$$= \frac{\frac{1}{2} \times 62{\cdot}5 \times (219)^2 \times 59}{1000} \text{ Kips.}$$

$= 75{,}000$ KiPs.

Lever arm, $= \frac{h_1}{3} = \frac{219}{3} = 73$ ft.

Horizontal *force (KIPS)*	L.Arm *(Ft.)*	Overturning *moment (Ft.Kips).*
75,000	73	54,80'000

3. *Horizontal water pressure downstream* :

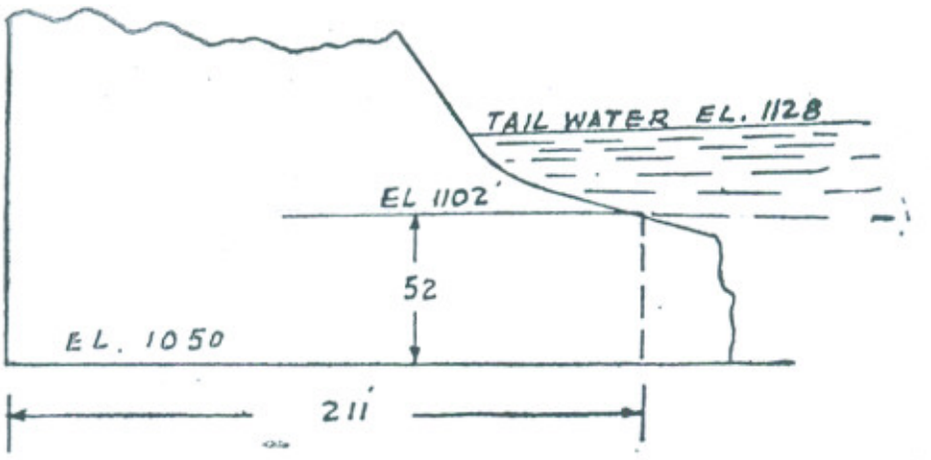
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$h_2 = 1128 - 1102$

$= 26$ ft.

Pressure

$= \frac{1}{2} w\ h \times 50$ lbs.



$$= \frac{\frac{1}{2} \times 62{\cdot}5 \times (26)^2 \times 50 \text{ Kips.}}{1{,}000}$$

$= 1055$ Kips.

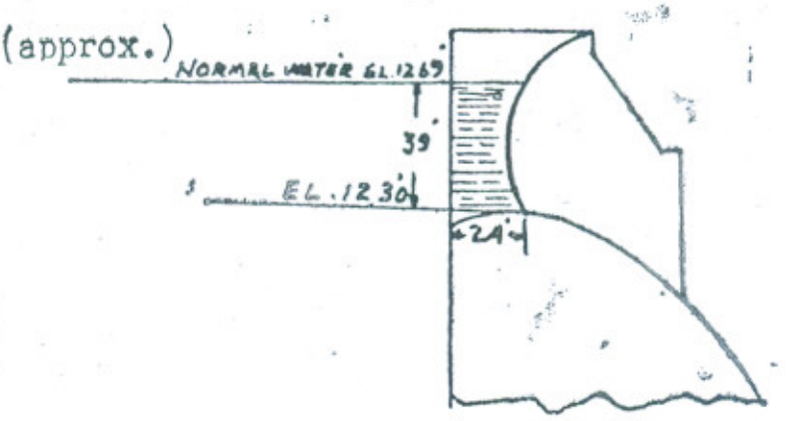
L. arm $= (52 + 26/3) = 60{\cdot}67$ ft.

Force (Kips)	*L.Arm (Ft.)*	*Resisting moment (Ft.Kips).*
1,055	60·67	64,000

4. Vertical water weight on the top *of the dam with the gates closed.*

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X—area of the water = 39 × (approx.) 24 = 936□′ (approx.)



Specific weight of water = 62·5lbs/cuft.

∴ Weight of water = $\frac{62{\cdot}5 \times 936 \times 50}{1{,}000}$ Kips.

= 2,925 Kips.

L.arm = $\frac{(211 - 24)}{2}$

= 199 ft.

Vertical Wt(Kips)	*L. Arm (Ft).*	*Resisting moment (Ft.Kips).*
2,925	199	5,82,500

5. *Vertical Tailwater Pressure*:

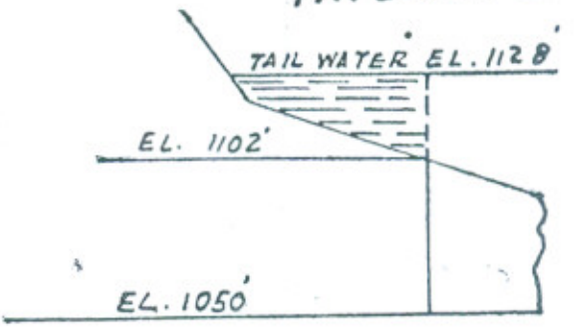
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X—area of the water = 1,075/□′ (as measured)

Specific weight of water = 62·5 lbs/cuft.

Verticalweight of water = $\frac{62{\cdot}5 \times 50 \times 1{,}075}{1{,}000}$ Kips

= 3,360 Kips.



L.arm = 21·3 ft. (as found in the laboratory).

Vertical wt.(Kips).	*L.Arm (Ft.)*	*Resisting moment (Ft.Kips).*
3,360	21·3	71,700

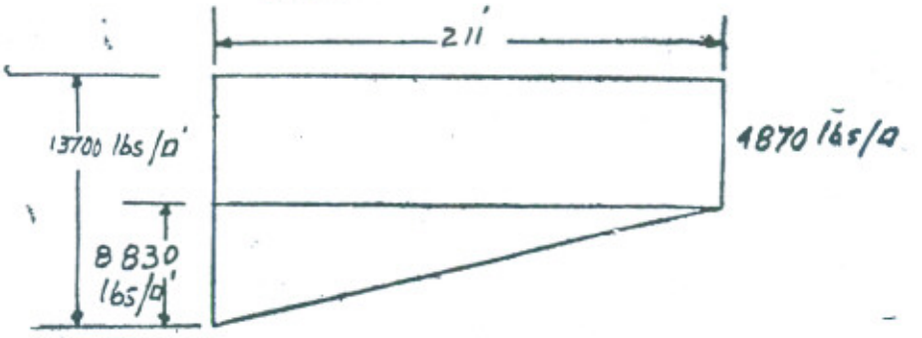
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6. *Uplift Pressure*:

h_1 = 1269 − 0105 = 219 ft.

h_2 = 1128 − 1050 = 78 ft.

Headwater pressure wh_1



= 62·5 × 219

= 13,700 lbs/□′

Tailwater pressure $= 62{\cdot}5 \times 78$
$= 4{,}870$ lbs/□′

From assumption, uplift varies linearly from 13700 lbs/□′ at heel to 4870 lbs/□′ at toe as shown in Fig. 5.

Average pressure $= \frac{13{,}700 + 4{,}870}{2} = 9{,}285$ lbs/□

Effective area of uplift $= 2/3$ base area.
$= 2/3 \times 211 \times 50$
$= 7{,}033$□

Uplift force ... $= \frac{7{,}033 \times 9{,}285}{1{,}000}$ Kips
$= 65{,}400$ Kips.

Lever arm. $$= \frac{4{,}870 \times 211 \times 211/2 \times 50 + \frac{8{,}830}{2} \times 211 \times \frac{2 \times 211}{3} \times 50}{\frac{8{,}830}{2} \times 211 \times 50 + 4{,}870 \times 211 \times 50}$$

$$= \frac{4{,}870 \times \frac{211}{2} + \frac{8{,}830}{2} \times \frac{2}{3} \times 211}{\frac{8{,}830}{2} + 4{,}870}$$

$= 123$ ft,

Vertical force (Kips).	*L. Arm (Ft).*	*Overturning moment (Ft. Kips).*
65,400	123	80,50,000

7. *Silt Pressure* :

Silt pressure $$\frac{Wsh^2}{2} \times \frac{(1-\sin\phi)}{(1+\sin\phi)}$$

$$= \frac{56.4h^2}{2} \frac{(1-0.575)}{(1+0.575)}$$

$= 8.25h^2$ lbs/ft width.

h $= 1230\text{-}1050$
$= 180$ ft.

$\therefore$ pressure $= 8.25 \times (180)^2 \times 50$ lbs

$= \frac{1,33,50,000}{1000}$ Kips.

$= 13,350$ Kips.

Lever arm $= \frac{h}{2 \cdot 5}$

$= \frac{180}{2 \cdot 5} = 72$ ft.

Silt force (Kips)	*L. Arm (Ft).*	*Overturniug moment Ft. (Kips).*
13,350	72	9,61,000

8. *Earthquake forces* :

(a) *Due to inertia of structure.*

Earthquake force $= \alpha W$

where W is the Wt, of the

50' concrete section $= 1,98,597$ Kips.

Force $= 0.15 \times 1,98,597$

$= 29,800$ Kips.

L. arm $= 75.8$

Force (Kips).	*L. arm (Ft).*	*Overturning moment Ft. Kips).*
29,800	75.8	22,58,000

(b) *Due tv increased water pressure:*

Force $= 5.25\ h_1^2$

$= 5.25\ (219)^2 \times 50$

$= 12,480$ Kips.

L. arm $= 0.424\ h_1$

$= 0.424 \times 219$

$= 93.0$ ft.

Force (Kips).	*L. arm. (ft).*	*Overturning moment (Ft. Kips).*
12,480	93	11,60,000

CASE B AT EL. 1050 :

Summation of horizontal forces, vertical forces resisting moment and overturning moment *i.e.* Σ P, Σ W, Σ R, M; Σ O.M.

Item	Horizontal forces Σ P (Kips)	Vertical forces Σ W (Kips)	Overturning moment Σ O.M (Ft. Kips).	Resisting moment Σ R.M. (Ft. Kips).
1. Concrete Weight.		1,98,597		2,57,18,000
2. Horizontal water pressure U/S.	75,000		54,80,000	
3. Horizontal water pressure D/S	- 1,055			64,000
4. Head water weight.		2,925		5,82,500
5. Tailwater weight.		3,360		71,700
6. Uplift		- 65,400	80,50,000	
7. Silt pressure	13,350		9,61,000	
SUMMATION:	87,295	1,39,482	1,44,91,000	2,64,36,445

Case B at E.L. 1050
1

Item	Horizontal forces Σ P (Kips)	Vertical forces Σ W (Kips)	Overturning moment Σ O.M. (Ft. Kips)	Resisting moment Σ R.M. (Ft.Kips).
(*i*) Case B	87,295	1,39,482	1,44,91,000	2,64,36,445
(*ii*) Earthquake forces :				
(*a*) Inertia of structure.	29,800		22,58,000	
(*b*) Increased water pressure	12,480		11,60,000	
Summation :	1,29,575	1,39,482	1,79,09,000	2,64,36,445.

Stability Computations :

1. **Factor of safety against overturning :-**

i.e. $F_s = \frac{\Sigma \text{ R.M.}}{\Sigma \text{ O.M.}}$

Case B

Σ R.M. = 2,64,36,445 Ft.Kips.

Σ O.M. = 1,44,91,000 Ft.Kips.

$F_s = \frac{2,64,36,445}{1,44,91,000}$

$= 1{\cdot}83$

Case B_1

Σ R.M. = 2, 64,36,445 Ft.Kips.

Σ O.M. = 1,79,09,000 Ft.Kips

$F_s = \frac{2,64,36,445}{1,79,09,000}$

$= 1{\cdot}48$

2. (*a*) *Sliding factor* : (*f*).

$f = \frac{\Sigma \text{ P.}}{\Sigma \text{W.}}$

Case B

Σ P. = 87,295 Kips.

Σ W. = 1,39,482 Kips.

$\therefore f = \frac{87,295}{1,39,482}$

$= 0{\cdot}63$

Case B_1

Σ P. = 1,29,575 Kips

Σ W. = 1,39,482 Kips.

$\therefore f = \frac{1,29,575}{1,39,482}$

$= 0{\cdot}93$

2. (*b*) *Shear friction factor* (f_s)

$$f_s = \frac{\Sigma WC + B_1 Rs}{\Sigma P}$$

Case B

$\Sigma P = 1,39,482$ Kips.

$C = 0{\cdot}65$

$B_1 = 50 \times 211$ □′

$R_s = 400$ psi.

$\Sigma P = 87,295$

$$\therefore f_s = \frac{1,39,482 \times 0{\cdot}65 \times \frac{400}{1000} \times 50 \times 211 \times 144}{87,295.}$$

$= 8{\cdot}0$

Case B_t

$\Sigma W = 1,39,482$ Kips.

$C = 0{\cdot}65$

$B_1 = 50 \times 211$ □′

$R_s = 400$ psi.

$\Sigma P = 1,29,575$ Kips.

$$\therefore f_s = \frac{1,39,482 \times 0{\cdot}65 \times \frac{400 \times 144 \times}{1,000} 211 \times 50}{1,29,575}$$

$= 5{\cdot}0$

3. *Position of Resultant* :

(*i*) Distance of resultant from d/s face of section (E).

$$E = \frac{\Sigma RM - \Sigma OM}{\Sigma W}$$

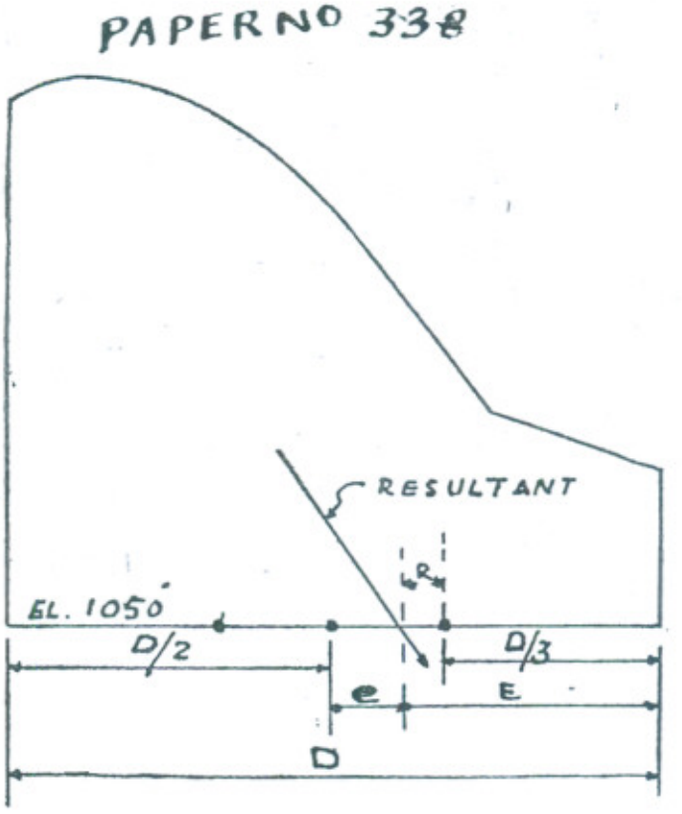
(*ii*) Decentricity (e) from centre-line of section (E).

$$e = \frac{\text{Base width}}{2} - E$$

$= D/2 - E$

(*iii*) Distance of resultant from inside.
middle—third i.e. (R)

$$R = E - \frac{\text{Base width}}{3}$$
$$= E - D/3.$$



Case B.

Σ O.M = 1,44,91,000 Ft. Kips.
Σ R.M. = 2,64,36,445 Ft. Kips.
Σ W = 1,39,482, Kips.

$$\therefore E = \frac{\Sigma \text{ R.M.} - \Sigma \text{ O.M.}}{\Sigma W}$$
$$= \frac{2,64,36,445 - 1,44,91,000}{1,39,482}$$
$$= 85.5$$

$$R = E - \frac{D}{3}$$
$$= 85{\cdot}5 - 70{\cdot}3$$
$$= 15{\cdot}2 \text{ ft.}$$

$$e = \frac{D}{2} - E$$
$$= \frac{211 - 85{\cdot}5}{2}$$
$$= 20{\cdot}0 \text{ ft.}$$

Case B_1

O.M. 1,79,09,000 Ft.Kips.
R.M. = 2,64,36;445 Ft.Kips.
W. = 1,39,482 Kips.

$$E = \frac{2,64,36,445 - 1,79,09,000}{1,39,482}$$
$$= 61{\cdot}5 \text{ ft.}$$

$$R = 61{\cdot}5 - \frac{211}{3} = -8{\cdot}8 \text{ ft.}$$

$$(e) = \frac{211}{2} - 61{\cdot}5$$
$$= 44{\cdot}0 \text{ ft.}$$

4. *Normal Stresses* :

(*i*) Including uplift.

N_H = Normal pressure at Heel,

N_T = Normal pressure at toe

then $N_T = \frac{\Sigma W}{B\,D} + \frac{6e.\Sigma W}{B\,D^2}$

$N_H = \frac{\Sigma W}{BD} - \frac{6e\,\Sigma W}{BD^2}$

(*ii*) excluding uplift

P_a = Normal pressure at Heel

P_b = Normal pressure at toe

U_H = 2/3 uplift pressure at Heel

U_T = 2/3 uplift pressure at toe.

then $P_a = N_H + U_H$

$P_b = N_T + U_T$

Case B :

(*i*) Including uplift

ΣW = 1,39,482 Kips

D = 211 ft.

B = 50 ft.

e = 30 ft.

$\therefore N_T = \frac{\Sigma W}{BD} + \frac{6e\,\Sigma W}{BD^2}$

$= \frac{1,39,482}{50 \times 211} + \frac{6 \times 20 \times 1,39,482}{50 \times (211)^2}$

$= 13{\cdot}25 + 7{\cdot}54$

$= 20{\cdot}75$ Kips/□′

$N_H = 13{\cdot}25 - 7{\cdot}54$

$= 5{\cdot}67$ Kips/□′

(ii) excluding uplift :

$P_a = N_H + U_H$

$= 5{\cdot}67 + 9{\cdot}13$

$= 14{\cdot}80$ Kips/□′

$P_b = N_T + U_T$

$= 20{\cdot}75 + 3{\cdot}25$

$= 24{,}00$ Kips/□′

Case B_1

(*i*) including uplift:

$\Sigma W = 1{,}39{,}482$ Kips.

$e = 44$ ft

$\therefore N_T = \frac{1{,}39{,}482}{50 \times 211} + \frac{6 \times 44 \times 1{,}39{,}482}{50 \times (211)^2}$

$= 13{\cdot}20 + 16{\cdot}6$

$= 29{\cdot}8$ Kips/□′

$N_H = 13{\cdot}20 - 16{\cdot}6$

$= -3{\cdot}4$ Kips/□′ (Tension).

(*ii*) U_H = 2/3 uplift at Heel

$= 2/3 \times Wh_1$

$= 2/3 \times 62{\cdot}5 \times 219$

$= 9{\cdot}100$ lbs/□′

$= 9{\cdot}1$Kips/□′

$U_T = 2/3 \times 62{\cdot}5 \times 78$

$= \frac{3{,}250}{1{,}000}$

$= 3{\cdot}25$ Kips/□′

(*i*) excluding uplift :

$P_a = N_H + U_H$

$= -3{\cdot}4 + 9{\cdot}1$

$= 5{\cdot}7$ Kips/□

$P_b = N_T + U_T$

$= 20{\cdot}8 + 3.25$

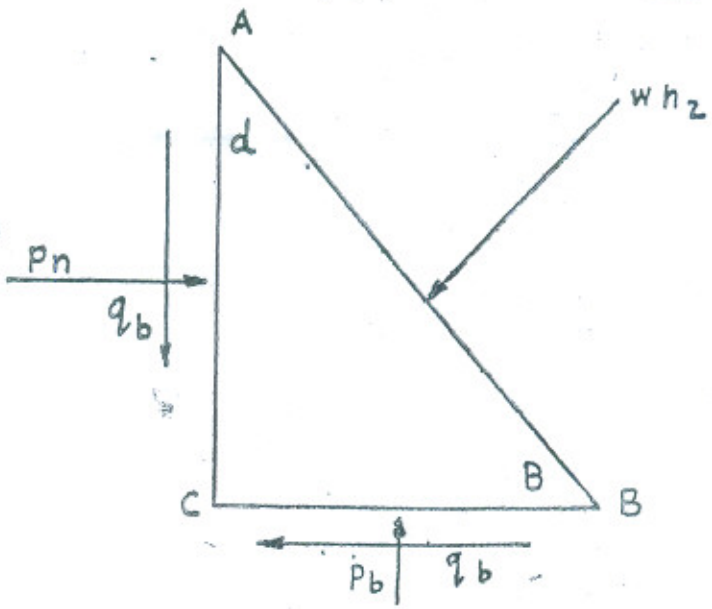
$= 33{\cdot}05$ Kips/□

Principal Stresses:

q_b = Horizontol shear stress at face.
p_b = vertical normal stress at face on a horizonal plane.

p_n = Horizontal normal stress at face.
wh_2 = Mean intensity of water pressure.
β = Angle of d/s face with the horizontal.

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Consider downstream face — to find horizontal shear and normal stress:

Resolving forces along AC

$q_b \cdot AC$

$\sin \beta$

$\therefore \quad q_b \; AC$

$= p_b \; BC - Wh_2 \; COS \; \beta \times AB$

$= \frac{AC}{AB} COS \; \beta = \frac{BC}{AB}$

$= p_b \; BC - Wh_2 \; \frac{BC}{AB} \times AB$

$= P_b \; BC - Wh_2 \; BC$

$= (P_b - Wh_2) BC$

$\therefore \quad q_b$

$= (p_b - Wh_2) \frac{BC}{AC}$

$\frac{BC}{AC}$

$= Cot \; \beta$

$\therefore \quad q_b$

$= (P_b - Wh_2) \; Cot \; \beta \; .....(1)$

Resolving forces along BC

$P_n \cdot AC$

$= q_b . BC + Wh_2 \; Sin \; \beta \times AB$

$P_n \; AC$

$= q_b \; BC + Wh_2 \; AC$

$\therefore \quad P_n$

$= q_b \frac{BC}{AC} + Wh_2$

$= (q_b \; Cot \; \beta + Wh_2) .....(2)$

To calulate horizontal shear stress :

Upstream face :

Since u/s face of dam is vertical, there is no horizontal shear stress on this face.

d/s face :

$q_b = (P_b - Wh_2) \cot \beta$

Same as No. (1).

Normal stresses on vertical planes :

$Pn = Wh_2 + q_b \cot \beta$ (downstream)

$Pn = Wh_1$ (Upstream).

To find angle of principal planes and values of Max. and Min. principal stresses.

Let AB be the principal plane on which principal stress R acts.
Fig. (i).

Resolving forces along AB

$$q_b\ AC\ COS\ \alpha + Pn\ AC\ \sin\alpha = q_b\ BC \sin\alpha + P_b\ BC\ COS$$

$$COS\ \alpha = \frac{AC}{AB}$$

$$\text{Sin}\ \alpha = \frac{BC}{AB}$$

$$\text{Tan}\ \alpha = \frac{BC}{AC}$$

$$\therefore\quad q_b\ AC \times \frac{AC}{AB} + Pn\ AC \times \frac{EC}{AB} = q_b\ BC \times \frac{BC}{AB} - P_b\ BC \cdot \frac{AC}{AB}$$

i. e. $q_b\ AC^2 - q_b\ BC^2 = P_b\ BC.\ AC - Pn\ AC.\ BC.$

Dividing by AC. BC

$$\therefore\quad q_b \frac{AC}{BC} - q_b \frac{BC}{AC} = P_b - Pn$$

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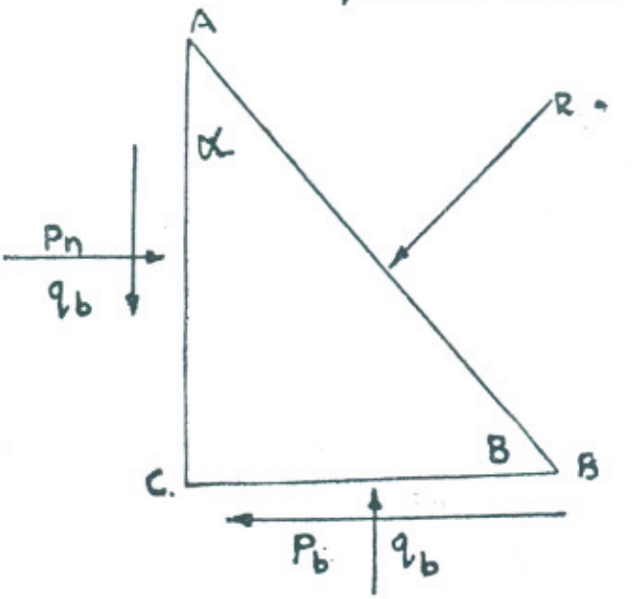


FIG. (1)

$$\therefore \quad q_b \left(\frac{AC}{BC} - \frac{BC}{AC}\right) = P_b - P_n$$

$$\text{i.e.} \quad q_b \, (\text{Cot}\alpha - \text{Tan}\alpha) = P_b - P_n$$

$$\text{Now COT } \alpha - \text{Tan } \alpha = \frac{2}{\tan 2\alpha}$$

$$\therefore 7 \quad \text{Tan } 2\alpha = \frac{2 \, q_b}{P_b - P_n}$$

$$\text{or} \quad \text{Tan } 2\alpha = \frac{-2q_b}{P_n - P_b}$$

$$P_1 \text{ or } P_2 = \frac{P_n + P_b \pm \sqrt{(P_n - P_b)^2 + q_b^2}}{2} \quad \sqrt{\frac{}{4}}$$

Principal stresses

Case B: excluding nplift : at EL. 1050

Downstream end :

P_b (d/s) $= 24{\cdot}00$ Kips/□′

Wh_2 (d/s) $= \frac{62{\cdot}5}{1000} \times 26$

$= 1{\cdot}63$ Kips/□′

$P_b - Wh_2 = 22{\cdot}37$ Kips

$\beta = 50°$

Cot $\beta = 0{\cdot}839$

$q_b = (P_q - Wh_2)$ Cot β

$= 22{\cdot}37 \times 0{\cdot}839$

$= 18{\cdot}9$ Kips/□′ (d/s).

q_b Cot $\beta = 18{\cdot}9 \times 0{\cdot}839$

$= 15{\cdot}9$

P_n (d/s) $= Wh_2 + q_b \text{ Cot } \beta$

$= 1{\cdot}63 + 15{\cdot}9$

$= 17{\cdot}53$ Kips/□′

Upstream end :

P_n (u/s) $= Wh_1$ (where $h_1 = 219$ ft.)

$= 1.63 + 15{\cdot}9$

$= 13{\cdot}7$ Kips/□′

$P_b = P$ (u/s)

$= 14{\cdot}8$ Kips/□′

$\beta = 90°$

$\therefore \text{Cot } \beta = 0$

$\therefore q_b = 0$

Principal Stresses and shear stresses at d/s end.

Case B

$q_b = 18{\cdot}9$ Kips/□′

$\therefore (q_b) = 356{\cdot}7$

P_n (d/s) $= 17{\cdot}53$ Kips/□′

P_b (d/s) $= 24{\cdot}00$ Kips/□′

$\therefore \dfrac{P_n + P_b}{2} = 20{\cdot}77$ Kips/□′

$P_n - P_b = -\ 6{\cdot}47$ ″

$\dfrac{(P_n - P_b)^2}{4} = \dfrac{42}{4}$

$= 10{\cdot}5$

P_1 or P_2 (D/s) $= \dfrac{P_n + P_b}{2} \pm \sqrt{\dfrac{(P_n - P_b)^2}{4} + q_b^2}$ &

$$= 20{\cdot}77 = \sqrt{10{\cdot}5 + 356{\cdot}7}$$

$$= 20{\cdot}77 \pm 19{\cdot}2$$

$$= 39{\cdot}94,\ 1{\cdot}60 \text{ Kips/}\square'$$

Tan 2α $$= \frac{-2\ q_b}{P_n - P_b}$$

$$= \frac{-2 \times 18{\cdot}9}{-6{\cdot}47}$$

$$= 5{\cdot}84$$

Giving α $= 40{\cdot}1°$

To find principal stresses and shear stresses at various pts. namely u/s, 3/4, 1/2, ...,3/4, d/s along a horizontal plane.

Case B

P_n (ups) $= 13{\cdot}7$ Kips/$\square'$

P_n (d/s) $= 17{\cdot}53$ KiPs/$\square'$

P_b (d/s) $= 24{\cdot}0$ Kips/$\square'$

P_b (u/s) $= 14{\cdot}8$ Kips/$\square'$

q_b (u/s) $= 0$ Kips/$\square'$

q_b (d/s) $= 18{\cdot}9$ Kips/$\square'$

Variations from u/s to d/s is linear (Assumtions 3 & 4),
so values of P_n, P_h, q_b, at various sections along the horizontal plane at E1· 1050 will be as per Table No. 1 below :

TABLE No. I. Values of P_n, P_b, & q_b at various section

along the horizontal plane E1·1050

Section	P_n	P_b	q_b
U/s	13·7	14·8	0
3/4	$13·7 + \frac{17·53 - 13·7}{8} =$ $13·7 + 0·48 = 14·18$	$148 + \frac{24 - 14·8}{8}$ $= 14·8 + 1·15 = 15·95$	$\frac{18·9}{8} = 2·36$
1/2	$14·18 + 0·48 = 14·66$	$15·95 + 1·15 = 17·10$	$2·36 \times 2 = 4·75$
1/4	$14·66 + 0·48 = 15·14$	$17·10 + 1·15 = 18·25$	$4·73 + 2·86 = 7·09$
0	$15·14 + 0·48 = 15·62$	$18·25 + 1·15 = 19·40$	$7·09 + 2·36 = 9·45$
-1/4	$15·62 + 0·48 = 16·1$	$19·4 + 1·15 = 20·55$	$9·45 + 2·36 = 11·81$
-1/2	$16·1 + 0·48 = 16·58$	$20·55 + 1·15 = 21·70$	$11·81 + 2·36 = 14·17$
-3/4	$16·58 + 0·48 = 17·06$	$21·7 + 1·15 = 22·85$	$14·17 + 2·36 = 16·53$
D/S	$17·06 + 0·58 = 17·54$	$22·85 + 1·15 = 24·00$	$16·53 + 2·36 = 18·9$

Case B (excluding uplift) at El. 1050.

Downstream end.

P_b (d/s) $= 33{\cdot}05$ Kips/□′

Wh_2 (d/s) $= 1{\cdot}63$ Kips/□′

$(P_b - Wh_2)$ $= 33{\cdot}05 - 1{\cdot}63$

$= 31{\cdot}42$ Kips/□′

β (d/s) $= 50^o$

Cot B (d/s) $= 0{\cdot}839$

P_b $= (P_b - Wh_2)$ Cot β

$= 31{\cdot}42 \times 0{\cdot}839$

$= 26{\cdot}4$ Kips/□′

q_b cot β $= 22{\cdot}1$

P_n $= Wh_2 + q_b$ Cot β

$= 1{\cdot}63 + 22.1$

$= 23{\cdot}73$ Kips/□′

Upstream end.

P_b $= P_a$

$= 5{\cdot}7$ Kips/□′

P_n $= Wh_1$

$= 13{\cdot}7$ Kips/□′

β $= 90^o$

Cot β $= 0$

$\therefore$ q_b $= 0$ u/s.

To find principal stresses and shear stresses at various section namely, u/s, 3/4, — , - 3/4, d/s along a horizontal plane at El. 1050.

Case B_1 (excludieg uplift)

P_n $= 13{\cdot}70$ Kips/□′

P_n (d/s) $= 23{\cdot}73$ Kips/□′

P_b (u/s) $= 5{\cdot}7$,,

P_b (d/s) $= 33{\cdot}05$,,

q_b (u/s) $= 0$,,

q_b (d/s) $= 26{\cdot}4$,,

Variations from u/s to d/s is linear (assumptions 3 & 4) So values of P_n, P_b and q_b at various sections along the horizontal plane at E1·1050 will be as per Table No. 3 below :—

TABLE NO. 3

Values of P_n, P_b, q_b at evarious sections along the horizontal plane at E1. 1050

Section	P_n	P_b	q_b
U/S	13·70	5·7	0
3/4	$13{\cdot}7 + \frac{23{\cdot}73 - 13{\cdot}7}{8}$ $13{\cdot}7 + 1{\cdot}25 = 14{\cdot}95$	$5{\cdot}7 + \frac{33{\cdot}05 - 5{\cdot}7}{8}$ $5{\cdot}7 + 3{\cdot}42 = 9{\cdot}12$	$\frac{26{\cdot}4}{8} = 3.3$
1/2	$14{\cdot}95 + 1{\cdot}25 = 16{\cdot}21$	$9{\cdot}12 + 3{\cdot}42 = 12{\cdot}54$	$3{\cdot}3 \times 2 = 6{\cdot}6$
1/4	$16{\cdot}21 + 1{\cdot}25 = 17{\cdot}46$	$12{\cdot}54 + 3{\cdot}42 = 15{\cdot}95$	$3{\cdot}3 \times 3 = 9{\cdot}9$
0	$17{\cdot}46 + 1{\cdot}25 = 18{\cdot}71$	$15{\cdot}95 + 3{\cdot}42 = 19{\cdot}37$	$3{\cdot}3 \times 4 = 13{\cdot}2$
− 1/4	$18{\cdot}71 + 1{\cdot}25 = 19{\cdot}96$	$19{\cdot}37 + 3{\cdot}42 = 22{\cdot}79$	$3{\cdot}3 \times 5 = 16{\cdot}5$
− 1/2	$19{\cdot}96 + 1{\cdot}25 = 21{\cdot}21$	$22{\cdot}78 + 3{\cdot}42 = 26{\cdot}20$	$3\,3 \times 6 = 19{\cdot}8$
− 3/4	$21{\cdot}21 + 1{\cdot}25 = 22{\cdot}46$	$26{\cdot}2 + 3{\cdot}42 = 29{\cdot}62$	$3{\cdot}3 \times 7 = 23\,1$
D/S	23·75	33·05	26·4

TABLE NO. 5

Results of the Calculations for cases

B & B_1 for Spillway Section at El.1050

Condition	F_S +	f	f_s	R	N_T Kips/□′	N_H Kips□′	Principal Stresses			
							Toe		Heel	
							Max.Comp Kips/□′	Shear + Kips/□′	Max.Comp Kips/□′	Min.Comp Kips/□′
CaseB	1·83	0·63	8	15·2	20·75	5:67	39·97	15·9	14·8	13·7
Case B_1	1·48	0·93	5	8·8	29·8	—3·4	55·19	26·4	13·7	5·7

In a similar manner, computations for all combinations of leads for the spillway section, bulkhead section and stairwell sections have been made at every 20′ interval starting from the top,considering horizontal sections and adding the new section to the previous one, going down right upto the base. However, only summary of results for the above three sections with their bases at E.L. 1050,1130 & 1132 respectively are given as per Table Nos 6,7 & 8 below.

Table No. 6, Spillway Section at El 1050

Summary of results for the various conditions of loading.

Condition	Factor of safety against over turning. F_s	Sliding factor f	Shear-fraction factor of safety. F_s	Distance of rasultant from inside middlethird. R	Normal stress including uplift		Principal stresses.			
							Toe		Heel	
					NT lbs/□″	NH lbs/□″	Max. eomp lbs/□″	Shear lbs/□″	Max. comp lbs/□″	Min comp lbs/□″
Case A	1·95	0·53	9·5	22·2	126	58	237	107	122	13
Case B	1·83	0·63	8	15·2	144	39.4	258	111	103	7
Case C	1.99	0·58	8	14·9	151	51.0	257	129	115	10
Case D	1·75	0·67	7	12·7	143	42.0	267	117	110	4
Case E	—	—	—	—	42	220	75	37	220	110
Case A_I	1·56	0·83	6	2·8	190	6	339	159	95	19
Case B_I	1·18	0·93	5	8·8	208	·24	385	194	96	24
Case C_I	1·58	·86	5	7·3	216	—14	368	184	95	23
Case D_I	1·41	·99	5	-13·8	223	—38	388	183	102	32
Case E_I	7·18	·15	25	—	—1	263	— 2	—1	263	132 (Shear)

Note : 1. At the time the stability and stress calculations of the main dam were started the elevation of a possible siltation of the head pond was assumed to be 1200. The calculations were completed with this figure. However, as more silt data was made available and the problem of the head pond siltation studied more thoroghly, it was found that a silt elevation of 1230 would be more appropriate. For the most severe loading condition of the dam, that is case B_1 and D_1, the stability and the stresses have been recalculated with the 1230′ level for the silt and these later figures have been shown in the report. Since the difference in safety factors and stresses are relatively small all other cases have not been recalculated. Thus for the unimportant loading conditions the figures in the report are more accurate for a silt level at elevation 1200.

Note : 2. · All stability figures and stressess have been calculated with a/base length ignoring the bucket portion.

Table No. 7. Bulkhead section at El. 1130

Summary of results for various conditions of loading

Condition	Fs.	F.	Fs.	R = (E-D/3)	NT Lbs/□″	NH Lbs/□″	Principal Stresses. Max. comp Lbs/□″ Toe.	Shear Lbs/□″ Toe.	Max. comp Lbs/□″ Heel.	Min comp. Lbs/□″
CASE B.	2.5	·47	14	29·8	55	76	100	50	116	28
B_I.	1·96	·74	9·1	14·7	94	31	170	85	71	6
D	1·96	·59	12	21·2	49	70	96	49	115	24
D_I.	1·58	·90	8·1	3·2	113	7·5	220·8	102	67·5	7·6

Table No. 8, Stairwell Section at El. 1132

Summary of results for various conditions of loading

B	2·27	·53	8	23·6	61	53	111	55	93	16
B_I.	1·77	·81	9.4	7·8	97	17	175	87	57	2
D	1·87	·67	12	13·3	81	29	147	73	73	3
D	1·48	·99	18	-5·2	121	-11	218	109	68	17

PAPER NO 358

STRESS AND STABILITY ANALYSIS OF WARSAK DAM

TABLE A

COMPARISION OF STRESSES AND ASSUMPTIONS FOR WARSAK DAM AND SOME OF THE EXISISTING CONCRETE GRAVITY DAMS

DAM	HEIGHT TO FLOOD WATER SURFACE, FT.	WEIGHT OF CONCRETE lb PER CU FT		UPLIFT ASSUMPTION			TYPE OF HEEL CUTOFF	USE OF FOUNDATION DRAINS	ASSUMED ICE PRESSURE, lb PER LIN ft	ASSUMED SILT PRESSURE, EQUIVALENT LIQUID, IN ADDITION TO WATER PRESSURE	EARTH-QUAKE ASSUMPTION, PER CENT OF GRAVITY	SLIDING RESISTANCE			MAXIMUM COMPRESSIVE STRESSES, lb PER SQ. FT.					TYPE OF FOUNDATION	DISTANCE OF ASSUMED HIGH WATER FROM TOP OF DAM, ft	2 RADIUS OF CURVATURE, ft.	DISTANCE OF RESULTANT INSIDE EXTREMITY OF MIDDLE THIRD, PERCENT OF BASE WIDTH
		TEST	ASSUMED	PERCENT OF HEADWATER AT HEEL	PERCENT OF HEADWATER AT DRAINS	PERCENT OF TAIL WATER AT TOE						BASED ON FRICTION ALONE OR SHEARATION OR BOTH	FRICTION FACTOR, TAN Ø	SHEARING STRESS FRICTION NEGLECTED, lb PER SQ. ft	VERTICAL		INCLINED (PRINCIPAL)		CONDITIONS				
															HEEL	TOE	HEEL	TOE					
WARSAK (AUTHOR'S EXAMPLE)	230	152	145	66.7	-	66.7	GROUT	YES	NONE	X 28.2 h^2	15	BOTH	·67	26,350	5,460-VE 38,000+VE	32,200	36,000	56,000	WITH QUAKE	HARD GRANITE SCHIST	55 ABOVE CREST	STRAIGHT	6·5
GRAND COULEE	533	155	—	66.7	-	66.7	"	"	"	NONE	10	"	0·85	46,100	85,700	67,300	87,700	110,400	"	GOOD GRANITE	20	"	9·8
SHASTA	440	154	—	66.7	"	66.7	"	"	"	"	10	FRICTION	0·82	50,300	72,600	67,000	84,300	107,000	"	METAMORPHOSED	12·5	2,500 RADIUS	16·0
NORRIS	242	159	150	66.7	-	66.7	"	"	"	"	10	"	·63	"	33,700	28,000	35,000	41,600	NO QUAKE	GOOD DOLOMITE	18	STRAIGHT	0·0
MORRIS	329	155	150	100	50	100	"	"	"	"	10	"	·78	"	46,600	37,600	46,500	62,500	—	POOR GRANITE	—	"	—
HIWASSEE	2075		154	66.7	33.3	66.7	"	"	"	"	NONE	"	·63	"	36,700	34,700	38,200	53,600	"	QUARTZITE AND SCHIST	0	"	0·0

X DRY WT. OF SILT = 90 lbs/cu. ft

SP. GRAVITY = 2.7

BOUYANT WT. (WS) $= \frac{90 \times 1.7}{2.7}$ x 56.4 lbs/cu. ft.

SILT PRESSURE EQUIVALENT LIQUID = $\frac{WS}{2} \times h^2$

$= 28 \cdot 2 h^2$

PAPER NO 338

TABLE No; 4

CASE B1 (EXCLUDING UPLIFT)

SECTION EL.

VALUES OF PRINCIPAL STRESSES & MAX: SHEAR AT VERIOUS SECTIONS AT EL.....

STRESS AND STABILITY ANALYSIS OF WARSAK DAM

Section	P_n	P_b	$-q_b$	P_n+P_b	$\frac{P_n+P_b}{2}$ ①	P_n-P_b	$(P_n-P_b)^2$	$\frac{(P_n+P_b)^2}{4}$	q_b^2	$\frac{(P_n-P_b)^2}{4}+q_b^2$	$\sqrt{\frac{(P_n-P_b)^2}{4}+q_b^2}$ ②	I+II	I- II	$-2q_b$	$\frac{-2q_b}{P_n-P_b}$	α
U/S	137	5.7	0	19.4	9.7	-8.0	64	16	0	16	4	13.7	5.7	0	0	0
3/4	14.95	9.12	3.3	24.07	12.04	-5.83	34	8.5	10.9	19.4	4.4	16.44	7.64	-6.6	1.18	24.9°
1/2	16.21	12.54	6.6	28.75	14.38	-3.67	13.4	3.4	43.6	47.0	6.86	21.24	7.52	13.2	3.62	37.3°
1/4	17.46	15.95	9.9	33.41	16.70	-1.51	2.28	0.57	98	98.6	9.94	26.64	6.76	19.8	13.1	42.5°
0	18.71	19.37	13.2	38.08	19.04	0.66	0.43	0.11	174	174.1	13.2	32.24	5.84	26.6	-40.3	44.3°
-1/4	19.96	22.78	16.5	42.74	21.37	2.82	7.9	2.0	272	274	16.6	37.97	4.77	31.0	-11	42.0°
-1 1/2	21.21	26.20	19.8	47.4	23.70	5.00	25.0	6.25	391	397.3	19.9	43.60	3.6	39.6	-7.9	41.4°
-3/4	22.46	29.62	23.1	52.08	26.04	7.16	50.6	12.7	532	550	23.45	49.49	2.6	46.2	-6.45	40.6°
d/s	23.73	33.05	26.4	56.78	28.39	9.32	87	21.8	695	717	26.8	55.19	1.59	52.8	5.65	40.0°

PAPER NO 338

TABLE No. 2
CASE B (EXCLUDING UPLIFT)
SECTION

STRESS AND STABILITY ANALYSIS OF WARSAK DAM

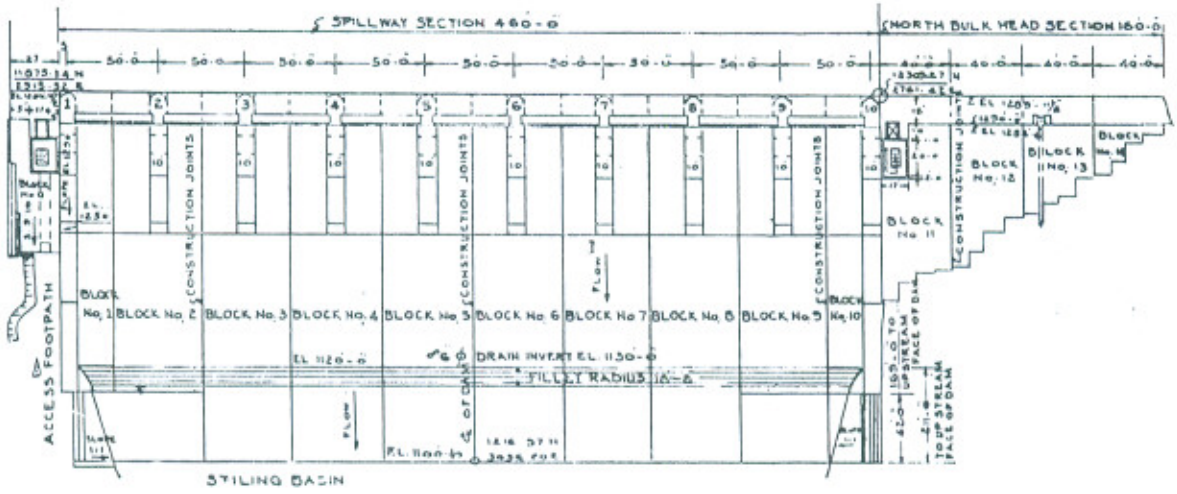
VALUES OF PRINCIPAL STRESSES & MAX: SHEAR AT VERIOUS SECTIONS AT EL. 1050

Section	P_n	P_b	$-q_b$	$P_n + P_b$	$\frac{P_n+P_b}{2}$ ①	$P_n - P_b$	$(P_n-P_b)^2$	$\frac{(P_n-P_b)^2}{4}$	q_b^2	$\frac{(P_n-P_b)^2}{4}+q_b^2$	$\sqrt{\frac{(P_n-P_b)^2}{4}+q_b^2}$ ②	I + II	I − II	$-2q_b$	$\frac{-2q_b}{P_n-P_b}$	α
U/S	13.7	14.8	0	28.5	14.25	− 1.1	1.21	.30	0	0.30	.548	14.8	13.7	0	0	0
3/4	14.18	15.95	− 2.36	30.13	15.065	−1.77	3.13	.75	5.58	6.33	2.52	17.59	12.55	4.72	2.67	34.7°
1/2	14.66	17.10	− 4.73	31.76	15.88	− 2.44	5.96	1.49	22.4	23.89	4.88	20.76	11.0	9.46	3.88	37.8°
1/4	15.14	18.25	− 7.09	33.39	16.695	− 3.11	9.67	2.42	50.16	52.58	7.25	23.95	9.45	14.18	4.55	38.8°
0	15.62	19.40	− 9.45	35.02	17.51	− 3.78	14.28	3.57	89.2	92.77	9.62	27.13	7.89	18.9	5.0	39.35°
− 1/4	16.1	20.55	− 11.81	36.65	18.325	− 4.45	19.8	4.95	139.9	144.85	12.03	30.36	5.3	23.62	5.3	39.65°
− 1/2	16.58	21.70	−14.17	38.28	19.14	− 5.12	26.2	6.55	201.0	207.55	14.4	33.54	4.74	28.34	5.51	39.85°
− 3/4	17.06	22.85	−16.53	39.91	19.95	−5.79	33.5	8.375	273.4	281.78	16.8	36.75	3.15	33.06	5.71	40.05°
D/S	17.54	24.0	−18.9	41.54	20.77	− 6.46	41.7	10.43	356.7	367.13	19.17	39.94	1.60	37.8	5.84	40.15°

PAPER NO 338

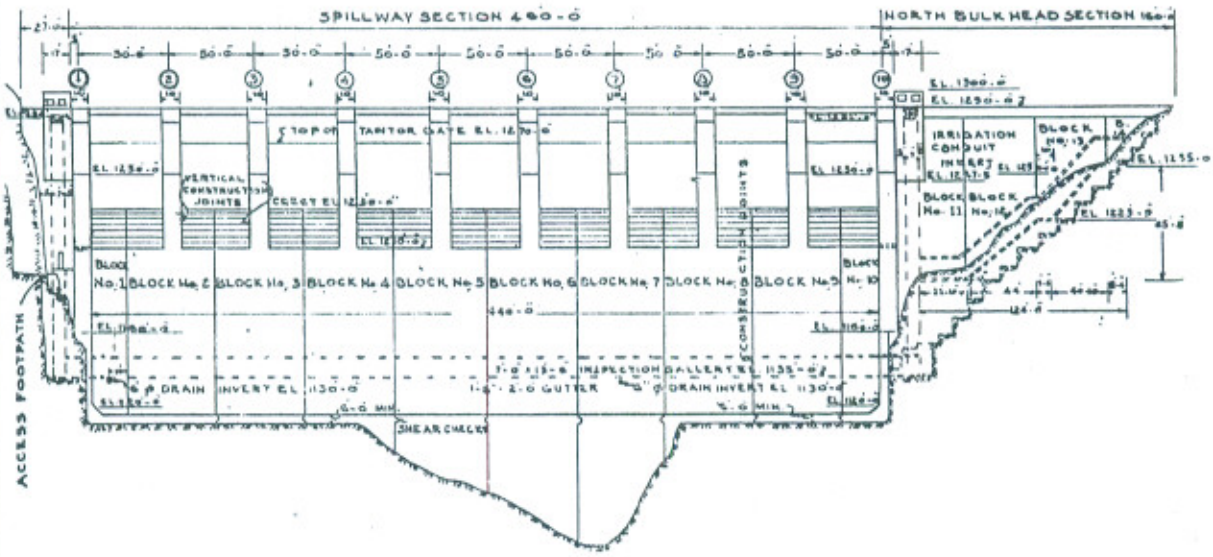
STRESS AND STABILITY ANALYSIS OF WARSAK DAM

PLATE No. 1



PLAN

SCALE:- 1" = 80



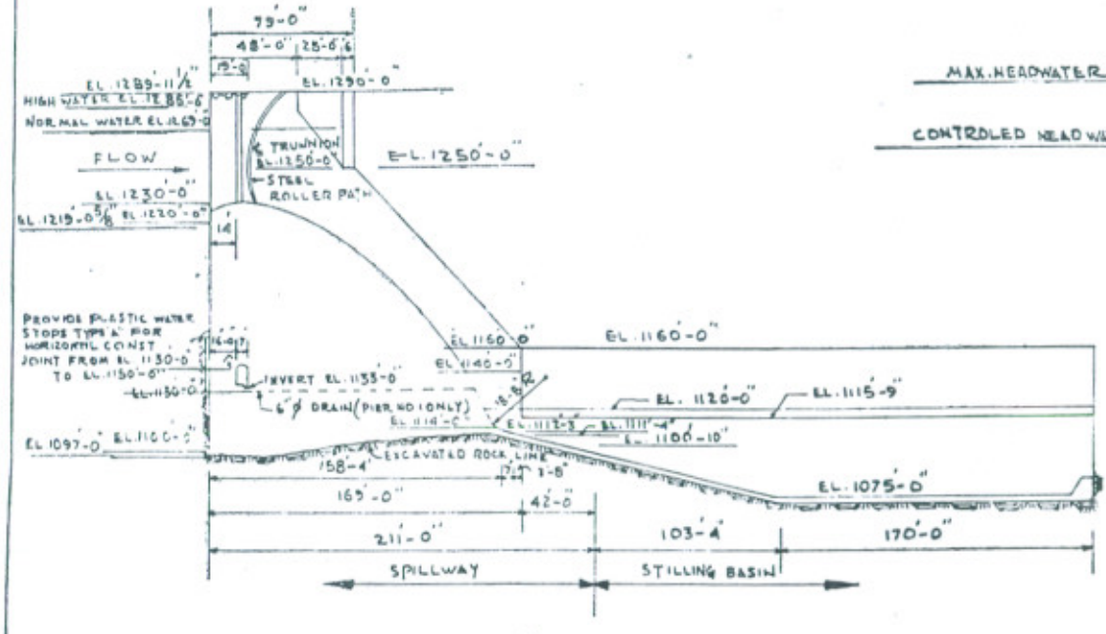
DOWN STREAM ELEVATION

SCALE:- 1"=80'

PAPER NO 338

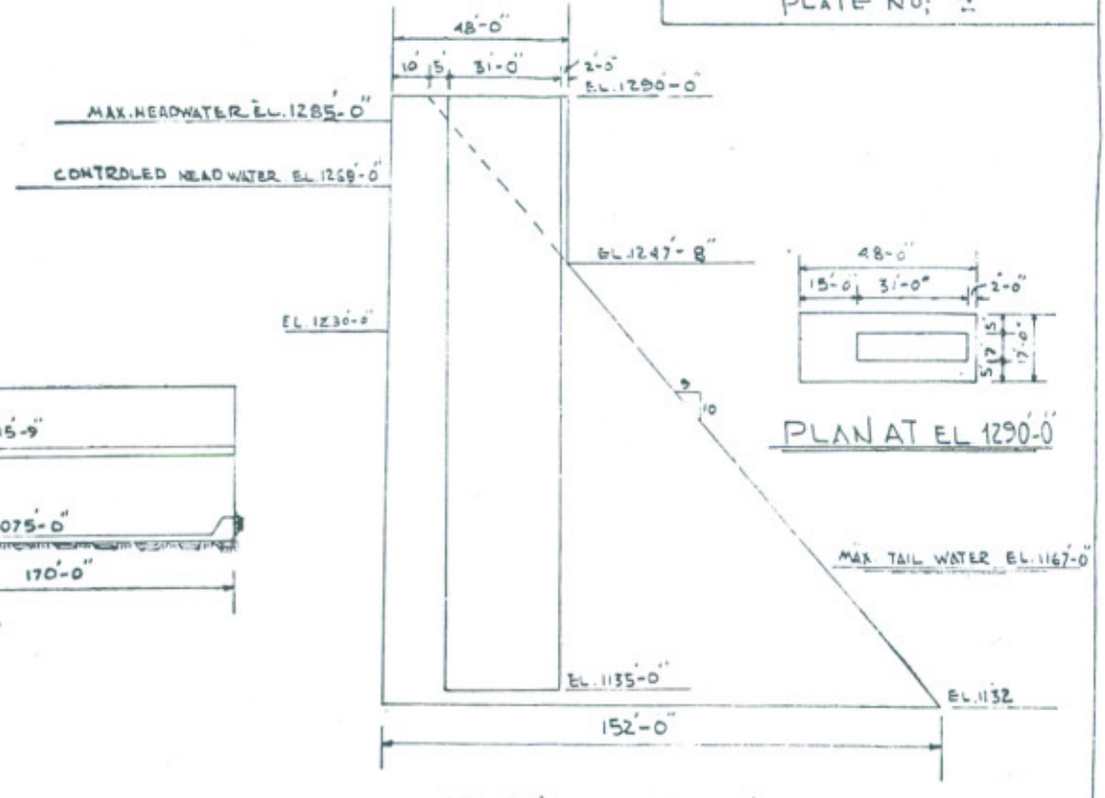
STRESS AND STABILITY ANALYSIS OF WARSAK DAM

PLATE NO; 2



SPILLWAY SECTION

SCALE 1"=80'



STAIRWELL SECTION

ASSUMED CROSS SECTION FOR STABILITY CALCULATION

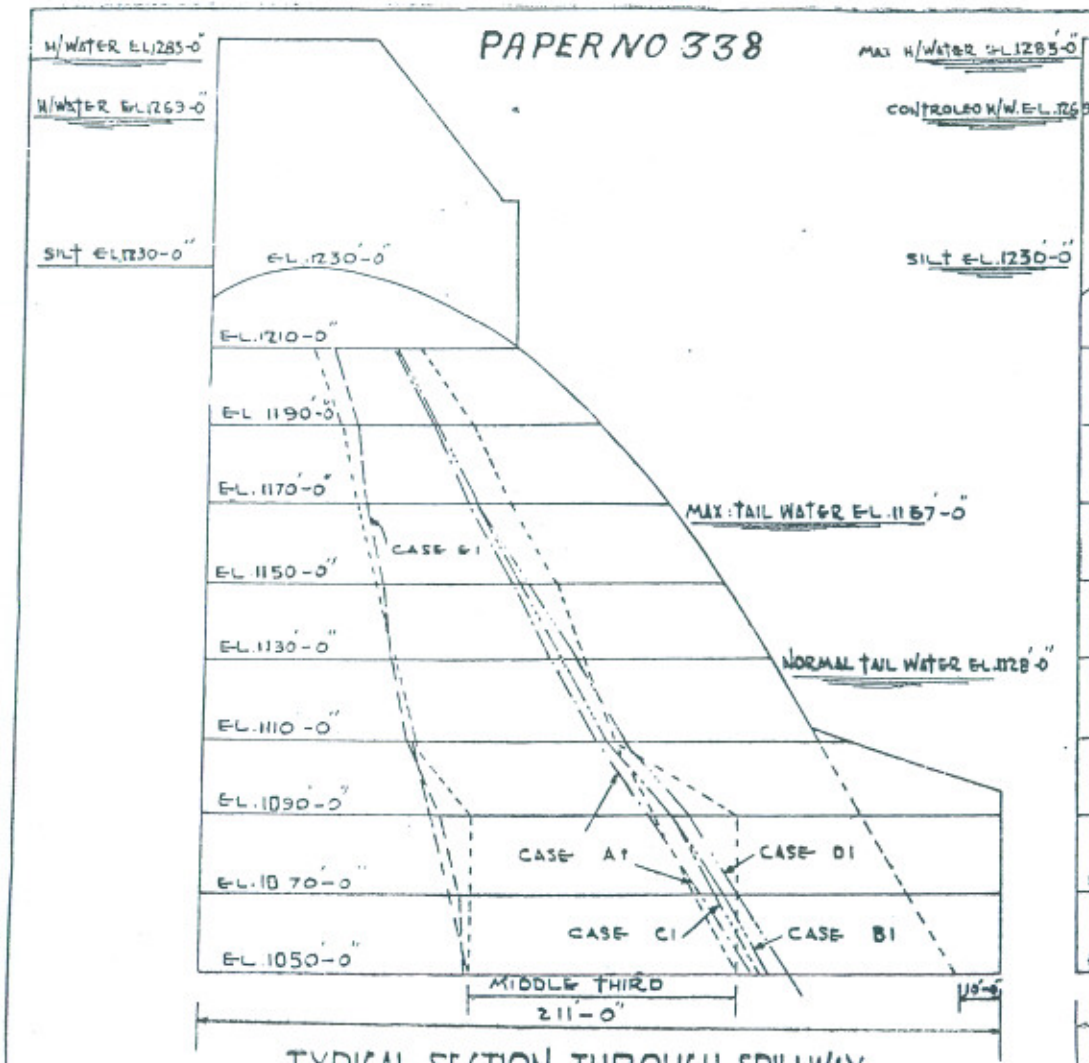
SCALE 1"=40'

TRACED:
ABDUL JALIL

PAPER NO 338

STRESS AND STABILITY ANALYSIS OF WARSAK DAM

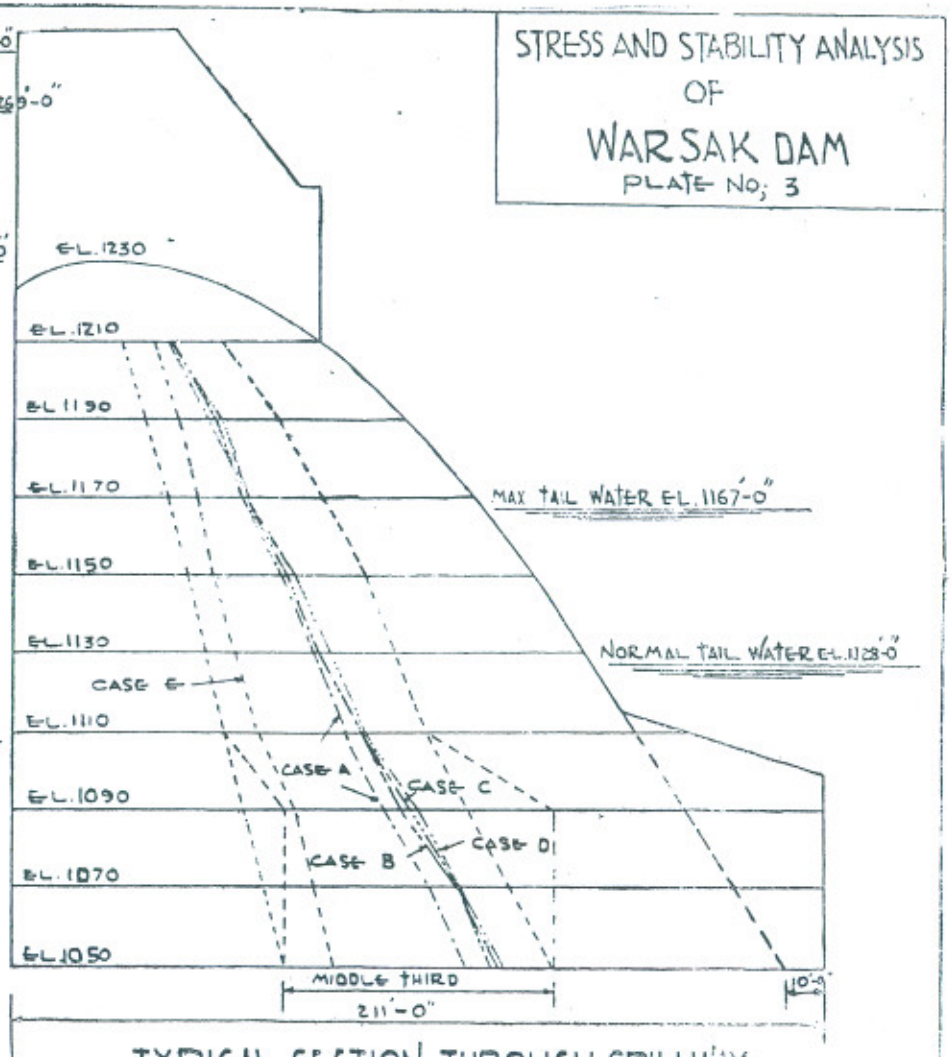
PLATE NO; 3



TYPICAL SECTION THROUGH SPILLWAY
SHOWING STABILITY CONDITIONS WITH EARTHQUAKE

SCALE 1"=40'

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ABDUL JALIL.

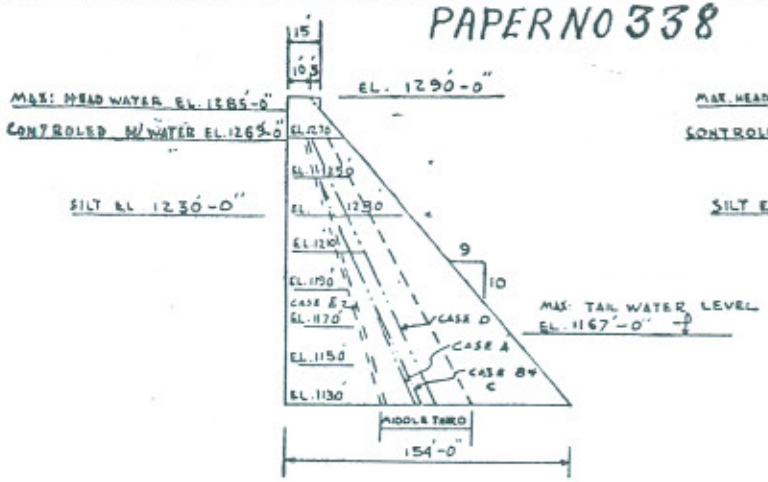


TYPICAL SECTION THROUGH SPILLWAY
SHOWING STABILITY CONDITIONS WITHOUT EARTHQUAKE

SCALE 1"=40'

PAPER NO 338

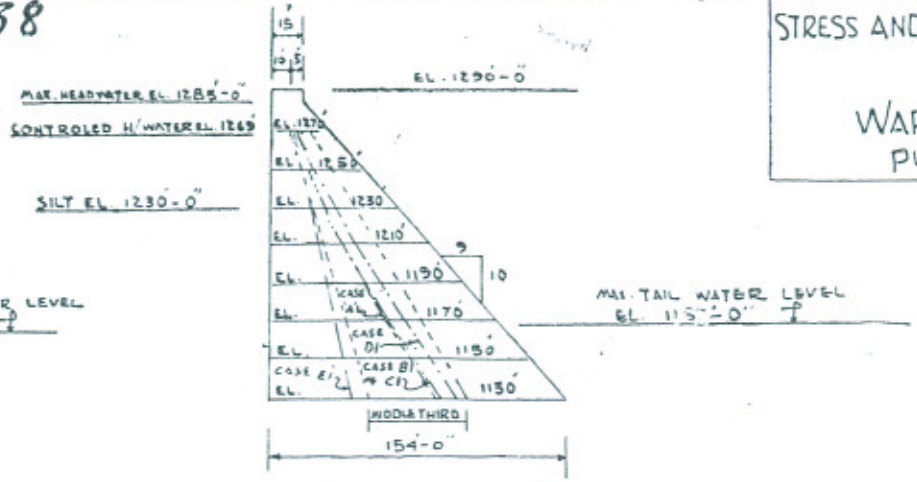
STRESS AND STABILITY ANALYSIS OF WARSAK DAM
PLATE NO; 4



STANDARD SECTION

STABILITY CONDITION WITHOUT EARTHQUAKE

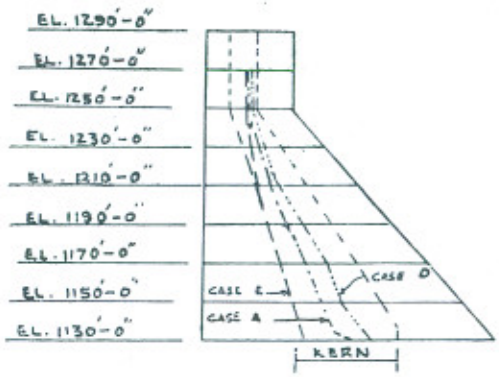
SCALE 1"=80'



STANDARD SECTION

STABILITY CONDITIONS WITH EARTHQUAKE

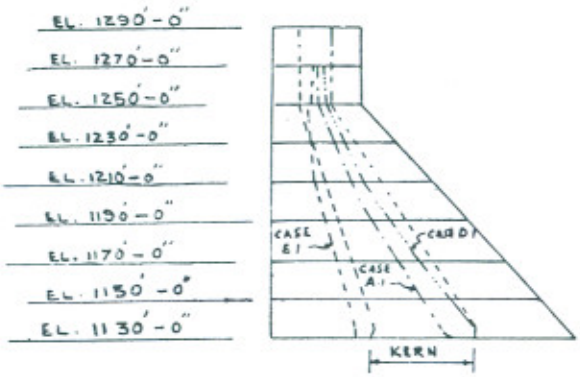
SCALE 1"=80'



STANDARD SECTION

STABILITY CONDITION WITHOUT EARTHQUAKE

SCALE 1"=80'



STAIRWELL SECTION

STABILITY CONDITION WITH EARTHQUAKE

SCALE 1"=80'

TRACED
ABDUL JALIL

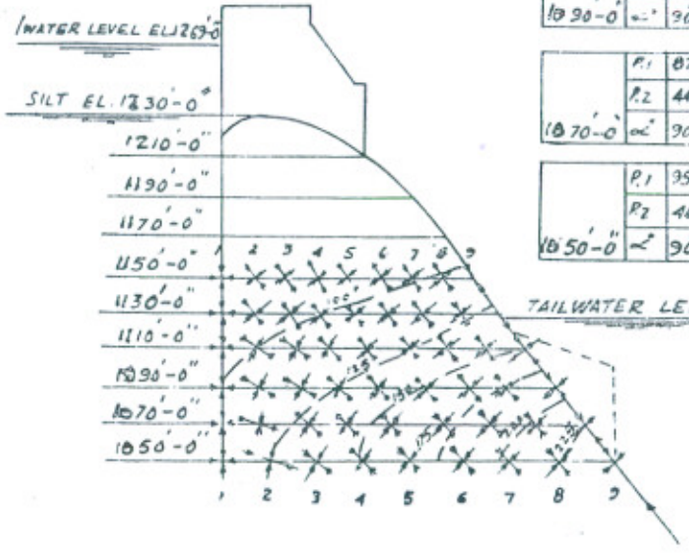
STRESSAND STABILITY ANALYSIS OF WARSAK DAM

PLATE-No; 5

PAPER NO 338

ELEV.		1	2	3	4	5	6	7	8	9
1150'-0"	P_1	52	54	57	61	65	68	72	76	80
	P_2	50	44	39	32	25	19	13	6	0
	$\alpha°$	0°	39°	36°	35°	34°	34°	34°	34°	34°
1130'-0"	P_1	60	62	61	74	80	86	92	98	103
	P_2	54	50	43	34	29	22	14	7	0
	$\alpha°$	90°	48°	39°	37°	35°	35°	34°	34°	34°
1110'-0"	P_1	69	69	76	82	90	98	107	116	124
	P_2	48	50	45	40	33	26	19	12	5
	$\alpha°$	90°	67°	50°	42°	39°	37°	35°	34°	34°
1090'-0"	P_1	78	81	90	102	113	126	139	151	161
	P_2	51	54	50	45	38	31	25	18	14
	$\alpha°$	90°	69°	55°	49°	45°	43°	41°	40°	40°
1070'-0"	P_1	87	91	102	116	132	148	165	183	200
	P_2	44	50	50	47	42	36	30	23	17
	$\alpha°$	90°	73°	59°	57°	47°	44°	42°	40°	40°
1050'-0"	P_1	95	100	110	126	143	162	180	198	218
	P_2	46	54	56	52	47	42	36	30	23
	$\alpha°$	90°	74°	66°	52°	47°	44°	42°	40°	40°

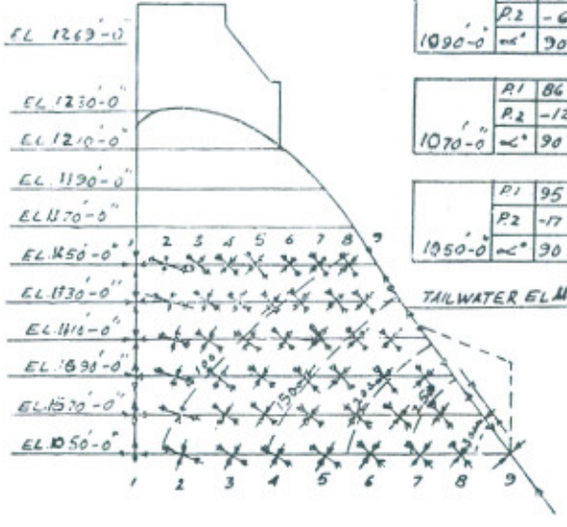
ELEV.		1	2	3	4	5	6	7	8	9
1150'-0"	P_1	51	52	57	66	76	96	96	107	117
	P_2	24	23	28	25	21	10	11	5	0
	$\alpha°$	90	74	55	46	43	38	36	35	35
1130'-0"	P_1	60	67	69	80	95	109	120	140	154
	P_2	19	27	29	27	22	17	9	6	0
	$\alpha°$	90	74	58	48	42	41	37	35	34
1110'-0"	P_1	67	70	78	92	109	128	146	165	186
	P_2	9	22	28	28	25	81	16	12	4
	$\alpha°$	90	77	62	50	44	40	37	35	34
1090'-0"	P_1	78	85	99	119	142	168	194	220	248
	P_2	-6	15	23	24	32	22	19	15	11
	$\alpha°$	90	78	65	55	49	46	43	41	40
1070'-0"	P_1	86	97	113	133	166	196	220	261	294
	P_2	-12	7	21	31	27	26	23	20	17
	$\alpha°$	90	78	66	56	50	46	43	42	40
1050'-0"	P_1	95	106	124	152	185	217	254	290	327
	P_2	-17	7	21	28	29	36	28	25	22
	$\alpha°$	90	79	67	57	51	46	44	42	40



SPILLWAY SECTION

CASE B- UPLIFT ALLOWANCE

SCALE 1" = 80'



SPILLWAY SECTION

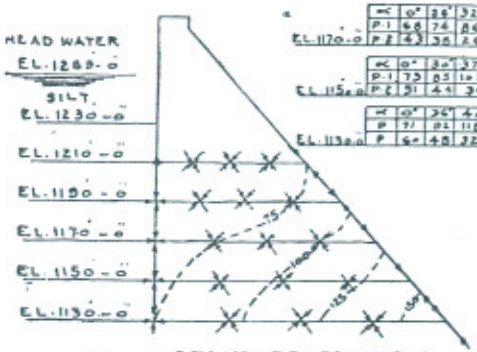
CASE B1 - WITH UPLIFT

SCALE 1" = 80'

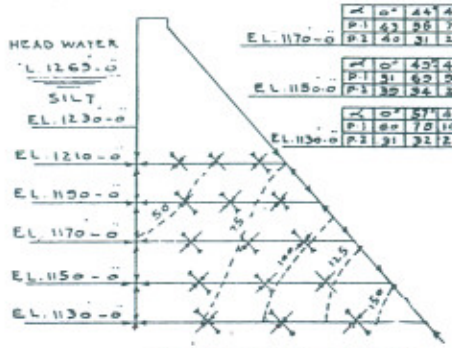
PAPER NO 338

STRESS AND STABILITY ANALYSIS OF WARSAK DAM

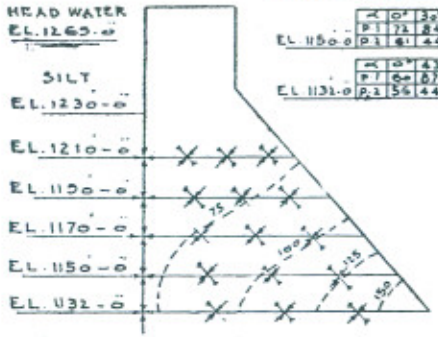
PLATE No; 6



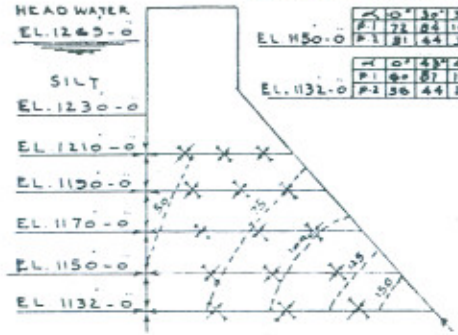
STANDARD SECTION
CASE B1 – NO UPLIFT
SCALE 1" = 80'



STANDARD SECTION
CASE B1 - UPLIFT ALLOWANCE
SCALE 1" = 80'



STAIRWELL SECTION
CASE B1- NO UPLIFT
SCALE - 1 = 80'



STAIRWELL SECTION
CASE B1- UPLIFT ALLOWANCE
SCALE - 1 : 80'