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Consideration on Designing Buttress Dams

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Part I

The Basic shape of Buttress for Buttress dams

(Excerpt of the originalpaper which appeared in transaction, ICOLD 1970)

The design criteria for buttress dams not common to other types of concrete dams are: 1) the vertical component of stress of the inclined upstream edge of each buttress should be kept in enough compression so that the second principal stress parallel to the edge never becomes tensile; 2) the allowable working compressive stress of buttress must not exceed the limit set to ensure safety against lateral buckling; 3) the buttress must be aseismic in its lateral direction. Of the above, 1) and 2) have a direct bearing on the shape of the buttress.

The design of gravity dams has two elements under consideration, the upstream and downstream faces of the dam. The buttress dams have one more element, the thickness of buttresses. In this respect, the latter type of dams has more degree of freedom which can be made adaptable to the requirements of efficient and safe design.

A buttress of (1) "uniform strength" has once been suggested to aim at a uniformity of the first principal stress. A concave or straight slope was given to the upstream face and the thickness of the horizontal sections was made thinner gradually toward the downstream face until it reached zero at the end,-a condition which gave rise to a possible buckling at the downstream side. Each buttress had to have so many longitudinal joints along the principal stress directions, otherwise the aimed behavior of the buttress was questionable.

The writer suggests herein a basic shape of buttress, in any horizontal section of which a uniform vertical component of stress and a constant factor of safety against shear are obtained, and a limitation of the vertical component of stress so as to ensure a necessary factor of safety against lateral bucking of the buttress. The basic shape of buttress has a parabolic curve at the upstream edge. The width and thickness increase in almost direct proportion to the height. The horizontal sections may be proportioned to add more lateral aseismicity. When the sections are analyzed by means of equivalent rectangles, the stresses can be formulated. The critical value of the vertical component of stress which causes buckling can be sought as a function of the span length center to center of buttress.

The basic shape is determined by water pressure and the weight of the dam. During an earthquake in an upstream and downstream direction, plus and minus dynamic stresses add to the basic stress. The hydrodynamic pressure in this case will be explained later on. The basic shape may require a partial modification due to the addition of stress resulting from loads on top of the buttress and slit pressure.

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Section 1

Basic Shape of Buttress

1.1 Curve of Upstream Shape (Refer to Fig. 1.1b)

The origin of the coordinate is taken at the apexdetermined on the design water level, measuring x vertically downward and y, y', y'' horizontally toward upstream. At any horizontal section of the buttress, the cross sectional area(including the effective areas added to the upstream and downstream ends, etc.) is denoted by A, tangential component load by T, normal component load by N, unit weight of water by ν, ditto of concrete by w; the ordinate of upstream edge of the buttress by y, ditto of point of application of N by y', ditto of center of gravity by y'', distance center to center of the buttress by L, the mean stress normal to the section by $\sigma_0=N/A$ and the ratio of tangential over normal componentby m, then

$$\frac{T}{N}=m \quad 1.10$$

$$dT=\nu Lxdx \quad 1.11$$

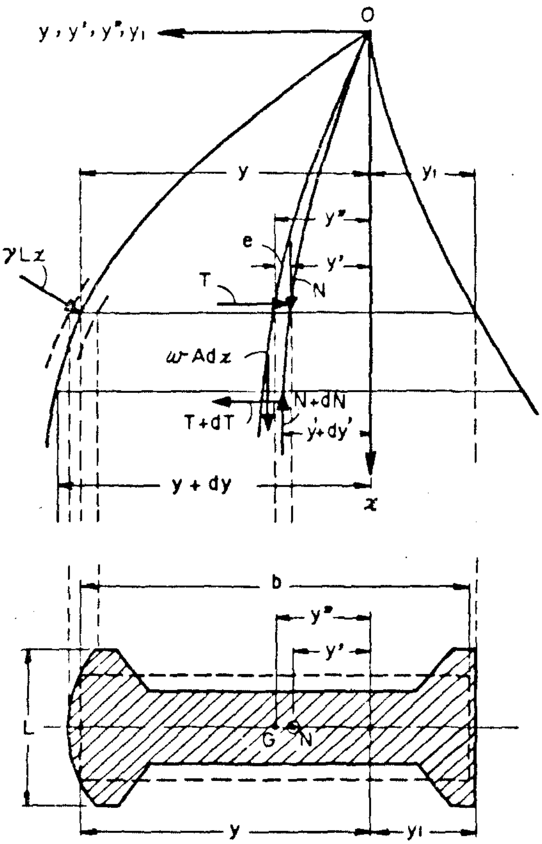

Fig. 1.1b Horizontal slice of buttress.

$$dN=\omega Adx+\nu Lxdy \quad 1.12$$

$$Tdx+d(Ny')-\nu Lxydy-wy''Adx=0 \quad 1.13$$

Assuming N and T are zero at $x=0$, then

From 1.11 $$T=\frac{\nu L}{2}x^2 \quad 1.14'$$

From 1.10 $$N=\frac{\nu L}{2m}x^2 \tag{1.15'}$$

From 1.12 $$\frac{dy}{dx}=\frac{1}{m}\left(1-\frac{w}{2\sigma_0}x\right) \tag{1.16'}$$

$$y=\frac{x}{m}\left(1-\frac{w}{4\sigma_0}x\right) \tag{1.18}$$

1.2 The Loci of the Center of Gravity and Point of Application of Normal Load in Any Horizontal Section

In order to rate the normal stress uniform in any horizontal section of the buttress, the locus of the center of gravity should coincide with that of the point of application of the normal load. Taking $y'=y''$ in the formula 1.13, and substituting 1.13 by 1.14, 1.15 and 1.16, it follows:

$$\frac{dy'}{dx}+\left(\frac{2}{x}-\frac{w}{\sigma_0}\right)y'+m-\frac{2}{m}\left(1-\frac{w}{4\sigma_0}x\right)\left(1-\frac{w}{2\sigma_0}x\right)=0 \tag{1.22}$$

$$y'=\frac{x}{2m}\left\{1-\frac{w}{2\sigma_0}x+(2m^2-1)\frac{-2e^{\frac{w}{\sigma_0}x}+\left(\frac{w}{\sigma_0}x\right)^2+2\left(\frac{w}{\sigma_0}x\right)+2}{\left(\frac{w}{\sigma_0}x\right)^3}\right\} \tag{1.23}$$

Note 1. In the original paper the writer included No, To and Mo as crest loads and moment to obtain more generalized solutions.

1.3 Horizontal Section Replaced by Equivalent Rectangle

The foregoing formulae 1.18 and 1.23 represent the shape of the upstream edge, and the locus of center of gravity of any horizontal section, thereby the shape of the section is left to one's choice. The component loads and sectional area defined by

$$T=\frac{\nu L}{2}x^2,\quad N=\frac{\nu L}{2m}x^2,\quad A=\frac{\nu L}{2m\sigma_0}x^2 \tag{1.30}$$

If the required sectional area A is replaced by an equivalent rectangle bt, it follows:

At the upstream edge

$$\overline{y}=\frac{x}{m}\left(1-\frac{w}{4\sigma_0}x\right) \tag{1.18}$$

At the center line of any section

$$y_{\prime}=\frac{x}{2m}\left\{1-\frac{\omega}{2\sigma_0}x+(2m^2-1)\frac{-2e^{\frac{w}{\sigma_0}x}+\left(\frac{w}{\sigma_0}x\right)^2+2\left(\frac{w}{\sigma_0}x\right)+2}{\left(\frac{w}{\sigma_0}x\right)^3}\right\}$$

$$=\frac{x}{2m}\left[1-\frac{w}{2\sigma_0}x-\frac{2m^2-1}{3}\left\{1+\sum_{n=1}^{\infty}\frac{3}{\lfloor n+3}\left(\frac{w}{\sigma_0}x\right)^n\right\}\right] \tag{1.23'}$$

where $\lfloor n+3=1.2.\dots n.\ \dots n+3$

For the width

$$\overline{b}=\frac{x}{m}\left[1+\frac{2m^2-1}{3}\left\{1+\sum_{n=1}^{\infty}\frac{6}{\lfloor n+3}\left(\frac{wx}{\sigma_0}\right)^n\right\}\right] \tag{1.31}$$

At the downstream edge

$\overline{y}_1=\overline{y}-\overline{b}$

$$=-\frac{x}{m}\left\{\frac{w}{4\sigma_0}x+\frac{1}{3}(2m^2-1)\left\{1+\sum_{n=1}^{\infty}\frac{6}{\lfloor n+3}\left(\frac{w}{\sigma_0}x\right)^n\right\}\right\} \tag{1.32}$$

For the thickness

$$\overline{t}=\frac{A}{\overline{b}}=\frac{\frac{\nu L}{2\sigma_0}x}{1+\frac{2m^2-1}{3}\left\{1+\sum_{n=1}^{\infty}\frac{6}{\lfloor n+3}\left(\frac{w}{\sigma_0}x\right)^n\right\}} \tag{1.33}$$

In the formula 1.18, the upstream edge becomes vertical when $x=H_0=2\sigma_0/w$, which shows that the buttress has substantially reached its maximum height. Denoting x/H_0 by ξ, it follows:

$$1+\sum_{n=1}^{\infty}\frac{6}{\lfloor n+3}\left(\frac{w}{\sigma_0}x\right)^n=1+\frac{1}{2}\xi+\frac{1}{5}\xi^2+\frac{1}{15}\xi^3+\cdots\cdots$$

If the value of σ_0 which will be mentioned in Sec. 4 is referred to, $H_0=150\sim380$m for $L=30\sim90$m.

In ordinary dams, ξ is considerably smaller than unity, therefore terms higher than ξ^2 can be neglected for the first approximation and for the ordinary value of $m<1$ and close to $1/\sqrt{2}$, the second approximation may hold. Finally

$$\bar{b}\doteqdot\frac{2}{3m}(1+m)^2x \qquad 1.312$$

$$\bar{t}\doteqdot\frac{3\nu Lx}{4\sigma_0(1+m^2)} \qquad 1.332$$

$$\bar{y}_1\doteqdot\frac{x}{3m}\left(2m^2-1+\frac{3}{4}\frac{w}{\sigma_0}x\right) \qquad 1.322$$

$$\bar{y}'\doteqdot\frac{x}{3m}\left(2-m^2-\frac{3}{4}\frac{w}{\sigma_0}x\right) \qquad 1.232$$

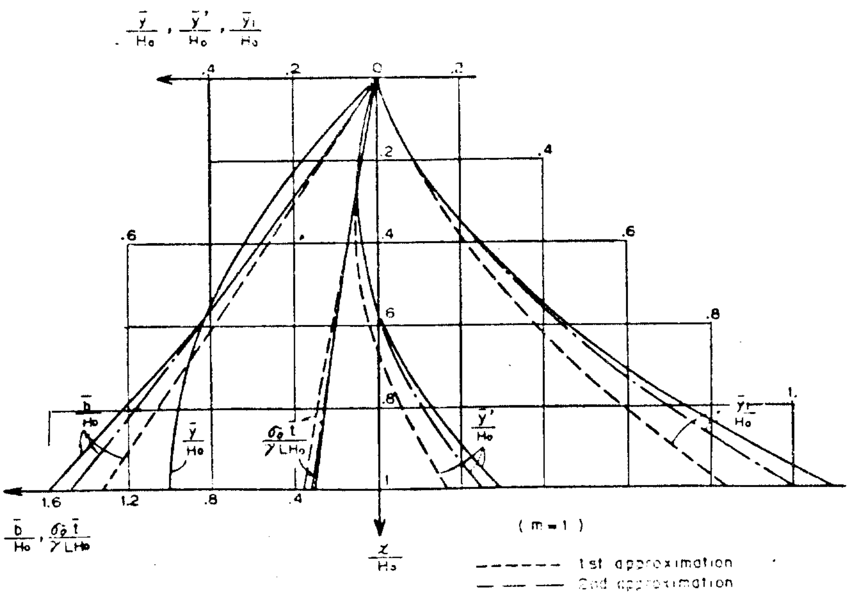

Fig. 1.5a Basic shape when $m=1$.

1.4 Evaluation of m

Denoting the shearing strength of concrete by τ, its coefficient of internal friction by f, and a safety factor by n, we have:

$$(\tau A+fN)/n\geq T$$

hence

$$\therefore\ m\geq\frac{1}{n}\left(\frac{\tau}{\sigma_0}+f\right) \qquad 1.41$$

1.5 Definition of the Basic Shape of Buttress and Its Graphing

Let us call the shape of buttress defined by y, $y'=y''$, σ_0 and m the basic shape of buttress. In graphing this, $2\sigma_0/w=H_0$ is regarded as a module, and therefore x/H_0, $\bar{y}/H_0$, $\bar{y}'/H_0$, $\bar{y}_1/H_0$, $\bar{b}/H_0$, $\sigma_0\bar{t}/\nu LH_0$ are used in Fig. 1.5 a,b.

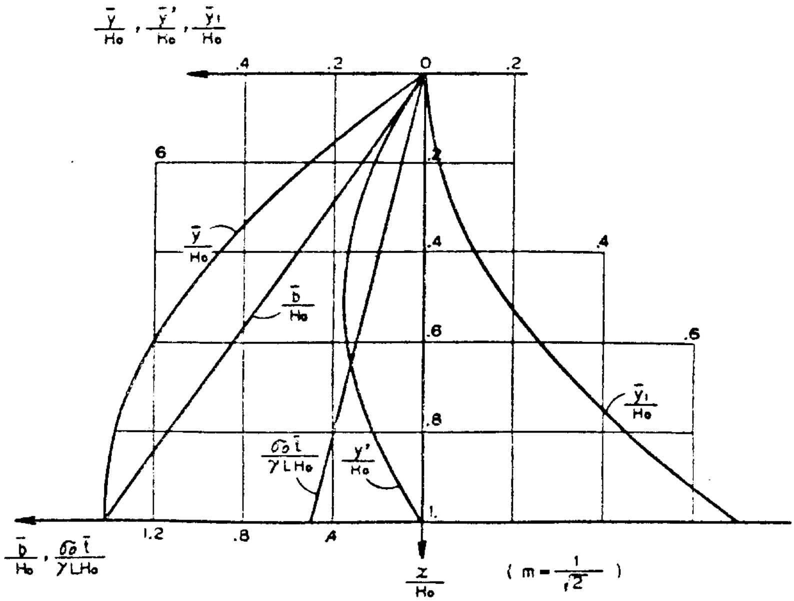

Fig. 1.5b Basic shape when $m=1/\sqrt{2}$.

Section 2

A Shape of Buttress in Which Vertical Component Load Has a Fixed Eccentricity

(omitted)

Section 3

Distribution of Stress in The Basic Shape

(Fig. 3.1)

3.1 Stress of x and y Directions

Consideration is given to a buttress with equivalent rectangular section bt, and at $x=0$, N and T are zero. Denote the component stresses in x and y directions by σ_x and σ_y respectively, and shearing stress by τ. Let us assume σ_x does not vary in y direction. The loads are due to the water pressure ν_x, and unit weight of concrete$w$.

Theu, it follows:

$$\frac{d}{dx}(\sigma_x t)-\frac{\partial}{\partial y}(\tau t)-wt=0 \qquad 3.11$$

$$\frac{\partial}{\partial y}(\sigma_y t)-\frac{\partial}{\partial x}(\tau t)=0 \qquad 3.12$$

we get

$$\frac{d}{dx}(\sigma_x tb)-wtb-\nu Lx\frac{dy}{dx}=0 \qquad 3.15$$

If $A=bt$, $\sigma_x=\sigma_0$ in the equation 3.15, it agrees to the equation 1.12 and y becomes $\overline{y}$. Insert $b=2(y-y'')$ and $y=\frac{1}{m}\left(1-\frac{w}{4\sigma_0}x\right)$ in 3.14, and it follows:

$$\frac{dy'}{dx}+\left(\frac{2}{x}-\frac{w}{\sigma_0}\right)y''+m-\frac{2}{m}\left(1-\frac{w}{2\sigma_0}x\right)\left(1-\frac{w}{4\sigma_0}x\right)=0 \qquad 3.16$$

which agrees to y' in 1.22. Therefore $\sigma_x = \sigma_0$ in the basic shape. Further, measuring η horizontally from the upstream edge

$$\tau t = -(\sigma_0 \dot{t} - wt)\eta + \tau_u t, \text{ where } \dot{t} = \frac{dt}{dx}$$

$$\therefore\ \tau = (\tau - \tau_u)\frac{\kappa}{b} + \tau_u \qquad 3.17$$

$$\text{where } \tau_u = \left(\frac{\nu L x}{t} - \sigma_0\right)\tan\theta_u,\ \ \tau_d = \sigma_0 \tan\theta_d,$$

with the suffix u for upstream and d for downstream edges,

Therefore τ presents astraight line. From 3.11 and 3.12 it follows:

$$\frac{\partial^2(\tau t)}{\partial x\ \partial y} = \frac{\partial^2(\sigma_y t)}{\partial y^2} = \sigma_0 \ddot{t} - w\dot{t}$$

At the upstream edge

$$\sigma_{yu} = \frac{\nu L x}{t} - \tau_u \tan\theta_u$$

and at the downstream edge

$$\sigma_{yd} = \tau_d \tan\theta_d$$

In the same way

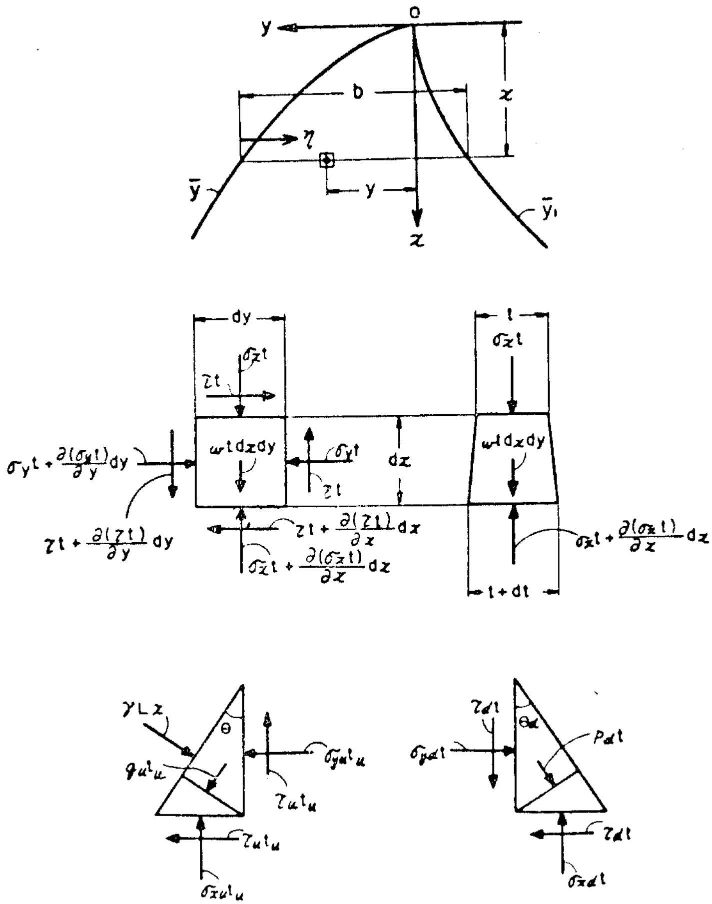

Fig. 3.1 Parallelepiped and prisms out of buttress.

$$\sigma_y=\sigma_{yu}-(\sigma_{yu}-\sigma_{yd})\frac{\eta}{b}-\frac{\sigma_0\ddot{t}-w\ddot{t}}{2t}(b-\eta)\eta \qquad 3.181$$

σ_y presents a parabolic distribution.

In the above

$$\theta_0\ddot{t}-w\ddot{t}$$

$$\doteqdot-\frac{9\nu Lw}{16\ \sigma_0}(1+m^2)\frac{1+2m^2+\dfrac{2m^2-1}{6}\ \dfrac{w}{\sigma_0}x}{\left(1+m^2+\dfrac{2m^2-1}{8}\ \dfrac{w}{\sigma_0}\ x\right)^3} \qquad 3.182$$

$$\doteqdot-\frac{9\nu Lw}{16\ \sigma_0}\ \frac{1+2m^2}{(1+m^2)^2} \qquad 3.182'$$

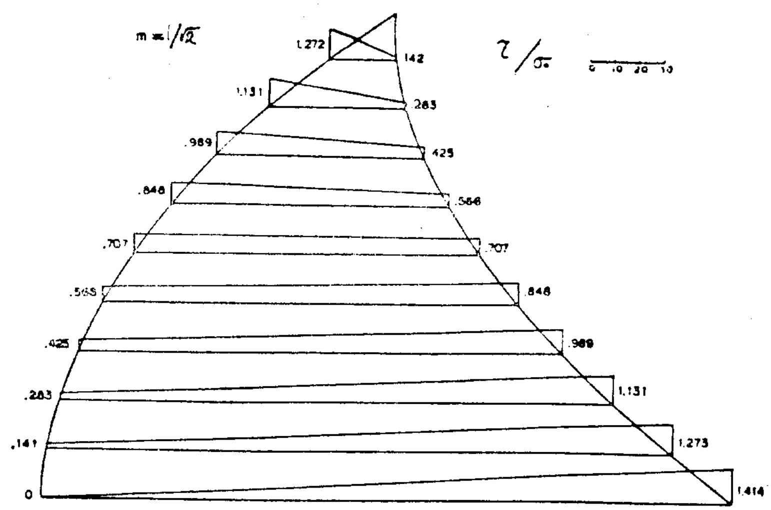

Fig. 3.11 τ/σ_0 for $m=1/\sqrt{2}$

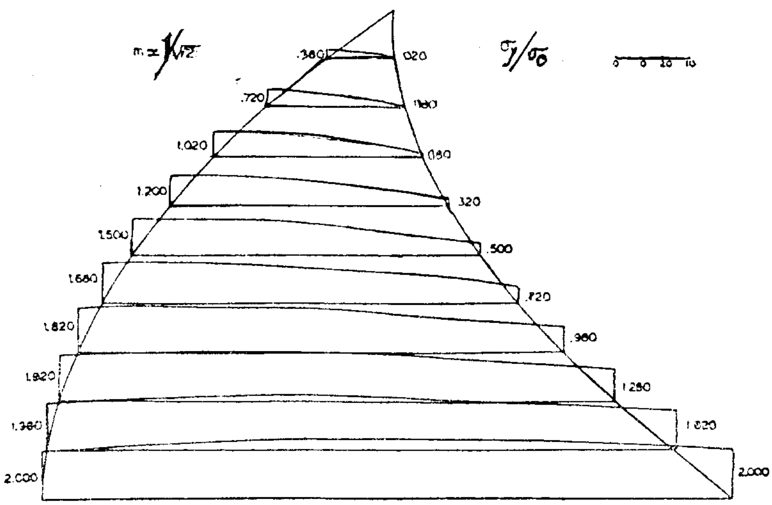

Fig. 3.12 σ_y/σ_0 for $m=1/\sqrt{2}$

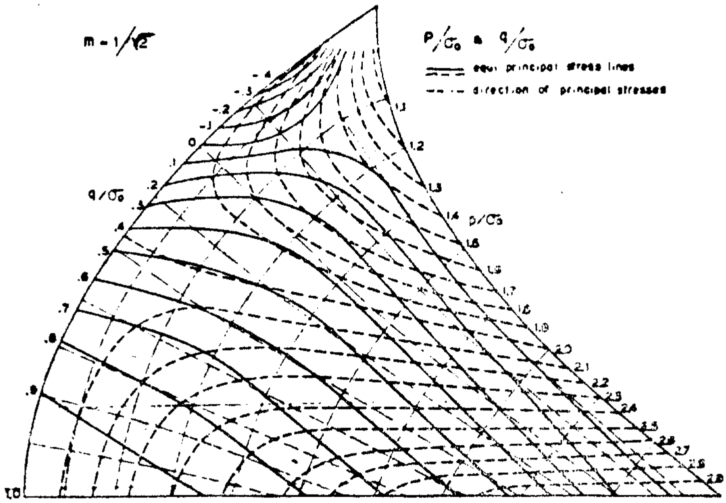

Fig. 3.2a Principal Stresses p/σ_0 and q/σ_0 for $m=1/\sqrt{2}$.

3.2 Principal Stresses

Denote the principal stresses by p, q and the maximum shearing stress τ_m, then

$$p, q=\frac{\sigma_x+\sigma_y}{2}\pm\tau_m \quad 3.21$$

$$\tau_m=\frac{1}{2}\sqrt{(\sigma_x-\sigma_y)^2+4\tau^2} \quad 3.22$$

At the upstream edge, let the thickness of buttress head be t_u,

$$p_u=\frac{\nu Lx}{t_u} \quad 3.23$$

$$q_u=\sigma_0-\tau_u,\ \tan\theta_u=\sigma_0\sec^2\theta_u-\frac{\nu Lx}{t_u}\tan^2\theta_u \quad 3.24$$

At the downstream edge,

$$p_d=\sigma_0\ \sec^2\theta_d,\ q_d=0 \quad 3.25$$

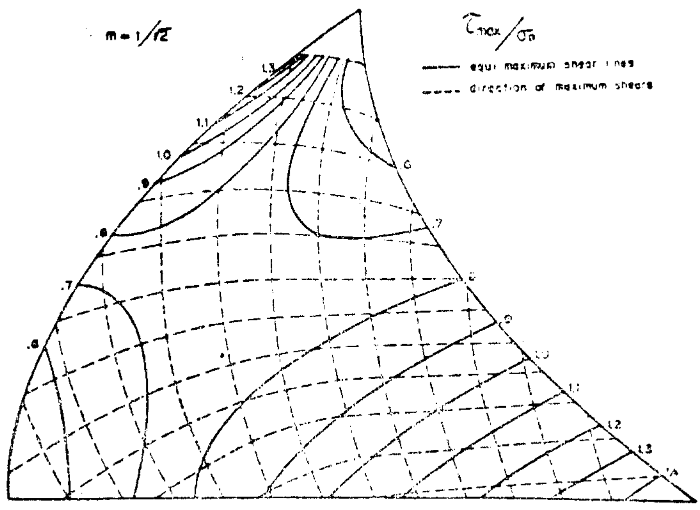

Fig. 3.2b τ_{max}/σ_0 for $m=1/\sqrt{2}$

For the equivalent rectangular section, and when $m=1\sim 1/\sqrt{2}$, $p_d=(5.2\sim 3.0)\ \sigma_0$ at H_0. In order to keep $q_u \geq 0$, the thickness of the buttress head t_u shall be larger than the mean thickness $\bar{T}$ for a certain distance measured vertically from the top of the buttress, so that:

$$\frac{t_u}{\bar{t}} \geq \frac{4}{3}\frac{(1+m^2)\left(1-\frac{w}{2\sigma_0}x\right)^2}{m^2+\left(1-\frac{w}{2\sigma_0}x\right)^2} \geq 1 \qquad 3.26$$

$$\therefore \frac{wx}{2\sigma_0} \leq 1-\sqrt{\frac{3m^2}{1+4m^2}} \qquad 3.27$$

Section 4

Buckling Due to Compressive Stresses

4.1 In Case the Whole Buttress is Regarded as a Vertical Column (Fig. 4.1)

Generally, the buttress is protected from buckling by being provided with a deck wall at the upstream edge, a flange at the downstream edge, or struts and crest beams, or its structural merits of double buttress or of hollow construction. In this section, each buttress is regarded as an independent standing wall. The buttress is assumed to have rectangular horizontal cross sections, Thus,

$$\left.\begin{aligned} &b=\frac{2}{3}\frac{(1+m^2)}{m}x, \qquad t=\frac{3\nu Lx}{4\sigma_0(1+m^2)} \\ &y=\frac{x}{m}\left(1-\frac{w}{4\sigma_0}x\right), \\ &y_1=y-b=-\frac{x}{3m}\left(2m^2-1+\frac{3w}{4\sigma_0}x\right) \end{aligned}\right\} \qquad 4.1$$

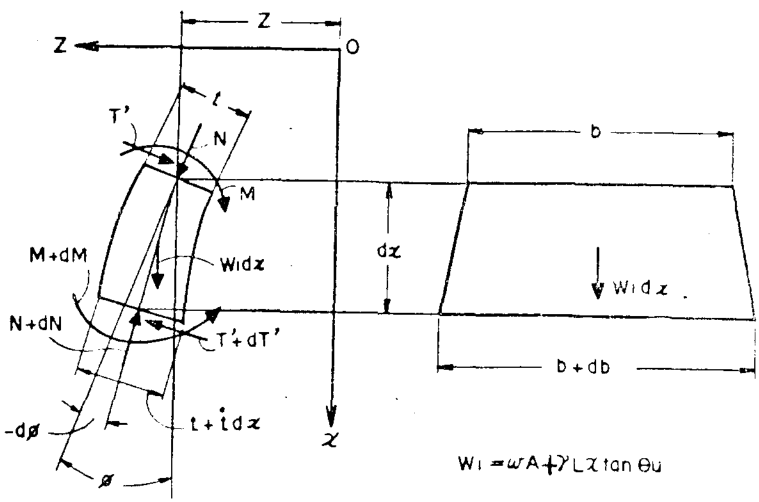

Fig. 4.1 Buckling of full width buttress.

Denoting the vertical component load by N and lateral shearing component force as it bends by T', the flexural deviation by Z, the flexural angle by ϕ, the bending moment by M, we obtain

$$\frac{dT'}{dx}=\frac{d}{dx}(N\phi) \qquad 4.10$$

Let $T'=0$ where $N=0$ and it follows:

$$\therefore\ T'=N\phi=\frac{\nu L}{2m}x^2\phi=\frac{dM}{dx} \quad \frac{d\phi}{dx}=\frac{M}{EI} \qquad 4.101$$

$$I=\frac{bt^3}{12}=\alpha x^4,\quad \alpha=\frac{3\nu^3L^3}{128m(1+m^2)^2\sigma_0{}^3} \tag{4.11}$$

Hence we obtain

$$x\fallingdotseq 0,\quad x^2\frac{d^2\phi}{dx^2}+4x\frac{d\phi}{dx}+\frac{\nu L}{2m\alpha E}\phi=0 \tag{4.12}$$

1) In case $\frac{9}{4}-c>0$ $c=\frac{\nu L}{2m\alpha E}$

Substitutions are made as follows:

$$x=e^{t'},\ \mu_{1,2}=-\frac{3}{2}\pm\sqrt{\frac{9}{4}-c} \tag{4.13}$$

We obtain

$$Z=\frac{A_1}{\mu_1+1}e^{(\mu_1+1)t'}+\frac{A_2}{\mu_2+1}e^{(\mu_2+1)t'}+A_3 \tag{4.142}$$

2) In case $\frac{9}{4}-c<0$

Denoting

$$\tau=\sqrt{c-\frac{9}{4}} \tag{4.15}$$

We obtain

$$Z=\frac{e^{-\frac{t'}{2}}}{\frac{1}{4}+\tau^2}\left[-A_1\left(\frac{1}{2}\ \sin\tau t'+\tau\cos\tau t'\right)+A_2\left(-\frac{1}{2}\cos\tau t'+\tau\sin\tau t'\right)\right]+A_3 \tag{4.162}$$

For the combination of two boundary conditions either ϕ or $d\phi/dx$ being zero at any two consecutive points t_1' and t_2', the determinant of the coefficients A_1 and A_2 shall be zero, hence we obtain $c=9/4$ in both cases 1) and 2). Replacing L/σ_0 by L_c/σ_c in 4.11 for distinction,

$$\frac{9}{4}=\frac{\nu L}{2m\alpha E}=\frac{64(1+m^2)^2}{3\nu^2L_c{}^2E}\sigma_c{}^3$$

The above formula shows that σ_0 grows to its critical value σ_c when the span length L grows to L_c, the critical value σ_c being valid where E remains constant.

$$\sigma_c=\frac{3}{4}\left[\frac{\nu^2E}{4(1+m^2)^2}\right]^{\frac{1}{3}}L_c^{\frac{2}{3}} \tag{4.17}$$

$$N_c=\frac{\nu x^2}{2m}L_c \tag{4.17'}$$

With $E=2.1\times10^6\ t/m^2$ for concrete, $\nu=1\ t/m^3$, and factor of safety of buckling$=F$, we obtain

$$\sigma_0=KL^{\frac{2}{3}},\ K=60.4\left[\frac{1}{(1+m^2)^2F^2}\right]^{\frac{1}{3}} \tag{4.171}$$

In this case, the load N is directly proportional to L, therefore, the safe load should be defined on the basis of L/F.

4.2 In Case the Buttress is Regarded as a Plate Restrained Along Periphery (Fig. 4.2)

(omitted)

Note 2.

In this paragraph the writer obtains the following formula for a long column with unit width.

$$\sigma_{xc}=K'\ L_c^{\frac{2}{3}},\ K'=46\frac{1}{(1+m^2)^{2/3}} \tag{4.211}$$

which corresponds to 4.171 when $F=1$,

While, he finds that by restraining the periphery of the buttress the values of $\overline{\sigma}_{xc}/\sigma_{xc}$ are

$H/H_c=\frac{wx}{2\sigma_0}$		0	.5	1
$m=$	1	10.5	4.57	2.58
	$1/\sqrt{2}$	35.2	5.21	2.81

It shows $\bar{\sigma}_{xc}/\sigma_{xc}>3$ except when H draws near $H/H_0=\frac{wx}{2\sigma_0}=1$. It enables us to expect more than $46\times3^{\frac{2}{3}}=96$ for K in the formula 4.171. Therefore restraining the periphery of the buttress increases the value of $F^{\frac{2}{3}}$ in 4.171 of the previous case (Para. 4.1) over 59 percent when the same value of σ_0 is used for proportioning the shape of buttress.

4.3 Double Buttress Structure

Provided a buttress of thickness t is split in a double leaf structure, the section modulus of the double leaves can be larger than that of a single leaf only when their center spacing distance j exceeds $t/2$. However, the structure shall be provided with vertical diaphrams in moderate spaces, resembling the hollow structure. Otherwise the isolated leaf of $t/2$ thickness will carry the design load for $L/2$, which is oaviously irrational by the fact that the critical loading stress is proportioned $L^{2/3}$.

4.4 Span Length of Buttress

(omitted)

Section 5
Application of the Basic Shape

5.1 Upstream Edge of the Basic Shape (Fig. 5.1)

When the face of the deck wall is a plane, the plane itself forms the upstream edge of the basic shape. When the upstream face has horizontally curved sections like a multi arched deck wall, the lines drawn parallel to the dam axis through the centers of gravity of the vertical component of water pressure on horizontal unit lamina of arch decide the upstream edge of the buttress. For example, a deck wall horizontally circular arched is taken herein. Denoting the subtend angle measured from the crown by θ_1, radius by r and the distance from the center of arch to the point of application of the resultant vertical water pressure exerted on the surface of the lamina by η_G, it follows:

$$\eta_G=\frac{r}{2}\left(\frac{\theta_1}{\sin\theta_1}+\cos\theta_1\right) \qquad 5.1$$

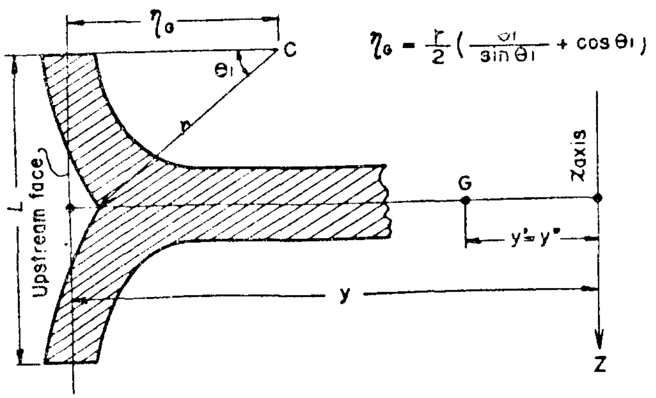

Fig. 5.1 Theoretical front of multi-arched buttress

If a deck wall is circular arched normal to the upstream edge described by y, the above distance is given approximately by

$$\eta'_G = \eta_G \sec\theta_u$$

5.2 Horizontal Cross Section of Buttress

The part of horizontal sections of the deck walls, round headed, flat or arched along the upstream edge, the overflow deck or flanges along the downstream edge, intermediate walls or diaphrams as stated in (4.3), which may transmit effectively the vertical component stresses are included in the effective sectional area A mentioned in Sec. 1. The rectangular section equalizing $A=bt$, $y-y''=b/2$, and $y_1=y-b$, has been used as a measure of determining σ_0 and L, and to predict the stress condition. Final evaluations shall be exercised on the actual form of the computed cross sectional area A.

5.3 Sizing the Crest of the Dam

The crest of buttresses must be shaped with suitable dimensions in width and thickness and be connected by beams for the safety of earthquake and other reasons. These crest loads will cause unbalanced stress distribution to some extent x_0 below the waterlevel. If N_0 exceeds T_0/m, the stress due to the excessive load shall be added to σ_0 below x_0, but its influence will become negligible as x increases. Elimination of excessive load by perforating buttress at a portion near the crest is possible. In case of the overflow type, the stress disturbance near the crest will be reduced by biasing the starting point of flow line toward the upstream side. Weight and resistant force caused by jet shall be taken into consideration for forming overflow deck.

5.4 Stress due to Silt Pressure (Fig. 5.4)

(omitted)

Note 3

The original paper deals with the two cases below.

1) When the silt level is planned pretty low, y for the upstream surface need not be altered. The deviation of y' to y'' below the silt level is formulated so as to obtain a slightly varied distribution of σ_x.
2) When the silt level is designed considerably high, necessary modification of y below the silt level is formulated, so as to make y' coincide with y''. The writer implies that the modification of the horizontal cross sectional areas A below the silt level, that is to determine their y'', may be attained easier by means of graphical solution.

5.5 Stresses due to Earthquake in the River Flow Direction

The horizontal inertia force of the own weight above x due to seismicity coefficient $\pm K$ is

$$T' = \frac{Kw\nu L}{6m\,\sigma_0} x^3 \qquad 5.51$$

The point of application of T' is $x/4$ high above the horizontal cross section at x. Denoting the vertical component stress due to the inertia by σ_e, the stresses at the up and downstream edges are

$$\sigma_{eu} = \mp \frac{Kw\nu L}{24m\,\sigma_0} \frac{x^4}{I_z}(y-y'') \qquad 5.511$$

$$\sigma_{ud} = \pm \frac{Kw\nu L}{24m\,\sigma_0} \frac{x^4}{I_z}(b-y-y'') \qquad 5.512$$

Taking $H_0=2\,\sigma_0/w$, and regarding the equivalent rectangular section,

$$\frac{\sigma_{cu}}{\sigma_0} \doteqdot \mp \frac{3}{2} K \frac{x}{H_0}\left(\frac{m}{1+m^2}\right) \qquad 5.513'$$

$$\frac{\sigma_{ed}}{\sigma_0} = \pm \frac{\sigma_{cu}}{\sigma_0} \qquad 5.514$$

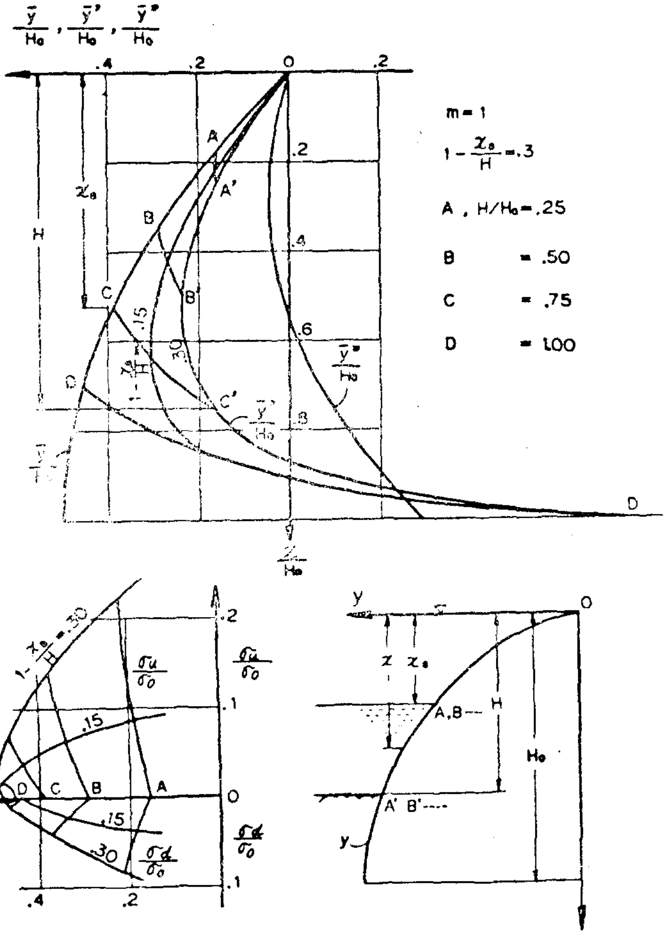

Fig. 5.4 Silt pressure effects when $1-x_s/H$ varies 0 to 0.30

The rate of the influence of seismicity of $K \leq 0.12$ over σ_0 is

$$\frac{|\sigma_e|}{\sigma_0} \leq 0.09 \frac{x}{H_0}$$

Further, when T' is put together with the basic load component T and N, the ratio of total horizontal and vertical component load m increases by the rate of Δm as follows:

$$\Delta m = \frac{T'}{N} = \pm \frac{2K}{3H_0} x \qquad 5.515$$

$$\text{for } K \leq 0.12, \quad |\Delta m| \leq 0.08 \frac{x}{H_0}$$

As to the hydrodynamic pressure due to up and downstream directions, we made an experimental research by means of an electric analogue test applied to $\overline{y}$ curved bi-dimentionla modesl, of which the test results are illustrated in Fig. 5.5. The author describes more in detail the process of the experiments he has done in Part II. The stress analysis using the test results of hydrodynamic pressure will be easily done also by ordinary graphic solution.

When the combined stresses involve seismic stress, the strength of concrete may be increased by about[4] 30 per cent above the standard loading test results. To examine the ratio of the horizontal over the vertical component load m during an earthquake, the strength of concrete can be increased by about 30 per cent in the same way. In this manner, the seismic effects by K more or less 0.12 are usually covered on the safe side.

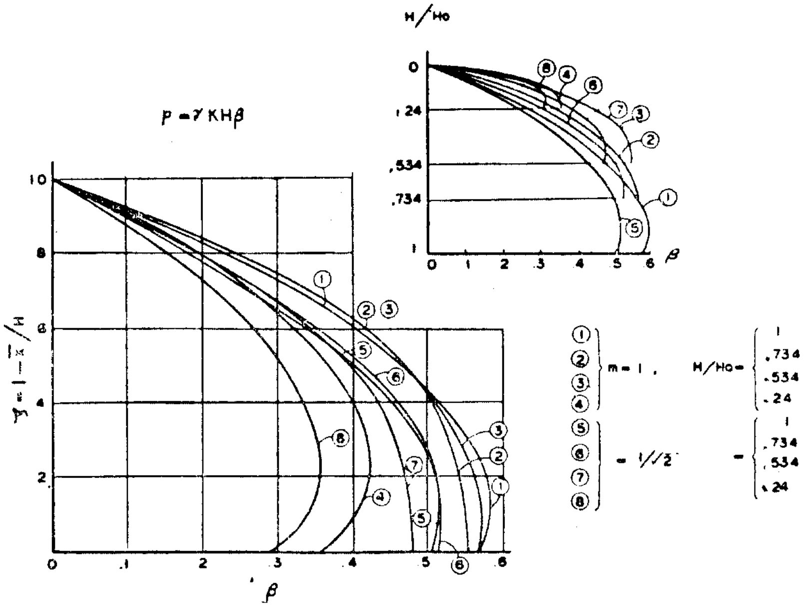

Fig. 5.5 Hydrodynamic pressure probed by tests on face models, for $m=1$ and $m=1/\sqrt{2}$

Conclusion

Provided that the additional stresses due to loads other than water pressure and the weight of the dam itself and all other essential elements in design are well taken cared of in determining the safety factor for m and σ_0, the basic shape of buttress can be applied to buttress dams of considerable height multiple arch dams. In this sense, apart from the fact that a buttress dam may be considered favorably on foundations of relatively inferior rockarea, the writer believes that his idea of the basic shape of buttress will make feasible higher buttress dams than heretofore considered and on wide gorges where a multiple arch type may be better than a single arch dam.

References

(1) SCHORER H. The buttress dam of uniform strength, No. 1803 Proc., ASCE, 1930.
(2) NOETZLI F.A. Improved type of multiple arch dam, No. 1535, Trans., ASCE, 1924.
(3) KUMAKAWA N. Analysis and experiment of hydrodynamic pressure acting on a dam having a parabolic sloped upstream face during earthquake, Communication paper, the 10th Congress on Large Dams, 1970.
(4) Design criteria for dams, chapter III, revised 1969 edition issued by the Japanese National Committee on Large Dams.