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CONTENTS

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Page

DISCUSSION

Proc. Paper 7866

[a] Discussion period closed for this paper. Any other discussion received during this discussion period will be published in subsequent Journals.

Page

TECHNICAL NOTES

Proc. Paper 7860

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7914 February, 1971 SM 2

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FLOW THROUGH ROCKFILL DAM

By Horace A. Johnson,[1] F. ASCE

INTRODUCTION

The passage of large flows through the partially completed Hell Hole Dam, a homogeneous, dumped, rockfill, presented an opportunity to attempt the determination of flow characteristics through rockfill under prototype conditions. Presented herein an analysis based on recorded data collected during the event. Determination of appropriate methods of calculating flow and the location of the phreatic line is attempted. The paper is restricted to the aspect of flow through the rockfill.

DESCRIPTION OF PROJECT AT TIME OF FLOW

Hell Hole Dam on the Rubicon River, California, is a dumped, sluiced, rockfill dam with appropriate filter layers and impervious core placed on the upstream face of the dumped rockfill. Upstream from the core additional rockfill is placed. Since this paper is limited to the study of flow through the uncompleted dam, the project will be described as it existed at the time of the December 1964 flood, when the flow occurred.

The maximum section of the dam at that time was as shown in Fig. 1. The downstream dumped rockfill was completed from abutment to abutment with the top at El. 4470. In addition a second tip of rockfill had been started from the right abutment with top El. 4470 to El. 4485 and top width of about 85 ft. The top of this tip had advanced approximately 350 ft from the right abutment.

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Fig. 2 shows the dam profile looking downstream. The extent of the second tip is shown thereon.

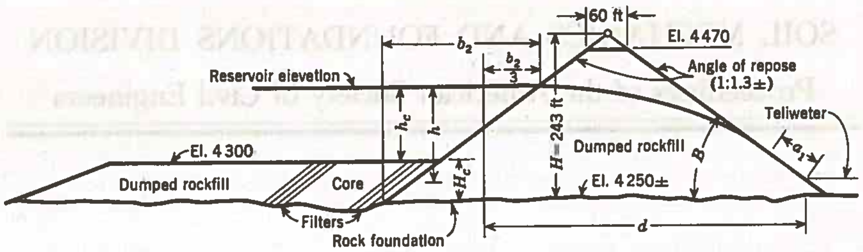

FIG. 1.—MAXIMUM CROSS SECTION OF HELL HOLE DAM—FIRST TIP, AS OF DECEMBER 1964

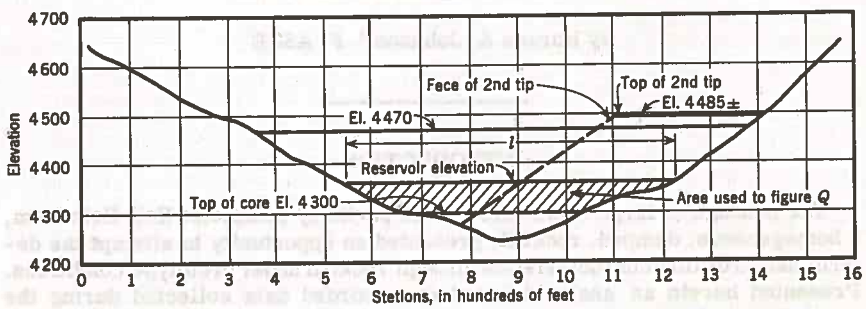

FIG. 2.—PROFILE ON DAM AXIS LOOKING DOWNSTREAM

There was a 13 ft-3 in. unlined horseshoe diversion tunnel through the left abutment. Length of the tunnel was about 1,800 ft. Water temperature was estimated to be approximately 45° F.

HISTORY OF FLOOD OF DECEMBER 1964

In December 1964, a very large flood occurred on the Rubicon River upstream from the damsite. The water behind the partially completed dam rose to El. 4400. Breaching occurred with water at that elevation.

Fortunately, during the flood, reservoir levels were recorded at various times. These observations, in conjunction with the flood hydrograph, allow the determination of flow through the rockfill. Also the elevation of water outcropping on the downstream face was noted at two different times.

DETERMINATION OF RATING CURVES AND FLOW THROUGH ROCKFILL

The rating curve for the diversion tunnel was computed to be 318 $h_t^{1/2}$, in which, h_t is the head from tailwater at El. 4252, to reservoir elevation, using

a value of n = 0.0385 in Manning's formula. Tailwater is considerably higher than El. 4252 for the final flows which occurred through the rockfill. However, the effect on tunnel discharge is small and was neglected.

The flow through the rockfill section can be figured for different elevations by use of the hydrograph, tunnel rating curve, and the observed reservoir elevations at different times. However, values of flows derived in this fashion are scattered because of minor variations between the hydrograph, (USGS, revision of April 25, 1967) which was synthetically derived, and actual inflows. Scattering is also caused by the fact that values of hydrograph flows are not instantaneous but are averaged over a period of several hours.

Another method to calculate flows through the rockfill is to make certain assumptions to get a smooth rating curve, and then to route the flood through the reservoir using this rating curve. If the calculated reservoir elevations do not agree with the observed elevations, then the rating curve must be adjusted.

In this case, the writer attempted to calculate the rating curve before the hydrograph was available to get a rough idea of flows through the rockfill. To do this, laminar flow was assumed, even though this is not true, and a value of permeability of k = 3 fps (92 cm per sec) was arbitrarily chosen. Unit flow was computed by $q = ka_1 \sin^2 B$ (2). With the angle of repose taken as 1 on 1.3, $\sin^2 B = 0.36$ and with $k = 3$, $q = 1.08\, a_1$ cfs/ft. Term a_1 was computed by (2):

$$a_1 = \sqrt{d^2 + h^2} - \sqrt{d^2 + h^2 - \frac{h^2}{\sin^2 B}} \qquad (1)$$

Fig. 1 shows a graphical representation of symbols.

These unit flows were taken over the areas shown in Fig. 2, with tailwater El. 4270. Actual tailwater was about El. 4252 at zero flow, El. 4260 at 5,000 cfs, El. 4270 at 12,500 cfs, and El. 4275 at 20,000 cfs. There was no actual measurement of tailwater at the dam and the preceding figures are rough projections from 2 gages downstream. One, the Foresthill gage is about 30 miles downstream on the Middle Fork of the American River, and the other, the Auburn gage about 50 miles downstream on the North Fork of the American River just below its confluence with the Middle Fork. The Rubicon is a tributary of the Middle Fork. Tailwater El. 4270 was assumed before any study was made of tailwater elevations.

A check was also made by sketching the flow net. The flow nets showed that the 50 ft of core above stream bed had practically no effect on the flow through the rockfill above about El. 4304 where freefall flow ceased. Freefall flow is flow over the core which occurs when the phreatic line downstream from the core is so depressed that hydraulic control is at the core.

Flow over the core is similar to flow over a weir and drops nearly vertically through the rockfill to the phreatic line therein. Freefall rating curves are developed later.

The rating curve obtained from the above computations is shown on Fig. 3, and is that portion of the curve extending from El. 4304 where freefall flow ends to El. 4394, where failure begins as indicated thereon.

Subsequently, when the tailwater elevations became known, the flows were recomputed using these values, and the rating curve replotted as indicated on Fig. 3. The computations for estimated actual tailwater elevations are shown in Table 1.

TABLE 1.—COMPUTATION OF FLOW THROUGH FILL

Reservoir elevation, in feet (1)	h, in feet (2)	d, in feet (3)	a_1, in feet (4)	q, in cubic feet per sec per foot (5)	l, in feet × q = Q, in cubic feet per second (6)
4,305	52	800	8.2	8.7	300 × 8.7 = 2,000
4,325	82	548	9.7	10.4	400 × 10.4 = 4,200
4,350	82	528	17.5	18.9	440 × 18.9 = 8,300
4,375—Center	101	508	28.0	30.0	300 × 30 = 9,000
4,375—Sides	87	420	14.8	18.0	300 × 18 = 4,800
4,375—Total	—	—	—	—	= 13,800
4,400—Center	122	488	41.0	44.0	300 × 44 = 13,200
4,400—Sides	75	390	19.0	20.0	350 × 20 = 7,000
4,400—Total	—	—	—	—	= 20,200

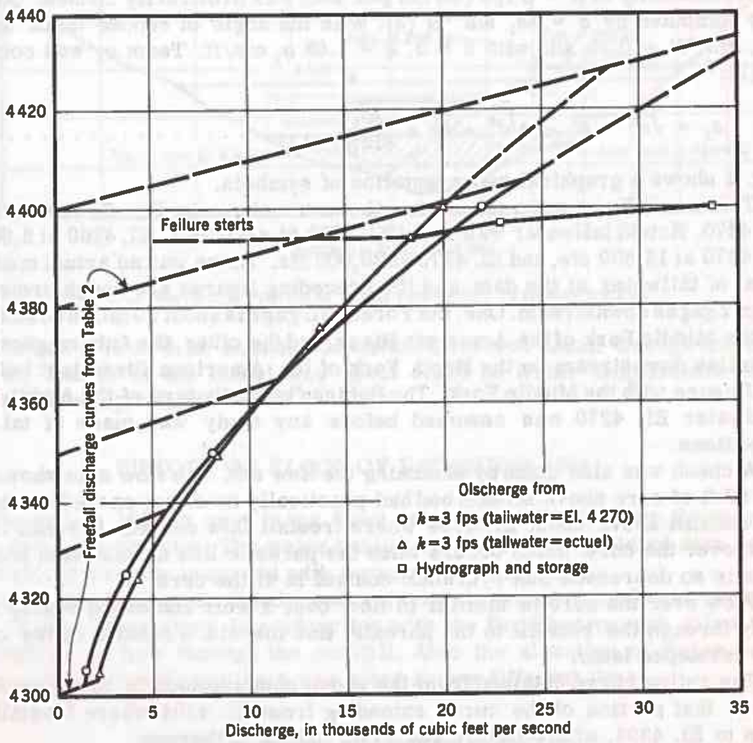

FIG. 3.—DISCHARGE THROUGH HELL HOLE EMBANKMENT

To prepare Table 1, various elevations were assumed. Term h is the difference between reservoir and corresponding tailwater elevations. Term d was determined as indicated in Fig. 1, and a_1 and q were calculated from Eq. 1. The total flow, Q, was found by multiplying q by l, the average width of rockfill through which flow occurred. For two reservoir elevations (El. 4375 and El. 4400), because of the larger triangular areas included between the foundation, a vertical line from the intersection of top of core and the foundation, and the reservoir elevation, the flow through the side areas was figured separately with h taken as an average distance between reservoir and foundation elevations, instead of tailwater.

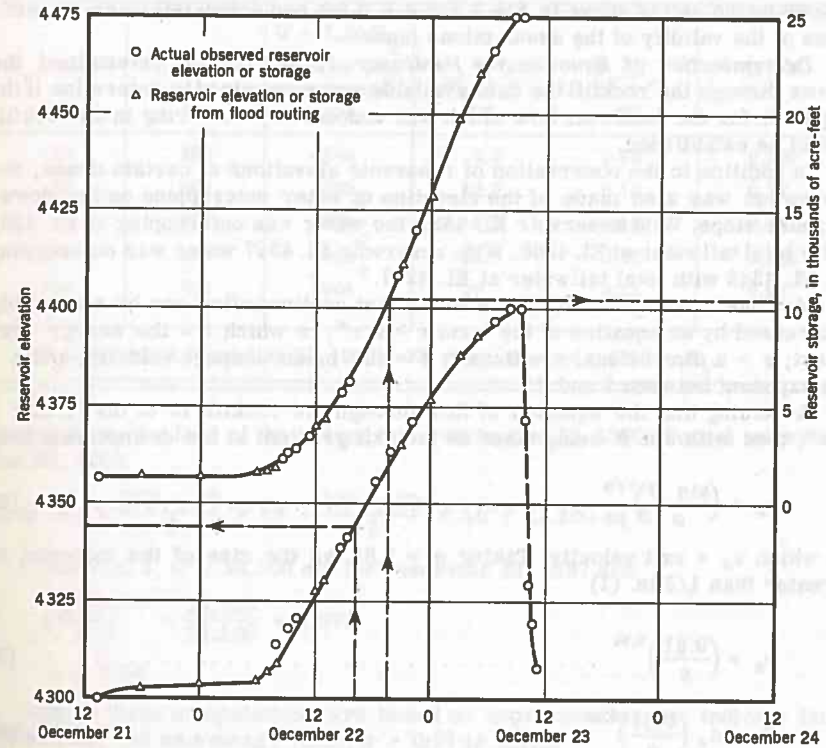

FIG. 4.—RESERVOIR ELEVATIONS AND STORAGE

In the mentioned computations, because of their fictitious nature, no account was taken of the convergence of flow in passing through the rockfill, nor of the fact that most of the discharge causing the tailwater was coming through the rockfill. Also the effect of the second tip of rockfill on the right abutment was ignored. If it were considered the permeability would have to be increased about 10 % to obtain equivalent flows.

It will be noted that the computed area through which flow was calculated as passing was the cross sectional area above El. 4300, as shown on Fig. 2. This would indicate that in calculating flows through a rockfill without any core, some assumption would have to be made to determine the elevation above which the area will be figured.

Using the embankment rating curve determined for k = 3 fps and the actual tailwater, and the tunnel rating curve, the flood represented by the USGS revised hydrograph of April 25, 1967, was routed through the reservoir, and the results plotted on Fig. 4. It will be noted that the agreement between the reservoir elevations and storages obtained by this routing and the actual observed values is excellent. It was not necessary therefore, to adjust the rating curve for the embankment.

Having obtained agreement between observed reservoir elevations and those determined by the flood routing, the flow through the rockfill is then correctly shown by the rating curve in Fig. 3 for k = 3 fps and actual tailwater, regardless of the validity of the assumptions made.

Determination of Equation for Nonlinear Flow.—Having determined the flows through the rockfill the data available were examined to determine if the equation for the nonlinear flow which was undoubtedly occurring in the rockfill could be established.

In addition to the observation of reservoir elevations at certain times, observation was also made of the elevation of water outcropping on the downstream slope. With reservoir El. 4339 the water was outcropping at El. 4285 with total tailwater at El. 4266. With reservoir El. 4397 water was outcropping at El. 4345 with total tailwater at El. 4277.

Various investigations (3) have shown that nonlinear flow can be reasonably expressed by an equation of the form $i = a\,v^n$, in which i = the energy gradient; a = a dimensional coefficient; v = the mean seepage velocity; and n = an exponent between 1 and 2.

Assuming that the equation of flow through the rockfill is of the form $i = av^n$, then with sin B being taken as the exit gradient at the downstream face

$$v_e = \left(\frac{\sin B}{a}\right)^{1/n} \qquad (2)$$

in which v_e = exit velocity. Taking n = 1.85, as the size of the material is greater than 1/2 in. (1)

$$v_e = \left(\frac{0.61}{a}\right)^{0.54} \qquad (3)$$

$$Q = A_e\left(\frac{0.61}{a}\right)^{0.54} \qquad (4)$$

in which A_e = the total vertical area between the water outcropping and the tunnel tailwater. This assumes that the effect of tailwater created by flow through the fill has a negligible effect on the flow.

Turning to the case where the water was outcropping at El. 4285, an examination of the photograph showing this flow indicates that the water was issuing for approximately one half the distance across the face at El. 4285 on the face of the second tip and at approximate El. 4295 on the face of the first tip. The total area, A_e, with tunnel tailwater at El. 4258, is

$$\frac{125 + 70}{2} \times 27 + \frac{(130 + 70)}{2} \times 37 = 6{,}300 \text{ sq ft}$$

From Fig. 3, for reservoir El. 4339, Q = 6,300 cfs and from Eq. 4:

$$\left(\frac{0.61}{a}\right)^{0.54} = \frac{6{,}300}{6{,}300} = 1.0$$

and a = 0.61

For the case where water is issuing at El. 4345, the photograph shows water issuing at this elevation on the downstream face of the first tip and water issuing from the face of the second tip at estimated El. 4300. Tunnel tailwater was El. 4260.

TABLE 2.—FREEFALL DISCHARGE CURVES[a]

H_c, in feet (1)	l, in feet (2)	$Q = 1.20\,lh_c$, in cubic feet per second (3)	h_c, in feet (4)	h_c/H_c (5)	H_c/H (6)
50	400	$480h_c$	4.0	0.08	0.205
80	500	$600h_c$	10.5	0.13	0.33
100	600	$720h_c$	15.5	0.155	0.41
130	750	$900h_c$	22.0	0.17	0.535
150	800	$960h_c$	27	0.18	0.62

[a] H_c is based on foundation El. 4250 × El. of top of core = 4250 + H_c.

Then $A_e = \dfrac{360 + 70}{2} \times 85 + \dfrac{120 + 70}{2} \times 40 = 22{,}300$ sq ft

From Fig. 3, Q = 24,000 cfs for reservoir El. 4397 and

$$\left(\frac{0.61}{a}\right)^{0.54} = \frac{24{,}000}{22{,}300} = 1.075$$

$$a = 0.54$$

Both of these computations are based on approximation but indicate fair agreement, and an average value, a = 0.57 is taken.

Using this value of a for reservoir El. 4339

$$Q = 6{,}300 \times \left(\frac{0.61}{0.57}\right)^{0.54} = 6{,}500 \text{ cfs}$$

a difference of only + 3 % from that obtained from Fig. 3.

For reservoir El. 4397

$$Q = 22{,}300 \times \left(\frac{0.61}{0.57}\right)^{0.54} = 23{,}400 \text{ cfs}$$

a difference of - 2.5 % from that obtained from Fig. 3.

As a further rough check on the value of a, the average value of the hydraulic gradient from reservoir elevation to outcrop, s = 0.115 for reservoir

El. 4339, and $s = 0.165$ for reservoir El. 4397 were inserted for i in the equation $v = (i/a)^{1/n} = (i/0.57)^{0.54}$ and the resulting velocities multiplied by the vertical areas above the core, El. 4300, to obtain the total flow, Q. This is equivalent to assuming that the phreatic line is a straight line between the upstream and downstream boundaries, which is probably not far from the actual conditions except for the drawdown at both boundaries which would decrease the intermediate gradient.

Making the above substitutions, for reservoir El. 4339, $v = (0.115/0.57)^{0.54} = 0.42$ fps and $Q = 39\ (400 + 300/2)\ 0.42 = 6{,}600$ cfs, for reservoir El. 4397, $v = (0.165/0.57)^{0.54} = 0.51$ fps and $Q = 97\ (770 + 300/2)\ 0.51 = 26{,}500$ cfs. These flows are, respectively, 4.8 % and 10.4 %, greater than those obtained from the rating curve.

TABLE 3.—VALUE OF h_c/H_c FOR Q = 18,500 cfs[a]

Reservoir elevation, in feet above mean sea level (1)	H_c, in feet (2)	h_c, in feet (3)	h_c/H_c (4)	H_c/H (5)	Freeboard, in feet from elevation 4,370 (6)
4394	50	94[1]	1.88	0.205	76
4394	75	69[1]	0.92	0.31	76
4394	100	44[1]	0.44	0.41	76
4423.5	150	23.5[2]	0.157	0.62	46.5
4456.2	190	16.2[2]	0.085	0.783	13.8
4465.7	200	15.7[2]	0.079	0.825	4.3
4469.0	203.5	15.5[2]	0.073	0.837	1.0
4473.5	220	3.5[3]	0.016	0.905	-3.5

[a] Determined from rating curve for k = 3 ft/sec and actual TW; freefall through rockfill; free overflow over top of dam, $Q = 3 \times 1{,}000\ h_c^{3/2}$ based on crest length of 1,000 ft.

With the value of a now known, it is possible to establish the freefall flow equation

$$q = \left(\frac{0.8}{a}\right)^{0.54} h_c = \left(\frac{0.8}{0.57}\right)^{0.54} h_c = 1.20\ h_c \text{ (Ref. 1)} \quad \ldots\ldots\ldots\ldots (5)$$

The freefall equation is based on the work of Parkin, Trollope, and Lawson (1) where the energy gradient directly over the crest in rockfill dam models was found to be 0.8. From this equation the freefall discharge curve at the bottom of Fig. 3 was plotted for the height of core (H_c) of 50 ft as actually existed.

The freefall discharge equations for various assumed values of H_c, i.e. higher core elevations, as shown on Fig. 1, are computed and shown in Table 2. In this table h_c is the value at which freefall ceases, as determined by

plotting these equations of Fig. 3 to intersection with the rating curve for k = 3 fps and actual tailwater.

From photographs taken during the flood it was apparent that at no time did the tailwater become the control at the downstream face. Also, these photographs clearly indicated that failure was caused by erosion and slides on the downstream face. This indicates that failure would be dependent upon the amount of flow issuing from the downstream face regardless of how this flow enters the upstream face of the dam. It appears that interesting results can thus be obtained by assuming core elevations different from that actually existing and determining heights of reservoir (h_c) above the different core elevations for the flow (18,500 cfs) which actually started failure of the rockfill,

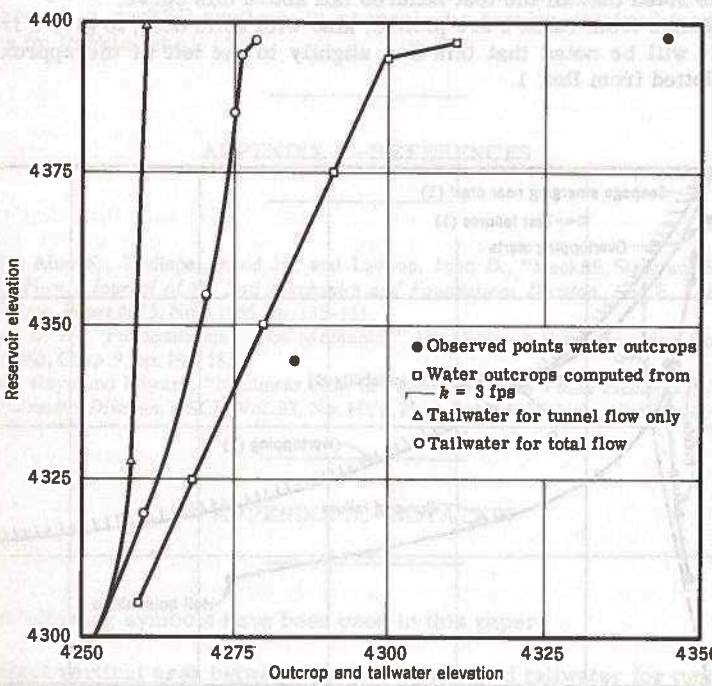

FIG. 5.—WATER OUTCROPS AND TAILWATER

and which should equally start failure under the assumed conditions. On this premise, Table 3 was constructed by increasing H_c, as shown on Fig. 1, holding the reservoir water level at El. 4394, until freefall becomes the control. Then H_c is further increased with freefall controlling the total outflow to 18,500 cfs (the outflow at commencement of failure), until the dam is overtopped. It should be noted that with each increase in H_c the size of flood required to produce an outflow of 18,500 cfs is changed.

Fig. 5 shows a comparison of reservoir water elevations against: (1) The actual water outcrop elevations; (2) elevations at which water would appear on the downstream surface if flow were laminar and k = 3 fps; (3) tailwater for tunnel flow only; and (4) tailwater for total flow, i.e. the flow through the tunnel plus the flow through the rockfill.

With the data from Tables 2 and 3 it is interesting to replot Fig. 6 obtained by Parkin, et al. (1) which is also Fig. 6 herein. In Fig. 6, H_c/H is plotted against h_c/H_c. On this are plotted the following data obtained by Parkin, et al., (1): (1) Model test failures (0); (2) hatched curve above which lies the region of overtopping; (3) solid curve below the hatched curve which indicates seepage emerging near crest; (4) dashed curve to the left of which freefall conditions operate; and (5) area denoted as Region of Instability surrounding the plotted test failures and lying between the freefall curve and the overtopping curve. The plot of h_c/H_c is extended to cover the Hell Hole failure point. The points obtained from Table 3 are plotted with solid dots and labeled Curve of Failure. It will be noted that all the test failures fall above this curve.

The points from Table 2 are plotted, also with solid dots, to give a freefall curve. It will be noted that this lies slightly to the left of the approximate curve plotted from Ref. 1.

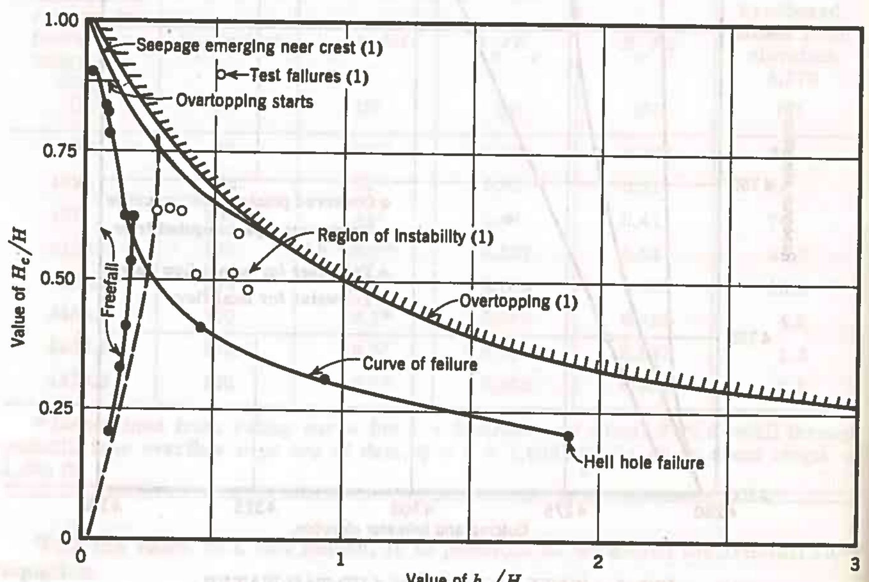

FIG. 6.—STABILITY DESIGN CHART

Any combination of values of H_c/H and h_c/H_c, for rockfills similar to Hell Hole Dam, which plots below the Curve of Failure should not fail. This is regardless of whether the flow over the core is occurring under freefall conditions or not. In fact the curves indicate that failure could occur even under freefall conditions. Because of the many uncertainties in determining the permeability of rockfill the writer believes that any design involving flow passing through rockfill must be very conservative and well below the Curve of Failure.

No grading curves for the rockfill materials, which were quarry run, were

obtained, but the author believes that the percentage of fine material is much greater than usually assumed.

CONCLUSIONS

The aforementioned analysis of flow through an actual rockfill suggests that flow through a rockfill can be calculated assuming laminar flow.

After computing the flow assuming laminar flow conditions, then by determining, or assuming a value of a, the height of water outcropping on the downstream face can be found. Two points on the phreatic line are then determined and its approximate location through the remainder of the fill can be determined. Fig. 6 indicates that results obtained from the analysis of the full scale flow through a rockfill confirm, in general, the results obtained from model studies by Parkin, et al. (1).

APPENDIX I.—REFERENCES

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APPENDIX II.—NOTATION

The following symbols have been used in this paper:

A_e = total vertical area between water outcrop and tailwater for tunnel flow only;
a = dimensional coefficient;
a_1 = slope distance on downstream face from tailwater to water outcrop;
B = angle of inclination from horizontal of downstream face;
d = horizontal distance from tailwater to point upstream from intercept of reservoir surface and upstream face 1/3 of horizontal distance from mentioned intercept to intersection of upstream slope and foundation;
H = height from foundation to intersection of upstream and downstream slopes;
H_c = height of core (assumed, as well as actual);
h = height of reservoir above tailwater;
h_c = height of water over core (assumed, as well as actual);
h_t = head from reservoir elevation to El. 4252;
i = energy gradient;

k = coefficient of permeability for laminar flow;
l = average length of fill through which certain unit flow, q, is passing;
n = Mannings coefficient;
n = exponent;
Q = total flow through rockfill;
q = unit flow per foot of dam;
v = mean seepage velocity; and
v_e = exit velocity at downstream face.

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IN-SITU INVESTIGATION USING SH-WAVES

By Marcis Kurzeme[1]

INTRODUCTION

The apparent velocity of elastic waves as observed at the surface of a layered soil or road pavement structure is dependent on the frequency of the wave motion, the properties of the materials present and the geometry of the layers. Observation of the surface wave velocity at various frequencies, therefore, provides a means of obtaining information on the properties of the propagating materials and structure by nondestructive methods. A method of investigating soils and pavement materials in-situ through the excitation and observation of continuous horizontally polarized shear waves (SH-waves) is described herein.

The use of continuously generated surface waves for in-situ testing of soils and pavements was initiated during the period of 1928-1938 at DEGEBO in Germany. The development of the technique to 1960 has been described by Jones (5). The surface wave technique of in-situ investigation receiving the most attention has been the Rayleigh wave method (6). This is now capable of being used with reasonable confidence to investigate most modern pavements. Pavement investigation using SH-waves has been neglected, apparently as a result of difficulties in generating SH-waves, and doubt as to the existence of observable SH-wave propagation in pavement type structures. The only previous investigations (1,4) using continuously generated SH-waves have been restricted to the observation of Love waves in two layer earth type structures.

Love waves are SH-waves propagating in a specific structure consisting of a uniform layer of material overlying a semi-infinite medium of higher rigidity. This is opposite to the rigidity contrast normally encountered in road pavements. Particle displacements constituting the wave motion of SH-waves are horizontal and transverse to the direction of wave propagation.

Note.—Discussion open until July 1, 1971. To extend the closing date one month, a written request must be filed with the Executive Director, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 97, No. SM2, February, 1971. Manuscript was submitted for review for possible publication on April 16, 1970.

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