Grand Coulee Riverbank Stabilization – Case History of the Design of Remedial Measures

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SYNOPSIS Planned peaking operations at Grand Coulee Dam will cause river fluctuations of up to 21 feet daily for the authorized plant and up to 38 feet daily for a proposed plant expansion. Treatment of the historically unstable riverbanks was required to offset the destabilizing effect of the peaking operations. Consideration of the geologic history of the bank materials, estimation of future pore pressure conditions, formulation of the stabilization criteria, and the success in reducing pore pressure levels in varved clays in the bank were some of the important aspects of this project.

INTRODUCTION

The third powerplant at Grand Coulee Dam in Washington State, U.S.A., added three 600-MW and three 700-MW generating units to the site's powerplant system. These units as well as additional units proposed for an extended third powerplant greatly increase the peaking power capability of the plant. The increased peaking power generation will in turn create fluctuations of 21 feet (38 feet for the extended plant) in the Columbia River downstream of the dam. The populated riverbanks are to be treated for a distance of 6 miles downstream of the dam.

Before full peaking operation of the third powerplant can take place, stabilization measures to mitigate any adverse impact of the peaking operation must be implemented. The scope of the originally proposed measures varied from relocation of the downstream population to a total engineering solution avoiding any resident relocation.

The banks of the Columbia River below Grand Coulee Dam revealed a geologic history of instability. Landsliding occurred during construction of the dam and during subsequent peaking operation of the powerplant despite strict limits on allowable drawdown related to peaking. For many years, these limits were on the order of 4 feet in 24 hours, 7 feet in 48 hours, and 8 feet in 72 hours. Large slides also resulted from the much greater but more gradual drop in river elevation following the maximums reached during spring runoff. Plans for the third powerplant addition at Grand Coulee Dam recognized that significant stabilization of the channel and riverbank slopes or resident relocation would be required. Excavation required for the third powerplant and forebay dam was placed along the toe of the riverbank over the 6-mile length to be treated. Although this toe loading provided additional resistance for large potential slide masses, it aggravated and even created very near river slope failures due to oversteepening of the fill resting on the Nespelem clay deposits which daylighted or nearly daylighted in the river channel.

The geotechnical problem facing designers at this site was to determine what slope stabilization measures, if any, could be implemented to allow peaking operation of the plant. The problem was made more complex because of the following considerations:

1. Much of the area to be stabilized was populated and although resident relocation was an option, there was strong sentiment against relocation by some residents including the native American population residing on tribal lands.

2. Because of the large high velocity flows associated with peaking, significant additional constriction of the river to buttress the potential slide masses was not permissible.

3. Because numerous political entities as well as private individuals were potentially affected, workable and equitable criteria for establishing the need for and level of stabilization works were required.

4. Artesian water pressures existed in several of the areas to be stabilized.

5. Varved clay deposits, which are the root of the stability problems at this site, are not generally regarded as candidate materials for drainage. Thus, any potential benefit from drainage had to be verified by testing.

The stability analysis for the existing slopes and for the slopes with proposed treatment alternatives followed a traditional approach of determining: (1) the material profile, (2) the geometry of potential failure surfaces, (3) the strength of the materials, (4) the pore pressures in the deposits, and (5) the resultant factor of safety using a limit equilibrium procedure satisfying all three equations of equilibrium. This paper will concentrate on reporting the more unique aspects of the problems encountered in conducting the stability evaluation at this site and describe the approaches used in overcoming these problems. The specific aspects to be covered include:

1. Relationship of shear strength and shear surfaces to geologic deposition history

2. Estimation of future pore pressure conditions with and without drainage treatment

3. Development of stabilization criteria

4. River channel protection

5. Results of subsurface drainage in clay by pumped wells and by shafts with radial drains

GEOLOGIC PROFILE AND MATERIAL STRENGTH DETERMINATIONS

Although 31 cross sections were used to represent the variations in geometry, pore pressure, and distribution of the component materials along the 6-mile reach studied, the stratigraphic profile consistently reflected the intriguing geologic history of the Columbia River Valley between Grand Coulee and the Okanogan River. A riverbank cross section (fig. 1) through the town of Coulee Dam about 1 mile downstream of Grand Coulee Dam demonstrates the components of the geologic profile consisting of: (1) an erosional (fresh) granite surface resulting from the intensive scour of glacial melt waters during early advance and retreat cycles of the glaciers (Circa 20,000 year B.P.), (2) a basal gravel deposition of the glacial outwash flows, (3) varved silts and clays (Nespelem Formation) deposited by Lake Columbia which formed when the Cordilleran Ice Sheet, in a series of advances and retreats, moved down the Okanagan River valley creating periodic ice dams across the Columbia River, and (4) glacial terrace deposits from glacial retreat.

The glacial load on the nespelem deposit produced overconsolidation of the soils and the glacial override also caused shearing of the deposits, reducing shear strength to residual values along horizontal to subhorizontal planes of unknown location and extent. Subsequent erosion of the valley by the Columbia River and the forces of gravity resulted in landsliding which also produced sliding surfaces with near residual strength. Because the potential slide planes along residual surfaces could appear anywhere throughout the clay interval, critical circular and noncircular shear surface searches were conducted [Chugh, 1981a] and representative critical surfaces were established to cover the total cross section. In general, two shear surfaces of a circular nature near the river bank and three noncircular surfaces covering larger slide masses were required to represent the critical slope conditions with respect to geometry and to provide a measure of the effectiveness of treatment as a function of distance from the bank (see fig. 1).

Back analysis of the renewed movement on historic landslides of various sizes along the river following construction and operation of Grand Coulee Dam established an effective residual shear strength of $\phi = 12^\circ$ for the clay materials. Direct shear tests on slickensided surfaces, precut surfaces, and unprepared surfaces varied from about $\phi = 10^\circ$ to $\phi = 16^\circ$. Rotational shear tests on the

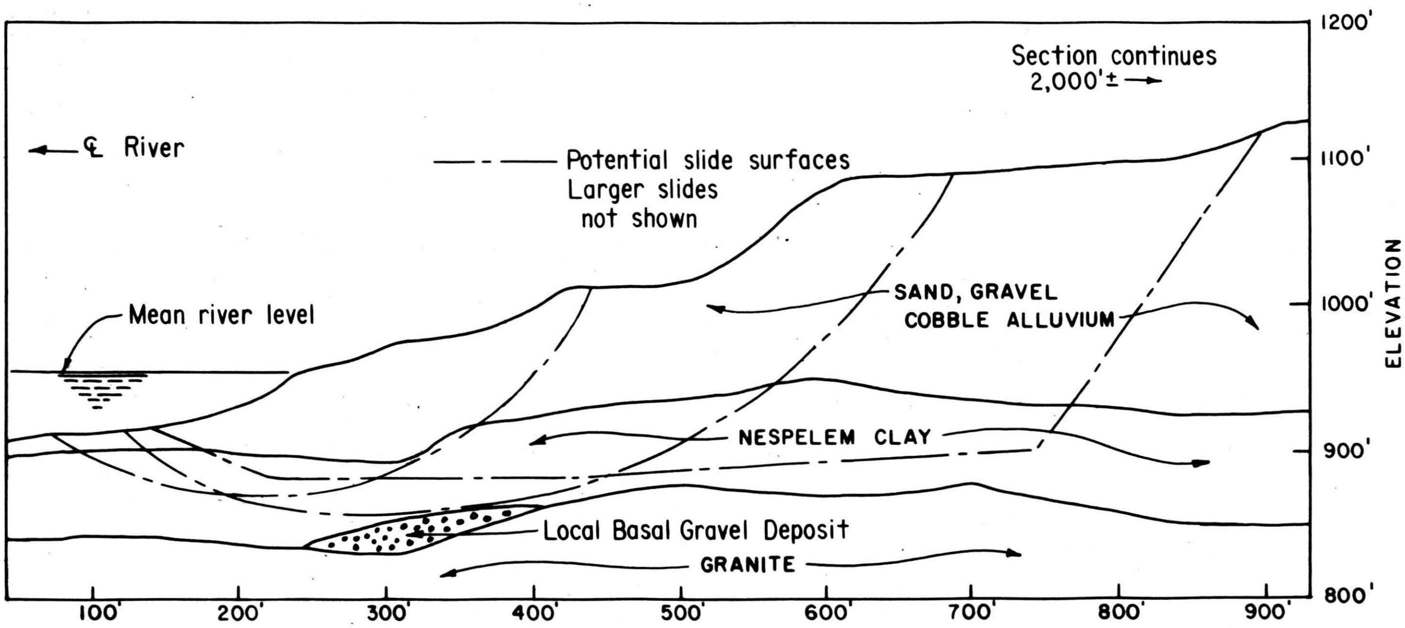

Fig. 1. Geologic profile of typical riverbank cross section

clay were generally 2° lower than direct shear tests. Back analyses were regarded as the most realistic for use in the studies, thus shear strengths along potential slide surfaces in clay were set at ϕ = 12° and C' = 0 unless the factor of safety obtained for existing conditions was less than 1.0. In those cases, a shear strength adjustment was made so that the surface just indicated stable conditions for known stable conditions under drawdown. This back analysis approach provided a frame of reference for estimation of the impact of peaking operations on slope stability without precisely knowing the current factor of safety of the slopes.

ESTIMATION OF FUTURE PORE PRESSURE CONDITIONS IN THE BANKS

In order to estimate the adverse impact on stability due to the planned peaking operations, it was necessary to develop projections of future pore pressures in the banks that would develop if no treatment was applied. The changes in pore pressure would result from two effects:

1. The mean river level would rise about 7 feet due to the raising of Chief Joseph Dam downstream.

2. The flucuations about the mean would increase from the present 12-foot restriction to about 21 feet[1] under the currently authorized plant and about 38 feet under a proposed extension.

Raising the mean river level would in general, at least temporarily, improve stability; however, it was necessary to establish the appropriate steady-state pore pressure in the banks under the raised mean river level in order to properly analyze the eventual most adverse conditions under drawdown.

Based on past piezometric readings, reflecting time periods when the river fluctuated around a certain mean elevation, pore pressure contours within the natural slope were developed for each of the 31 stability analysis cross sections. Piezometer readings used in these developments were then examined for two indicators of the future pore pressure response to a rise in mean river level: (1) indications of change due to rapid (daily) swings in river level and (2) indications of reponse due to a significant long-term change in river elevation (e.g. during spring runoff, a 20-foot rise might last for 3 months). Based on such empirical data, it was judged that: (a) locations which showed a response due to a short-term river fluctuations would, in the long term, have a steady-state pore pressure increase equal to the full rise in mean river level, (b) locations which showed no effect due to a long-term river level increase defined the limit point in the bank of no effect of long-term rise in mean river level, and (c) locations which showed no short-term response but did indicate a long-term response would, in the long-term, have a rise in pore pressure due to mean river level raise intermediate between case (a) and (c) with the magnitude of the rise based on a linear proportion of distance.

Two-dimensional finite element seepage analyses using calibrated permeabilities and boundary conditions based on current steady-state pore pressure conditions were made to check the empirical solution. The finite element analysis confirmed the general pattern of results developed from the empirical formulation, thus, the empirical approach was adopted to predict the steady-state pore pressure.

An estimate of the excess pore pressure on potential sliding surfaces under future drawdown conditions was also based on an empirical approach. The increase or decrease in pore pressure (fig. 2A) from the static level associated with a constant mean river elevation was plotted as a function of the change in river elevation from mean river level (fig. 2B) to obtain the relationships shown in figure 2C. The ratio of pore pressure change to river change was generally quite consistent for the rising and falling condition. In all cases, there was a net decrease in pore pressure from the static level for the case of a rapid river rise followed by a rapid river drop, modeling the peaking operation of the river. To obtain the pore pressure on a potential slide surface under future drawdown conditions, the pore pressure, at a particular point in the hillside, was developed by extrapolation as shown in figure 2C and described on table I.

This rather straight forward procedure was used to adjust the pore pressure contours developed from numerous pore pressure measuring locations monitored under current conditions to estimates of future conditions. Although other more involved procedures were studied [Chugh, 1981b], this empirical approach produced consistently reasonable results and is considered to be a reliable procedure.

DEVELOPMENT OF STABILIZATION CRITERIA

The river valley slopes over the 6-mile reach:

1. Consisted of a geologic profile prone to instability

2. Existed currently in various states of stability (ancient landslides, historic landslides, unfailed slopes)

3. Varied in stability as a function of distance from the riverbank because of local variations in topography, subsurface profile, material strengths, ground-water conditions, and past treatment of the toe of the slopes.

It was recognized that the effect of daily peaking operations could have a significant effect on small potential landslides, but barring a progressive-type failure from toe erosion, which was to be prevented by armoring of the channel, the impact of peaking operations on the stability factor of slides of major proportions would be very small. Further

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[1] References to river level fluctuations and elevations are with respect to the highway bridge about 1 mile downstream of the dam. Variations in these parameters occur along the 6 miles of riverbank being studied.

Table I. - Example procedure for estimation of future pore pressures

Condition measured	Current reading (ft)	Piezometer response to river level change (specific piezometer)	Estimate of future reading (ft)
Mean river level	955		962
Piezometer with river at mean level (steady-state)	960 Excess pore pressure (5')	Responds to daily fluctuation	967 Excess pore pressure (5')
Piezometer under 5 foot drawdown of river	958.5 Excess pore pressure (8.5')	30% of river level change	965.5 Excess pore pressure (8.5'
Piezometer under 10 feet of drawdown	NA	30% of river level change	964 Excess pore pressure (12')
Piezometer under 18 feet of drawdown	NA	30% of river level change	961.6 Excess pore pressure (17.6

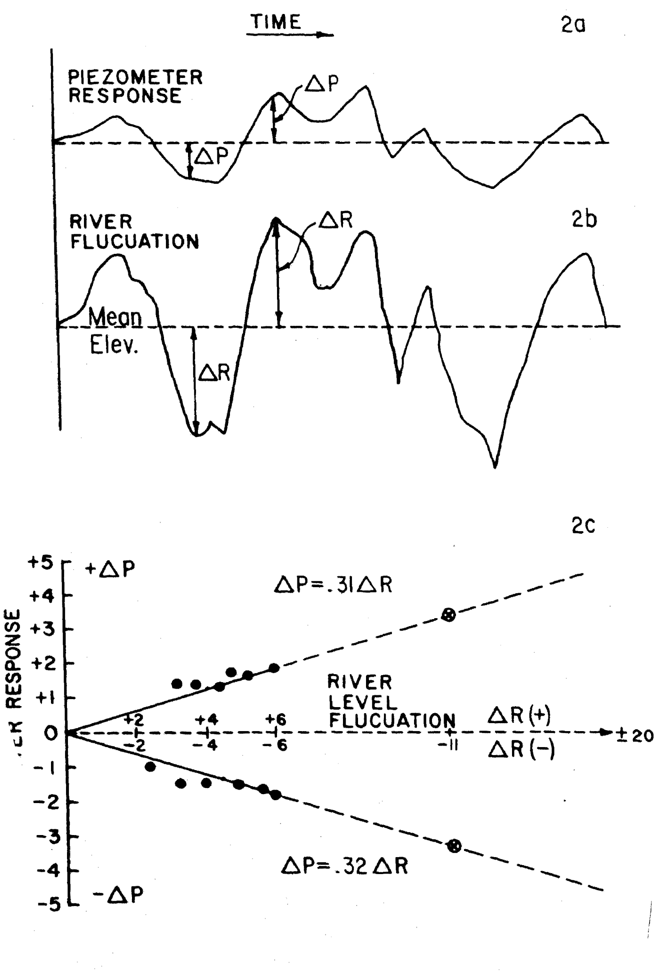

Fig. 2. Projected piezometric response to river level fluctuations

it was recognized that raising the factor of safety of such massive slides to any signific degree by treatment works would not be realistic.

Based on the above understanding, the goals of the stabilization were established:

1. To prevent toe erosion by armoring the channel. (This action was considered as a minimum requirement of all options includir resident relocation.)

2. To offset any calculated adverse impact of the peaking operations by treatment works.

3. To provide an additional margin of safety to guard against general past instability of the hillsides and to cover uncertainties in the estimation of the impact of peaking operation on stability.

The criteria developed to accomplish these goals required that stabilization measures be added that would be equivalent to twice the calculated harm due to increased peaking operations. Thus, for example, if the estimated factor of safety for a potential slide surface was 1.2 under current operating conditions, an the factor of safety under future peaking conditions was 1.1, the stabilization measures would be required to add 2 x (1.2 - 1.1) = 0.2 to the factor of safety under future peaking (1.1 + 0.2 = 1.3). The initial factor of safety and the final factor of safety estimate under this criterion were therefore not critical determinations because the improvement to be developed was relative. This approach appropriately recognized the difficulty that would have been encountered in trying to establish a reliable factor of safety for each of the potential slide surfaces over the entir 6-mile reach.

The initial factor of safety used in each case was based on best estimates of pore pressure, most critical orientation of failure surface geometry, and residual strength parameters in clay. For those cases where such a determination indicated a factor of safety less than 1 for known stable conditions, small adjustments in clay strength parameter were made so that the initial estimate of factor of safety would at least match reality. In addition to the preceeding basic criterion, additional criteria were used as follows:

1. A surface with a factor of safety of 1.35 using residual strengths and future peaking conditions would not require stabilization treatment.

2. Following stabilization treatment under future drawdown conditions, a minimum factor of safety was required as follows:

   - near river surfaces (< 500 feet from the river) - F.S. = 1.2

   - other surfaces non-Federal land - F.S. =1.1

   - other surfaces Federal land - F.S. = 1.05

Recall that when necessary these factors of safety would not be actual values but rather values reflecting variations from an F.S. set at 1.0 for current conditions.

RIVER CHANNEL PROTECTION

A major factor in accomplishment of the bank stabilization both in terms of cost and providing the required level of treatment was the provision for reshaping and armoring of the embankment fill that had been previously placed near the rivers' edges.

Cross sections below river level were taken at 100-foot intervals along the 6-mile reach (fig. 3A). Analysis of the stability of these banks showed that they were oversteepened. Hydraulic studies indicated that additional fill to buttress these slopes would be detrimental to riverflow performance. Thus, a concept of reshaping the banks to a stable slope while balancing cut and fill was used (fig. 3B). Using discharges expected under the extended plant of 400,000 ft^3/s, both mathematical and physical model studies were made to aid in riprap sizing determinations. After the required slope to ensure stability was established, a minimum 4-foot graded riprap blanket was specified. The 50 percent size of this blanket was about 24 inches and the maximum size was 40 inches. The completion of this reshaping and armoring is currently being performed under a $ 35 million contract involving approximately 1.5 million yards of excavation, 2.8 million yards of fill, and 3.1 million tons of riprap armor.

A major concern in the protection of the river channel is the presence of clay exposures in the riverbed. Monitoring pins for inspection of the erosion of these clays were placed by divers and subsequent observations showed some erosion occurring under current discharge levels. Special flow erosion tests using samples from the riverbed and site water also indicated the possibility of erosion of the clay exposures in the riverbed. The reshaping

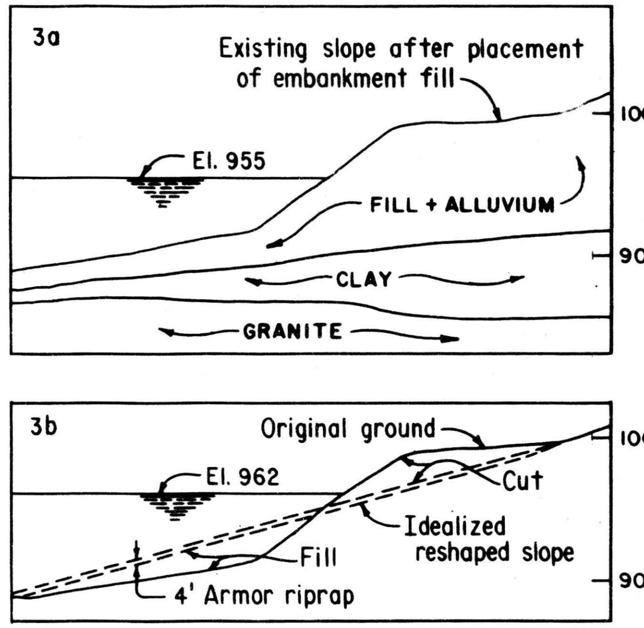

Fig. 3. Slope reshaping concept

covered a number of these exposures near the toe of the slope. However, there was concern that blanketing exposures in the channel bed would aggravate erosion. Thus, during the first phase of operation in which maximum peaking flows would be 260,000 ft^3/s and the river level will be 7 feet higher, the exposures will not be treated but will continue to be monitored. The search for a reliable treatment method continues in the event monitoring indicates significant erosion.

SUBSURFACE DRAINAGE TREATMENT

In order to meet stabilization criteria in some of the areas along the 6-mile reach, the stabilization provided by the reshaping had to be supplemented by reduction in pore pressure. Pumpout tests in drill holes had indicated that withdrawal of water from the granite bedrock effected some reduction in pore pressure in nearby clay piezometers. Stabilization measures for the Visitors Arrival Center near the damsite had shown a similar effect. Thus, a possible treatment procedure was to establish a single or dual line of pumped wells near the riverbank which could reduce clay pore pressures over a portion of the cross section improving the factor of safety for all potential slide surfaces passing through that portion of the cross section and, in effect, isolating the bank from the pore pressure effects of the river peaking operation. To enhance the pore pressure reduction in the clay, a concept using a shaft with radial drains extending into the clay was also considered (see figure 4).

The shaft and radial drain system offered the following advantages: (1) after shaft installation, drains could be drilled easily and inexpensively, (2) broader coverage could be obtained, (3) drains could be readily directed to zones which may indicate a need for

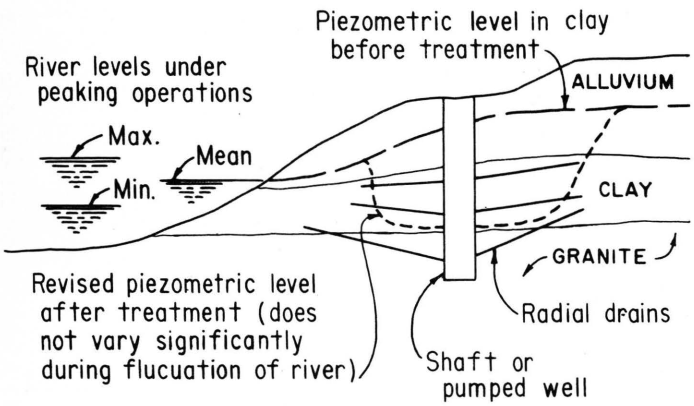

Fig. 4. Near river subsurface drainage concept

additional pore pressure reduction, (4) drainage could be accomplished in the clays underlying the underwater riverbank slope and (5) maintenance of pumps would be reduced because drainage would be by gravity except for a single sump pump in the shaft. The shaft system was considerably more costly on an initial capital cost basis than was a well system. Because the added effectiveness of a shaft and radial drain system had not been demonstrated and because it was necessary to demonstrate the thoroughness of the pore pressure reduction within the clay layer, a full-scale test program involving a comprehensive pattern of piezometers, pumped wells, and a shaft with radial drains (see fig. 5) was constructed in one of the areas requiring subsurface drainage for stabilization. Numerous piezometers recorded the sequence of drainage events enumerated on figure 6. As is evident on figure 6 for one such piezometer and in table II for a number of piezometers, the drawdown achieved in clay piezometers by the pumped wells and the later abrupt increase in drawdown by installation of radial drains demonstrated the technical acceptability of both systems. The piezometric coverage of the clay sequence in narrow segments was accomplished by use of multiple piezometers in a single drill hole. These installations revealed greater drawdown in the clay near the granite contact. This effect, along with the lateral attenuation in drawdown normally expected, was considered in the stability analysis with treatment in place.

CONCLUSIONS

1. Geologic factors have once again demonstrated their fundamental importance in the analysis and treatment of a slope stability problem.

2. Pore pressure changes resulting from peaking operations were effectively and efficiently predicted using an empirical extrapolation approach.

3. Stabilization criteria based on relative harm and improvement provides a workable means for handling large landslide masses exposed to works of man.

4. Subsurface drainage of varved clays using pumped wells was found to be a workable system. A shaft with radial drains enhanced the drainage effects significantly.

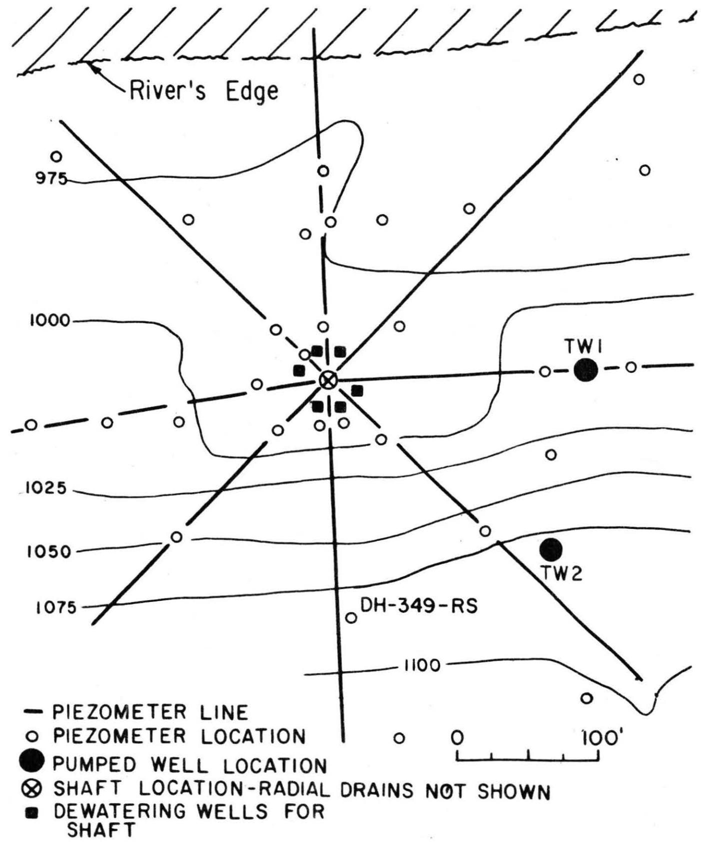

Fig. 5. Plan view of subsurface drainage concept

Table II. - Summary of subsurface drainage (based on 14 representative piezometers isolated in lower clay intervals)

*Drainage method	Range of drawdown achieved (ft)	Average drawdown achieved (ft)
Pumped wells TW1 and TW2	0-48	17.5
Dewatering wells for shaft	18-77	46.3
Shaft with radial drains	33-85	58.1

*Note distances to piezometers vary with method.

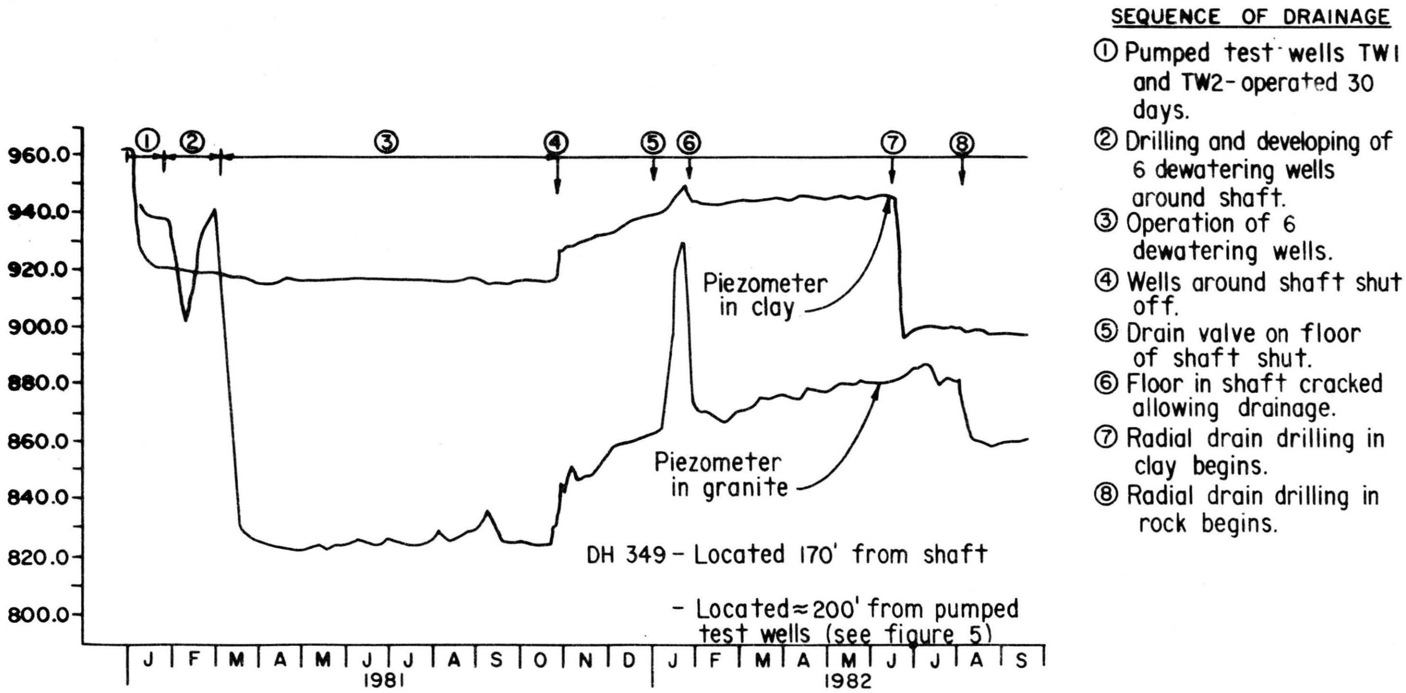

Fig. 6. Piezometric response to subsurface drainage (typical piezometer isolated in clay and granite)

REFERENCES

Chugh, A. K. and J. D. Smart, (1981a), Suggestions for Slope Stability Calculations, Computers and Structures, Vol. 14, No. 1-2, pp. 43-50.

Chugh, A. K. (1981b), Pore Water Pressure in Natural Slopes, International Journal for Numerical and Analytical Methods in Geomechanics, 5, 449-554.