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# Powerhouse Slope Behavior, Fort Peck Dam, Montana

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SYNOPSIS Landslides occurred in the Bearpaw shale slope adjacent to the powerhouses at Fort Peck Dam in the geologic past. Excavation of the slope toe for construction of reservoir outlet works in 1934 initiated progressive sliding of colluvium which continued to 1974. The active slide area had an average movement rate of 4 ft/yr from 1944-1945 and average movement rates of 1-2 ft/yr from 1953-1971. These movements caused no distress to the powerhouses or other facilities. In 1974, the slope was stabilized by excavating 1.6 x $10^6$ cu. yd. of material, resulting in a 1 on 6 overall slope. A field residual strength given by c' = 0.1 ksf, $\phi$' = 10° or c' = 0, $\phi$' = 11.5° for effective normal stresses of 3-4 ksf was calculated from the slides using 1950's topography and groundwater levels.

## INTRODUCTION

The Bearpaw shale slope adjacent to the powerhouses at Fort Peck Dam experienced landsliding sometime during its geologic history. Excavation of the slope toe during construction of reservoir outlet works in 1934 initiated progressive sliding of colluvium which continued until 1974. In that year, 1.6 x $10^6$ cu. yd.* of material was excavated when the 200 ft high slope was flattened to an overall inclination of 1 on 6 in the main slide area. This excavation appears to have stabilized the slope.

The Fort Peck powerhouse slope has been studied by the U.S. Army Corps of Engineers from 1934 until the present. Construction and movement history of the powerhouse slope is summarized herein. Mohr-Coulomb shear strength parameters calculated for limiting equilibrium of slide masses are also presented.

This paper was derived largely from reports by Fleming, et al (1970), Omaha District (1972), and Hamel (1973). These reports contain more detailed information on Bearpaw shale, the Fort Peck project, and the powerhouse slope. Jaspar & Peters (1979) present additional information on Bearpaw shale at Gardiner Dam 230 mi. north of Fort Peck in Saskatchewan, Canada.

*English units are used throughout this paper as they were the units used in the United States during work on the Fort Peck Dam project and the powerhouse slope.

## PROJECT LOCATION AND DESCRIPTION

### General

Fort Peck Dam is a multipurpose dam on the Missouri River in northeastern Montana about 70 mi. south of the Canadian border. The embankment dam has a crest length of 4 mi. (including a 2 mi. dike section), a maximum height of 250 ft, a maximum base width of 3500 ft, and a crest width of 50 ft. This embankment contains 125,628,000 cu. yd. of material most of which was hydraulically placed. Construction of Fort Peck Dam began in 1933 and was essentially completed in 1940.

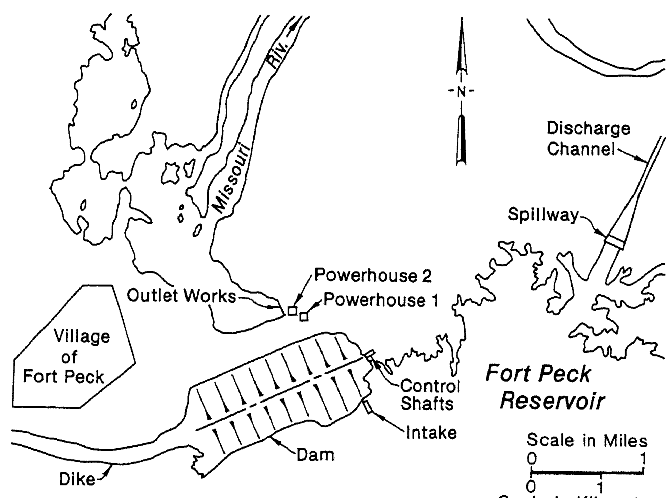


Figure 1 - Plan - Fort Peck Area

The spillway is located 3 mi. east (right) of the dam (Fig. 1). The powerhouses are downstream of the dam on the east side of the valley. Powerhouse 1 was started in 1940 and, due to World War II, not completed until 1952. Powerhouse 2 was constructed from 1957 to 1961.

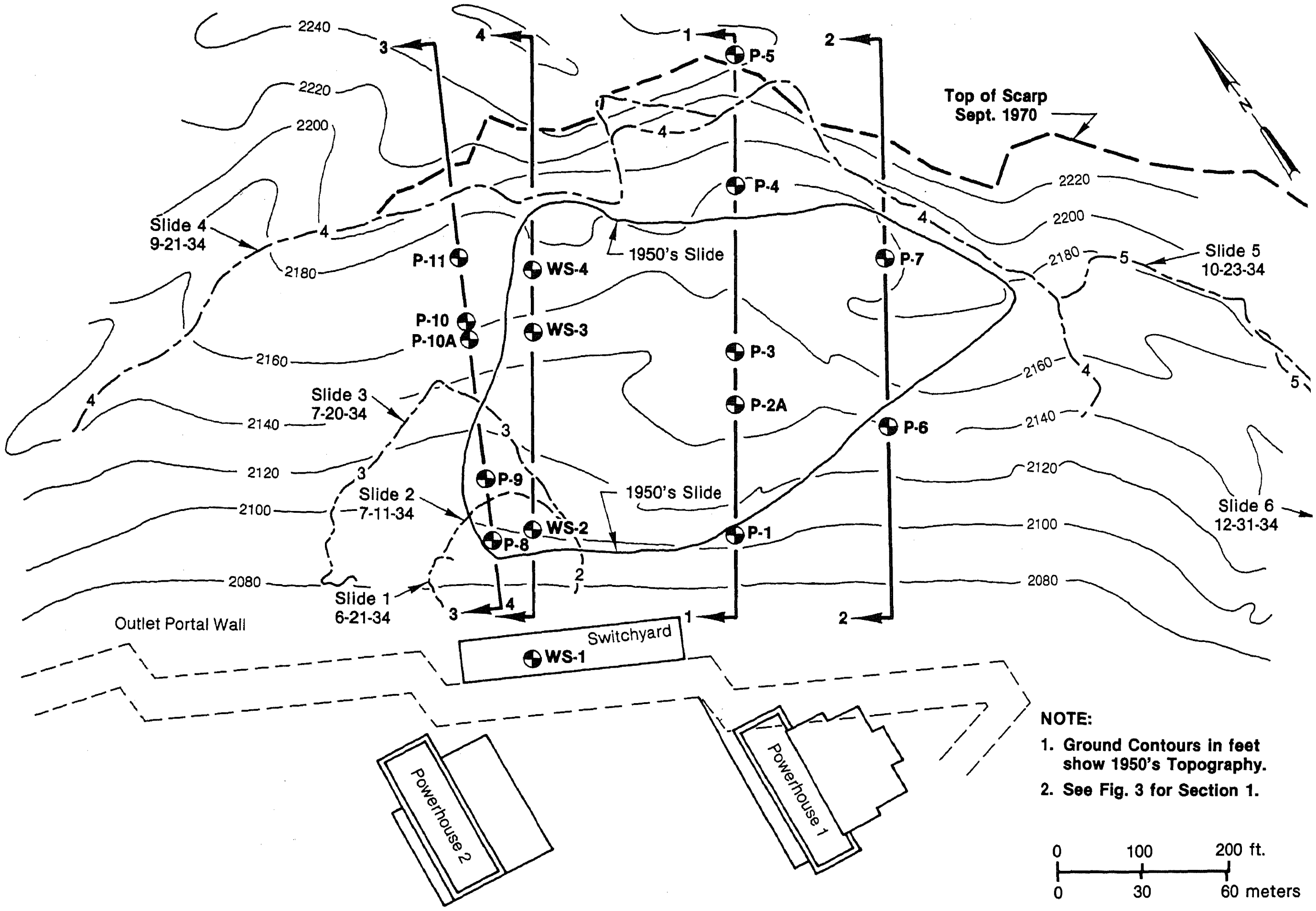


Figure 2 - Plan - Fort Peck Powerhouse Slope

Four 24.7 ft inside diameter reinforced concrete lined tunnels extend through the right abutment of the dam from control shafts near the dam axis (Fig. 1). Two tunnels are used for power and two are used for reservoir discharge. Tunnel construction began in 1934. Most of the tunnel length was driven through the right abutment but the outlet ends were constructed by cut and cover. A massive concrete gravity retaining wall was constructed in the open cut area along the tunnel outlet portals. The slope extending to a maximum height of about 200 ft above the outlet portal wall is the subject of this paper (Figs. 2 & 3).

## Climate

The climate at Fort Peck is harsh and dry. Sub-zero (°F) temperatures are common in winter and temperatures reach the 90's (°F) and higher in summer. From 1935-1968, the mean annual precipitation was 11.2 in. Winter snows are light; most of the precipitation falls as rain from May - July.

## Geology

Bedrock is the Bearpaw shale, an Upper Cretaceous age, compaction-type clay shale (or claystone) of marine origin. It is dark gray to black in color and rather poorly bedded. Seams of light colored bentonite from a fraction of an inch to 2 ft thick occur in th Bearpaw shale. Thin limestone beds, fossi and pyrite horizons, and concretion zones ar less numerous than bentonite seams. All o these features serve as stratigraphic markers The Bearpaw shale is about 1100 ft thick a Fort Peck and dips east at 3 ft/mi.

In its unweathered state, the Bearpaw shale i a firm clay shale which is usually considere a weak rock. Firm shale weathers readily o exposure, ultimately producing a clay soil o high plasticity. Weathering effects are mos pronounced at the surface but extend to depth of 30 - 50 ft. The highly weathered surficia zone is usually 10 - 20 ft thick. The severa foot thick transition zone from weathered t firm shale is termed sub-firm shale. Inde: and engineering properties of weathered an firm Bearpaw shale are given by Fleming, et a (1970) and Hamel (1973).

Undisturbed firm Bearpaw shale has a join pattern which was probably originall horizontal and vertical but is now bes described as irregular (Fleming, et al, 1970) The numerous gravity or normal faults in th powerhouse slope are probably associated wit stress relief effects and sliding of shal masses as the Missouri River entrenched it valley (Fleming, et al, 1970; Thomson

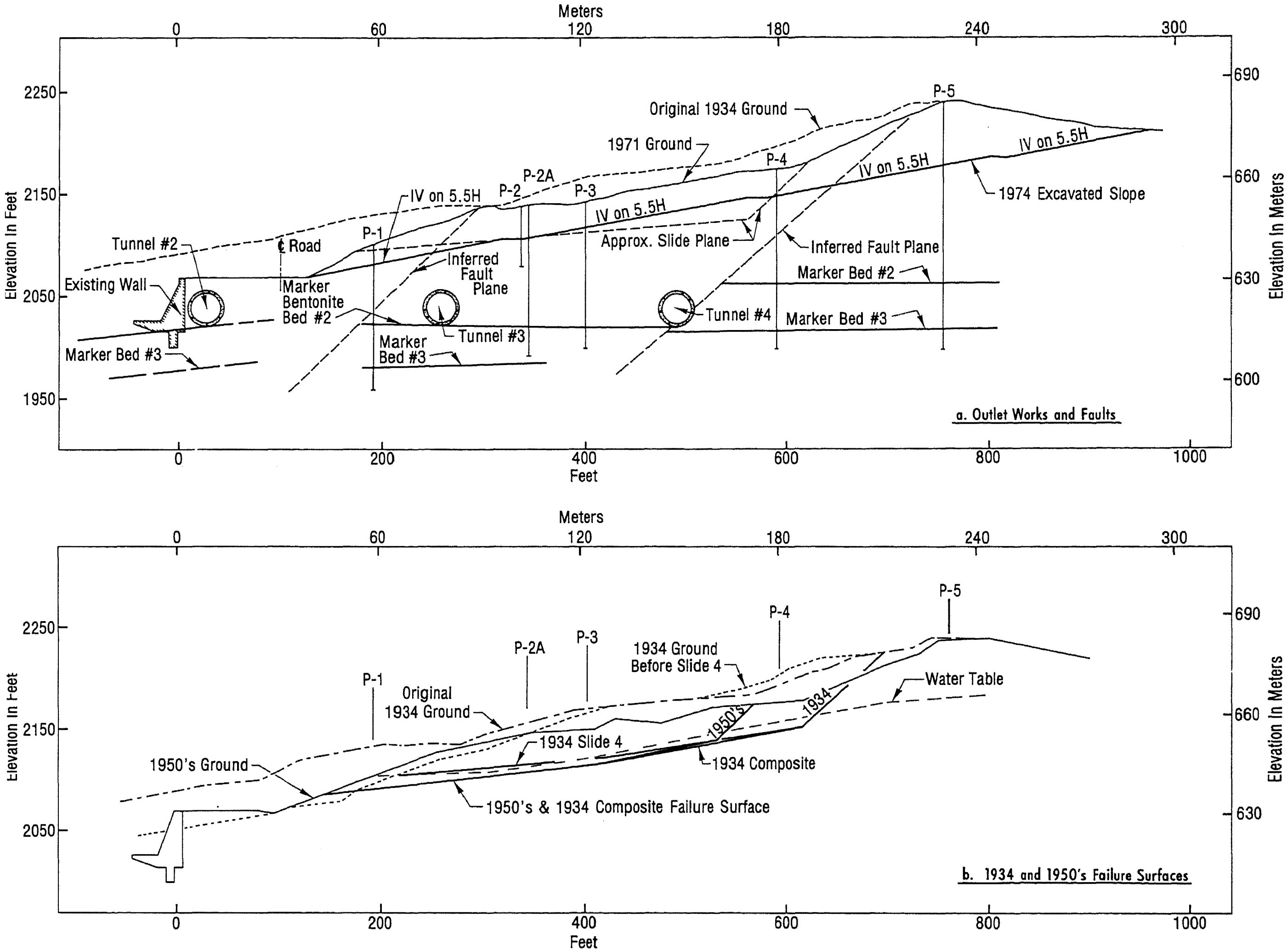


Figure 3 - Cross-Section 1
a. Outlet Works and Faults
b. 1934 and 1950's Failure Surfaces

Morgenstern, 1979; Ferguson & Hamel, 1981). These faults dip 35° - 60° with an average dip of about 45°.

Scrutiny of maps, cross-sections, and aerial photographs showing the original topography of the powerhouse slope indicates that old landslide masses existed there prior to the excavation for the outlet works. The original ground surface had a mean inclination of 1 on 5 (10°) in the lower part of the slope and the ground surface was hummocky.

## CONSTRUCTION AND MOVEMENT HISTORY

### History: 1933 - 1934

Subsurface exploration and initial clearing operations for Fort Peck Dam began in 1933. Subsurface exploration for the tunnels and outlet works was done in the spring and early summer of 1934. Borings were made and test pits were excavated but the logs are rather incomplete by modern standards.

Excavation for the outlet works began in June 1934. Weathered shale, sub-firm shale, and much of the firm shale was excavated by power shovels without blasting. Initial excavation was planned for a 1 on 1 slope. A cross-section drawing shows an initial excavation of 45 ft maximum depth at the slope toe near Powerhouse 2 (Fig. 2). There was a 24 ft wide bench at about mid-height of this excavation. The upper 30 ft of excavation sloped at 1 on 1.5 and the lower 30 ft sloped at 1 on 1.

Two small slides occurred during this initial excavation. The second of these was probably Slide 1 dated 21 June 1934 on available drawings (Fig. 2). Then it was decided to flatten the excavated slope to 1 on 3. Shortly thereafter, a major slide occurred. This was probably Slide 2 dated 11 July 1934 on available drawing and photo (Figs. 2 & 4). Slide 3 dated 20 July 1934 (Fig. 2) was an upslope and downstream extension of Slide 2.

Figure 4 - Aerial View of Slide 2, 10 July 1934
(Photo 34/706)

Additional slides occurred as excavation for the outlet works continued. Slide 4 dated 21 September 1934 involved much of the area upslope from the powerhouses (Figs. 2, 5, 6, & 7). Slides 5 and 6 dated 23 October and 31 December 1934, respectively, were upstream extensions of Slide 4 (Fig. 2).

All of the slides during slope excavation in 1934 were translational block slides. In each case, the basal failure surface was along a bentonite bed and the rear of the failure mass was defined by a fault. It is probable that 1934 Slides 1-6 all involved re-activation of ancient landslide masses.

In the outlet works excavation, maximum cuts of about 50 ft depth were made at the toe of the original slope north of Powerhouse 1. The average depth of material removed from the toe of the original slope was about 35 ft. Some 20 ft of material was also excavated from a fairly level area southwest of the toe of the original slope in the vicinity of the powerhouses. The center of the slope was unloaded further in late 1934 after Slide 4 and approximately the same time fill was placed in coulees on the slope. Information on this latter excavation and filling is very meager.

Figure 5 - Aerial View of Slide 4, 2 Oct 1934
(Photo 34/1204)

Cut and fill contours in Fig. 8 show the difference between pre-construction 1934 topography and April 1971 topography. These contours include topographic changes due to slide movements and slope grading from 1934 to 1971 as well as topographic changes due to 1934 construction activity. Despite this limitation, it seems likely that most of the topographic change indicated in Fig. 8 resulted from 1934 construction activity.

## History: 1935 - 1952

Little information is available on slope behavior from 1935 - 1952. Movement of slide debris on the slope continued during this period. Material was removed from the road ditch along the slope toe as required and there are records of such material removal in July and December 1943 and August 1944 (Omaha District, 1972). The surface of the slope was also graded periodically to seal cracks and improve surface drainage.

In August 1943, a line of iron pins was installed up the most active part of the slide along Section 1 (Figs. 2 & 3). Surveys of these iron pins showed downslope movements of 2 - 3 ft from July 1944 to March 1945 and downslope movements of 1 - 2 ft from March to September 1945 for the active portion of the slide (P-1 to P-4, Figs. 2 & 3). Total downslope movements for the 1.2 yr period from July 1944 to September 1945 ranged from 3 - 5 ft (Omaha District, 1972).

Figure 6 - Ground View of Slide 4, 2 Oct 1934
(Photo 34/1177)

History: 1952 - July 1970

Further investigations of the slope were done from 1952-59 in connection with design of Powerhouse 2. Twelve 6 in. diameter borings (P-1 through P-11, Fig. 2) were drilled from December 1952 to February 1953. Two piezometers, one shallow and one deep, were installed in most of these holes. The piezometers consisted of 2 in. diameter iron pipes. The deep piezometer pipe, which extended ssentially to the bottom of the hole, had eakened couplings at levels where the boring ogs indicated movement might occur. Shallow iezometers were installed to monitor water ressures along the failure surface inferred rom the boring logs.

ater levels were measured from 1953 - 1971 in nose piezometers which were not destroyed by lide activity or buried by slide debris )maha District, 1972; Hamel, 1973).

our inclinometer casings (WS-1 through WS-4, ig. 2) were installed in January and February 1957 prior to foundation excavation for Powerhouse 2. Plastic casings of 3-1/4 in. outside diameter were surrounded by lean cement grout in 6 in. diameter drill holes. A "Wilson-Shannon Tiltmeter" (early "Slope Indicator") was used to measure deflection of the plastic casings. Casing WS-1 was cut off and buried during construction of the switch yard of Powerhouse 2 in 1958 or 1959. By October 1959, casings WS-2, 3, and 4 were deformed at relatively shallow depths by slide movements. Water level measurements taken from 1958 - 1971 in the inclinometer casings were consistent with those in the adjacent piezometers.

Slope charts showing height vs. inclination relationships for stable and unstable slopes in Bearpaw shale at Fort Peck were prepared in 1953-56 during design of Powerhouse 2 (Lane, 1961). These charts indicated that an overall inclination of 1 on 6 would be necessary for stability of the powerhouse slope (Fleming, et al, 1970).

The limit of the 1950's slide area is shown in Fig. 2. This slide continued to move, at least intermittently, through July 1970 (Hamel, 1973). The slide movements caused no distress to the powerhouses or related facilities. Slide debris was simply removed as necessary from the road and ditch along the slope toe.

Figure 7 - Slickensided Scarp of Slide 4,
2 Oct 1934 (Photo 34/1184)

History: August 1970 - 1973

From August-November 1970, the lower part of the powerhouse slope was graded, a storm drain was installed along the portion of the slope toe downstream from Powerhouse 2, and the road behind the outlet portal wall was graded and paved. Grading and removal of material from the lower part of the slope initiated new slide movements with resultant scrutiny and monitoring of the powerhouse slope (Hamel, 1973; Omaha District, 1972). Only some of the more significant surface movement data can be presented here. The new slide movements initiated in August and September 1970 caused no distress to the powerhouses or related facilities.

In September 1970, the top of the main slide scarp in the powerhouse slope was surveyed. The top of scarp in September 1970 was considerably upslope from the outlines of 1934 Slides 4, 5, and 6 in the central and upstream parts of the slope (Fig. 2). If it is assumed that the 1934 slide outlines represent the tops of the 1934 slide scarps, the retreat of the slide scarp from 1934 to 1970 can be determined (Hamel, 1973).

The scarp retreat was scaled from 1 in. = 50 ft drawings of the slope at ten stations about 100 ft apart. Scarp retreat ranged from zero halfway between Sections 1 and 2 to 200 ft at the upstream edge of Slide 4 (Fig. 2). The average scarp retreat at the 10 stations was 86 ft for the 36 yr. period from 1934-1970. Scarp retreat was probably episodic, from discrete slide movements, rather than steady, from continuing processes, during this time. Nevertheless, the data imply annual rates of scarp retreat of 0 to 5.6 ft/yr, averaging 2.4 ft/yr for the 10 stations (Hamel, 1973).

The tops of the surviving piezometer and inclinometer pipes (Fig. 8) were surveyed in September and December 1970 and in July and December 1971 (Hamel, 1973; Omaha District, 1972). The major components of top of pipe movement were downslope though many pipes had significant lateral movements and some pipes had appreciable downward movement. Pipes in the toe of the active slide generally showed larger movements than pipes further upslope. Maximum movements were at P-2A and P-3 (Figs. 2, 3 & 8), both of which moved downslope 36 ft from February 1953 to September 1970 and another 2 ft from September 1970 to December 1971.

Even though downslope movements of slide debris on the powerhouse slope probably occurred in episodic increments rather than as steady creep, average annual movement rates for the tops of pipes were computed (Hamel, 1973; Omaha District, 1972). This provided a basis for comparing movements for different locations and time periods and also for extrapolating future movements of slide debris.

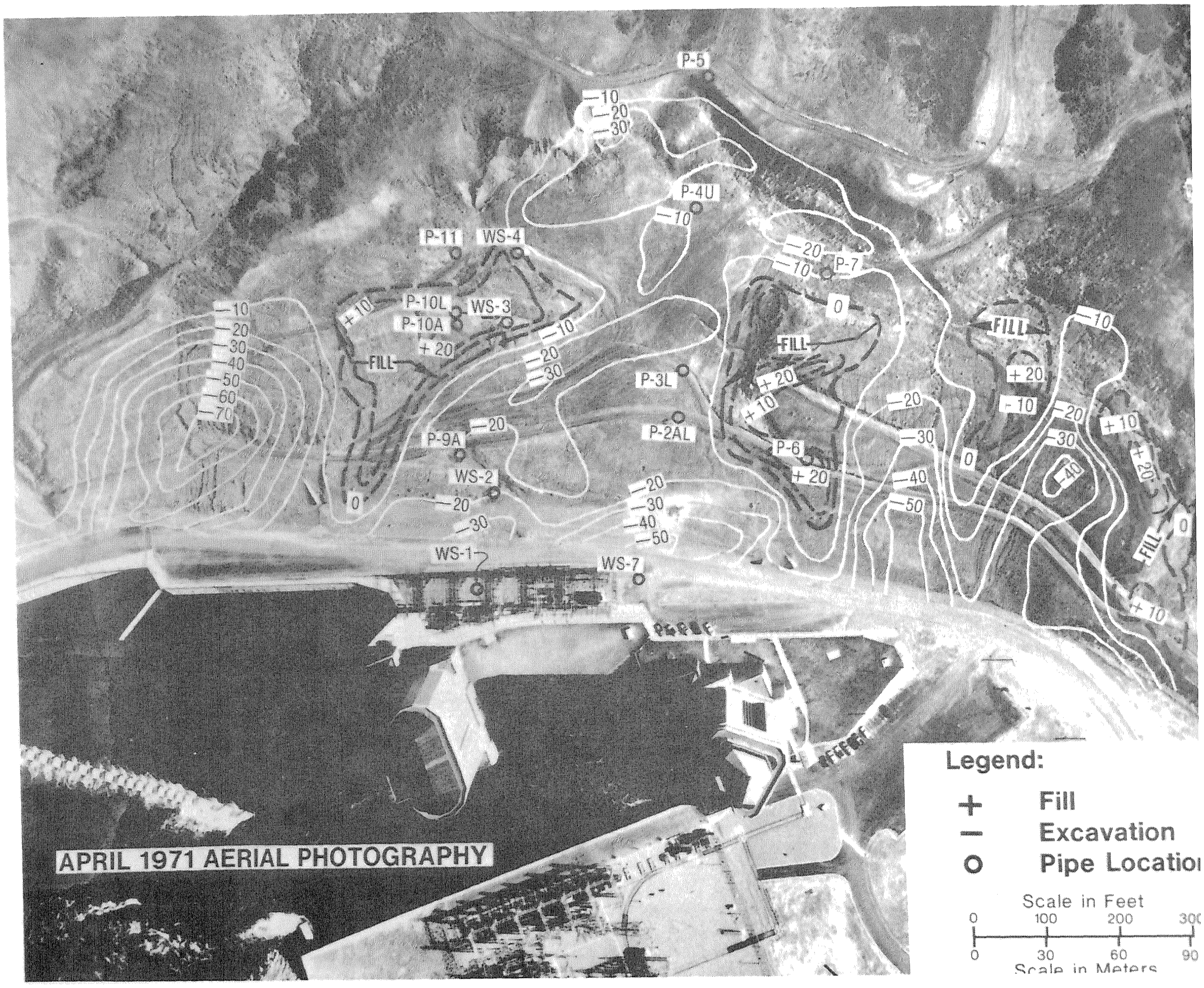


Figure 8 - Cut and Fill Contour Map (Pl. 16, DM MFP-116) Contours in Feet

Contours of average annual downslope movement rates of tops of pipes in the powerhouse slope are shown in Fig. 9. For piezometers, rates were computed for the 18.7 yr. period from February 1953 to December 1971. For inclinometer casings, rates were computed for the 14.7 yr. period from February 1957 to December 1971. Average annual movement rates from pipe installation to December 1971 were virtually identical to those from installation to September 1970.

The contours in Fig. 9 show the general movement pattern of slide debris on the powerhouse slope over nearly two decades. Maximum movement rates of about 2 ft/yr. occurred in the active toe portion of the slide at and near Section 1 (Figs. 2 & 3). Further downstream, movement rates of 1.0-1.5 ft/yr. occurred at and near the downslope portions of Sections 3 and 4 (Fig. 2). Section 2 at the upstream edge of the active slide (Fig. 2) was relatively stable with movement rates of 0.1 - 0.2 ft/yr.

These data suggest average long-term movem rates of 1-2 ft/yr. for Bearpaw shale col vium which has not reached a state of qua equilibrium and average long-term movem rates about an order of magnitude slower Bearpaw shale colluvium which has reac quasi-equilibrium. These long-term movem rates are similar to those reported by ot investigators for clay shale slide deb (Hamel, 1973).

Comparison of Figs. 8 and 9 indicates t areas of maximum slope movement correlate w zones of maximum excavation at the toe of original slope. This comparison also sugge that fill placement may have increased stab ity of portions of the powerhouse slope, e. by buttressing unstable colluvial masses reducing surface water infiltration.

Stabilization measures for the powerho slope were investigated from 1970 - 19 (Hamel, 1973; Omaha District, 1972). The c feasible stabilization measure involved fl

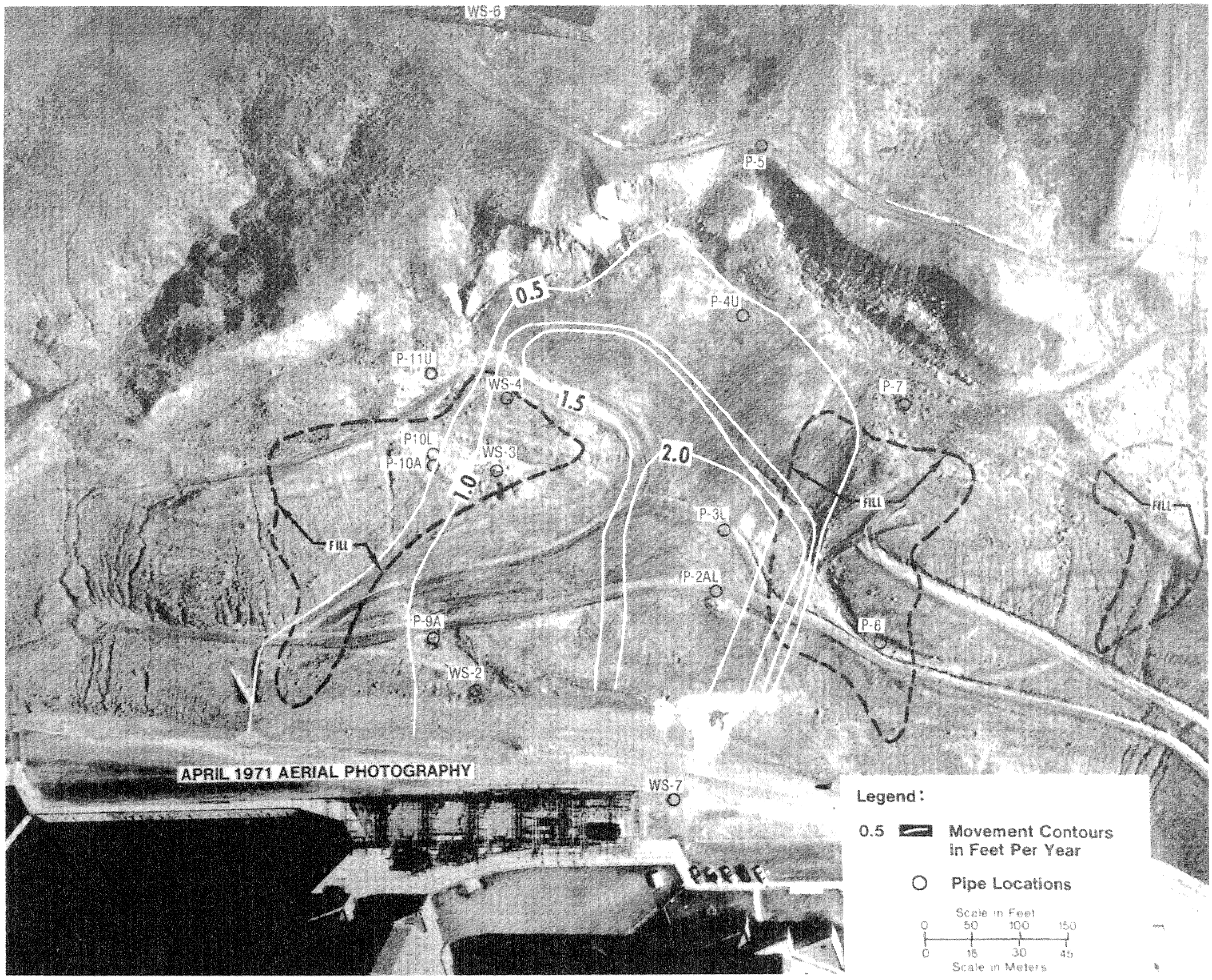


Figure 9 - Movement Contour Map (Pl. 17, DM MFP-116)

tening the slope to an overall inclination of about 1 on 6 in the active slide area (Fig. 2) with somewhat steeper inclinations for distances several hundred feet upstream and downstream from the active slide area. Grading plans indicated this would involve some 1.6 - 1.8 x $10^6$ cu. yd. of excavation.

History: 1973 - 1983

A contract was let and the powerhouse slope was excavated in 1973 - 1974. The contractor used a bucket wheel excavator (Holland loader) and spoil was placed in disposal areas downstream from the powerhouse slope. The 1973 contract unit price for excavation and disposal was $0.37 per cu. yd., lower than the $0.39 per cu. yd. price for a larger contract for the spillway excavation in 1934. (The original 1934 excavation of the powerhouse slope was by Government hired labor.) This unit cost reduction resulted mainly from the larger and more efficient earthmoving equipment in 1974.

Powerhouse slope excavation was done from September 1973 to July 1974. A total of 1.62 x $10^6$ cu. yd. was removed. The active slide area was graded to an overall inclination of 1 on 6. Berms and ditches were provided for surface drainage and attempts were made to establish a vegetative cover. The vegetative cover did not flourish due to poor soil conditions and limited rainfall and there has been some surface erosion of the graded slope over the past decade. Overall, the powerhouse slope appears to have remained stable since 1974.

## CALCULATION OF SHEAR STRENGTH FROM SLIDES

### General

The Morgenstern-Price (1965) method of stability analysis was used to calculate effective stress Mohr-Coulomb shear strength parameters required for limiting equilibrium of 1934 and 1950's slides for each of the four cross-sections in Fig. 2. Failure surfaces analyzed for Section 1 are shown in Fig. 3b. Hamel (1973) gives failure surfaces for the other cross-sections, data on failure mass geometry and groundwater conditions, and strength parameters calculated for equilibrium.

Groundwater levels measured in piezometers and inclinometer casings in the 1950's were used in strength calculations. Water tables were typically parallel to and only a few feet above basal sliding surfaces (Fig. 3b). It is likely that these water tables were perched on the basal sliding surfaces.

### 1950's Slide

The 1950's slide was analyzed first because information on failure mass geometry and groundwater conditions was more complete for this slide. Zero cohesion friction angles calculated for limiting equilibrium of the 1950's slide were 12° at Section 1, 10.5° at Section 2, and 11.5° at Sections 3 and 4. Shear stress $\tau$ and effective normal stress $\sigma'$ values calculated from strength parameters required for limiting equilibrium of the 1950's slide are plotted in Fig. 10. Two Mohr-Coulomb failure envelopes can be fitted to the data. One envelope has c' = 0.1 ksf, $\phi$ ' = 10°; the other has c' = 0, $\phi$ ' = 11.5°. Both envelopes apply to effective normal stresses on the order of 3 to 4 ksf.

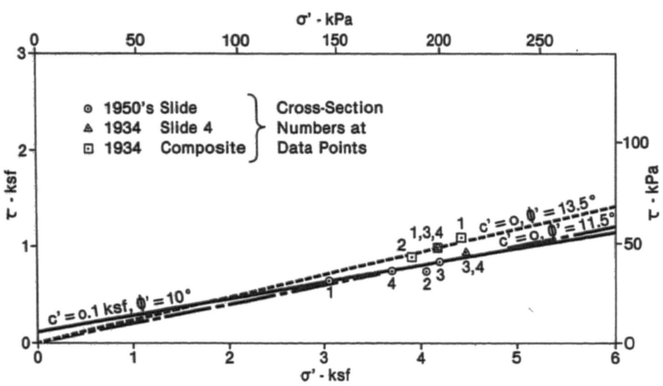


Figure 10 - Mohr-Coulomb Shear Strength Envelopes from 1934 and 1950's Slides

### 1934 Slides

Strength values were calculated for limiting equilibrium of 1934 Slide 4 at Sections 1, 3, and 4 (Fig. 2). Slide 4 was not analyzed at Section 2 where its toe location was not reliably known. A composite 1934 failure surface was also analyzed for Sections 1-4. These composite failure surfaces were believed to be those along which much of the 1934 (and earlier) slide movement occurred. Composite 1934 failure surfaces were generally close to the 1950's failure surfaces (Fig. 3b; Hamel, 1973).

Ground profiles after 1934 Slide 3 but before 1934 Slide 4 were used in analyzing all 1934 slides. The 1950's water tables were used in these analyses. It was considered likely that 1934 groundwater conditions were similar to those observed from 1953-71.

Zero cohesion friction angles calculated equilibrium of 1934 Slide 4 were 13.5° Section 1 and 12° at Sections 3 and 4. Z cohesion friction angles calculated equilibrium along the composite 1934 fail surfaces were 14° at Section 1, 13° at Sect 2, and 13.5° at Sections 3 and 4. Shear effective normal stress values calculated f strength parameters required for limit equilibrium of 1934 slides are plotted in F 10. These data are well fitted by a Mc Coulomb failure envelope with c' = 0, $\phi$ 13.5° for effective normal stresses on order of 4 ksf.

### Discussion

Shear strengths calculated from the 1 slides are slightly higher than those c culated from the 1950's slides (Fig. 10). failure surfaces of the 1934 slides must h existed in the slope and experienced la movements prior to 1934.* It is possible t the failure surfaces "healed" somewhat dur a period of geologic time prior to 1934, e. by consolidation and/or formation of mine precipitates. It is also possible that 1934 slides involved movement directi slightly different from those in the anci landslides and that 1934 and later moveme further reduced shear strengths in the sliding directions. Alternatively, the c ference between shear strengths calcula from the 1934 and 1950's slides may si have resulted from inaccuracies in the topography and groundwater levels which not as reliably known as those for the 19 slides.

The shear strength calculated from the 19 slides is considered the field resi strength of Bearpaw shale colluvium on powerhouse slope. This field resi strength characterized by c' = 0.1 ksf, 10° or c' = 0, $\phi$ = 11.5° for $\sigma'$ = 3-4 (Fig. 10) is significantly larger than lal atory residual strength of Bearpaw sh Laboratory residual c' = 0, $\phi$ ' = 4°-7° $\sigma'$ = 8-16 ksf and c' = 0, $\phi$ ' = 6°-8° fo = 3-24 ksf were reported for Bearpaw s from the Fort Peck area by Fleming, et (1970) and Townsend & Gilbert (19 respectively.

It seems unlikely that a field-scale fai surface in Bearpaw shale colluvium would be as homogeneous, smooth, or coated oriented clay particles as the failure sur of a small specimen in a laboratory resi strength test.

Shear strengths calculated from the 19 slides on the powerhouse slope are equal upper bound shear strengths calculated unstable Bearpaw shale slopes at Gardiner

*Stability analyses with pre-excavation ground profiles, c' = 0, $\phi$ ' = 11.5°, 1950's water tables indicate the powerh slope was only marginally stable, especi at Sections 1 and 2 (Fig. 2), prior to exc tion for the outlet works (Hamel, 1973).

Saskatchewan. There c' = 0.00-0.06 ksf, = 5.0°-10.4° (Fleming, et al, 1970) and = 0, $\phi'$ = 9°-12° (Jaspar & Peters, 1979) calculated for equilibrium of several able slopes with effective normal stresses arable to those in the Fort Peck powerse slope. The upper bound strengths at liner Dam were calculated from slopes dised by excavations. Much lower strengths ar to be operable in the Bearpaw shale dation of Gardiner Dam and it is not ently clear whether strengths calculated unstable slopes are applicable to dation behavior in such cases (Jaspar & rs, 1979).

## ARY AND CONCLUSIONS

lslides occurred in the Bearpaw shale slope acent to the powerhouses at Fort Peck Dam the geologic past as the Missouri River renched its valley. The existence and imcations of these ancient landslides were recognized fifty years ago when the reserr outlet works were designed and construct- Excavation of the toes of ancient landde masses in 1934 caused progressive lure of colluvium on the slope. Part of s colluvium was excavated in 1934. Collum left on the slope continued to move until 4 but these movements did not cause any tress to the powerhouses or other facilis. In 1974, the slope was stabilized by avating 1.6 x $10^6$ cu. yd. of material.

erience with the Fort Peck powerhouse slope consistent with that involving huge luvial masses elsewhere. Recognition of h masses prior to construction is of amount importance. Disturbance of such ses by construction activities generally ses progressive movements which are ficult, if not impossible, to stop except removal of most, if not all, of the slide ris. In many cases, it is possible to live h large movements of colluvial masses as g as they do not cause distress to struces or facilities.

ement of Bearpaw shale colluvium on the t Peck powerhouse slope from 1934-1974 was sodic in response to excavation, climatic ditions, and perhaps other factors. The ive slide area moved about 4 ft/yr in 1944 1945. Average long term movement rates m 1953 - 1971 were 1-2 ft/yr for the more ive slide areas and 0.1-0.2 ft/yr for the e stable slide areas. The field residual ar strength of Bearpaw shale colluvium on powerhouse slope is given by c' = 0.1 ksf, ' = 10° or c' = 0, $\phi'$ = 11.5° for effece normal stresses of 3-4 ksf. This field idual strength significantly exceeds oratory residual strengths.

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