80th Anniversary of the Fort Peck Dam Construction Slide

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Abstract - Beginning at about 1:15 PM on September 22, 1938, the upstream slope of the dam experienced a large failure as the construction work had progressed to within 20 feet of the final dam crest elevation. One hundred eighty men were working in the area. Thirty four men were injured. Eight men lost their lives, six of whom were never found and are buried somewhere in the dam.

As a result of this slope failure, the original design Board of Consultants was expanded to assess the cause of the failure, and review options for completing the dam construction including repairs in the failure area. There were many lessons learned. The failure as well as other performance factors during first filling of the reservoir altered the U.S. Army Corps of Engineers (USACE) design and construction practices significantly. The failure also imparted important lessons to the U.S. community of practice for dams and levees.

This paper is written in two parts. Part I covers the background of the project through the reconstruction of dam in the area where the slide occurred and the early performance of the dam following first filling of the reservoir. Part I concludes with the controversies that the expanded board of Consultants encountered in preparing a report on the cause of failure, and in completing their review of the redesign of the section of the dam requiring repair. Part II explores the broader Technology and Human Factors that influenced the design and post failure investigation. This part begins with a discussion of the differences of opinions about technical matters that were revealed in the project literature. It presents a summary of the state of the practice at the time of design and construction, and briefly traces technology advances during the next 40 years following the slide. This includes a discussion of evolution of technology related to the shear strength of shales, and the liquefaction of sands.

I. THE SLIDE AND PROJECT IMPACTS TO THE USACE

A. Background

Fort Peck dam was first conceptualized in 1928 under a program known as House Document 308. This program called for comprehensive engineering studies of navigable streams in the United States to investigate the feasibility of basin development including navigation, irrigation, control of floods, as well as water power. The Corps' Kansas City District office submitted a report to the Division Engineer in St. Louis in 1932, advocating a site near a former 1867 trading post in Montana.

The construction of Fort Peck Dam in Montana was authorized by President Franklin Roosevelt as part of the New Deal on October 15, 1933, to overcome the Great Depression. Funds were quickly released to begin work on the project, and on October 23, 1933 seventy men began work at the dam site. Fort Peck became the single largest project of the New Deal, employing more than 10,500 people at the peak of construction and an estimated 40,000 to 50,000 workers over the course of the seven years of construction. At its completion, Fort Peck Dam was the largest dam by volume in the world, five times more voluminous than the next biggest dam. An oblique aerial photograph of the failure is shown on Figure 1.

Figure 1: Photograph of the Fort Peck Slide on Sept. 22, 1938

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Figure 2 was taken from a 1964 Tezaghi Lecture paper by Arthur Casagrande and shows a plan view of part of the dam before and after the partial failure illustrating the location and extent of the slide. A profile of the dam (looking downstream) showing the relative location of the upstream slide and the foundation conditions along the dam axis is shown on Figure 3. Likewise, Figure 4 contains two cross-sections of the dam at Station 22 drawn at different scales showing the dam configuration both before and just after the slide occurred.

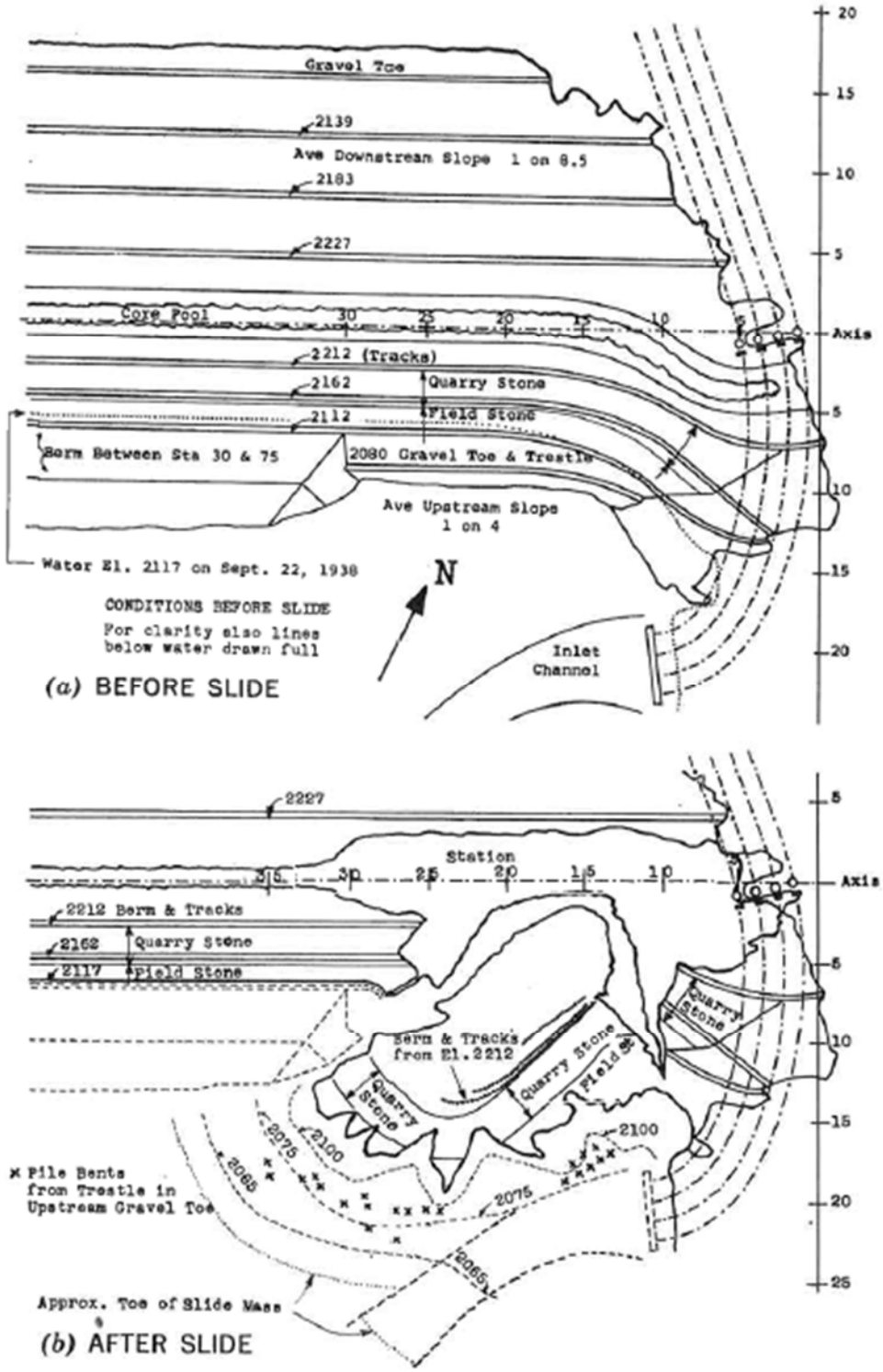

Figure 2. Plan View of Dam near Right Abutment (a) before and (b) after slide

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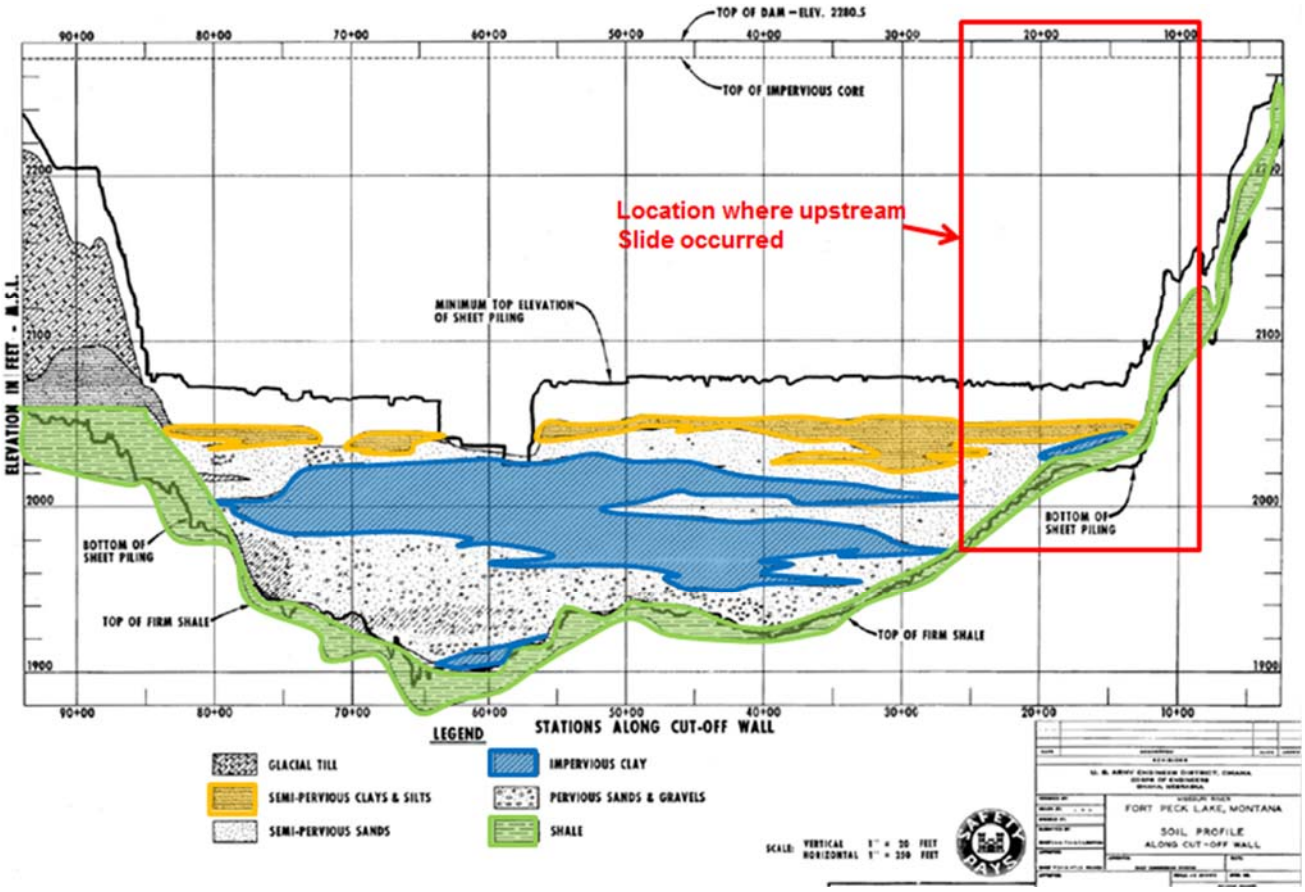

Figure 3: Profile along Dam Axis showing Location of the Fort Peck Dam Slide (Vertical scale exaggerated 12.5 times)

B. Embankment Design and Construction

Hydraulic filling was selected as the most cost effective option to construct the embankment dam, otherwise the cost of building the dam would not have been economically justifiable. Hydraulic fill construction methods at the site involved excavating materials from at or below the river level with dredging methods and pumping the water/solids mixture through pipes as a slurry to the upstream and downstream edges of the embankment and discharging the mixture to the central portion of the dam. Heavier particles settle out in the outer portion of the hydraulic fill area with finer silts and clays settling out in a central pool area to create a core that becomes impervious once the water drains out through adjacent, porous layers.

Limited documents have been located regarding the original designers and processes. The District Engineer, Major T.B. Larkin, arrived on November 21, 1933. Major Clark Kittrell succeeded Lieutenant Larkin as the District Engineer on July 14, 1937 and served until June 1940.

Due to the fast tracked design and construction schedule and multiple project features, the authors believe the design process was dynamic and involved numerous engineers, technicians, and draftsman under the supervision of the Fort Peck District Engineer. Engineering experts from the Missouri River Division and HQUSACE offices would have been very involved with engineering decisions as well.

A Board of Consulting Engineers was employed to advise on designs and construction methods. This Board included (USACE, 1939):

William Gerig - USACE, Railroads, Dams, Panama Canal
D.C. Henny - US Bureau of Reclamation, Blankets and Cutoffs
Louis C. Hill - US Bureau of Reclamation, Consultant to USACE on Tygart Dam
Warren J. Mead - Professor, Head of M.I.T. Geology, on Hoover Dam "CRB"
Thaddeus Merriman - Consulting Engineer, Hydraulics, Chief of Catskill Aqueduct

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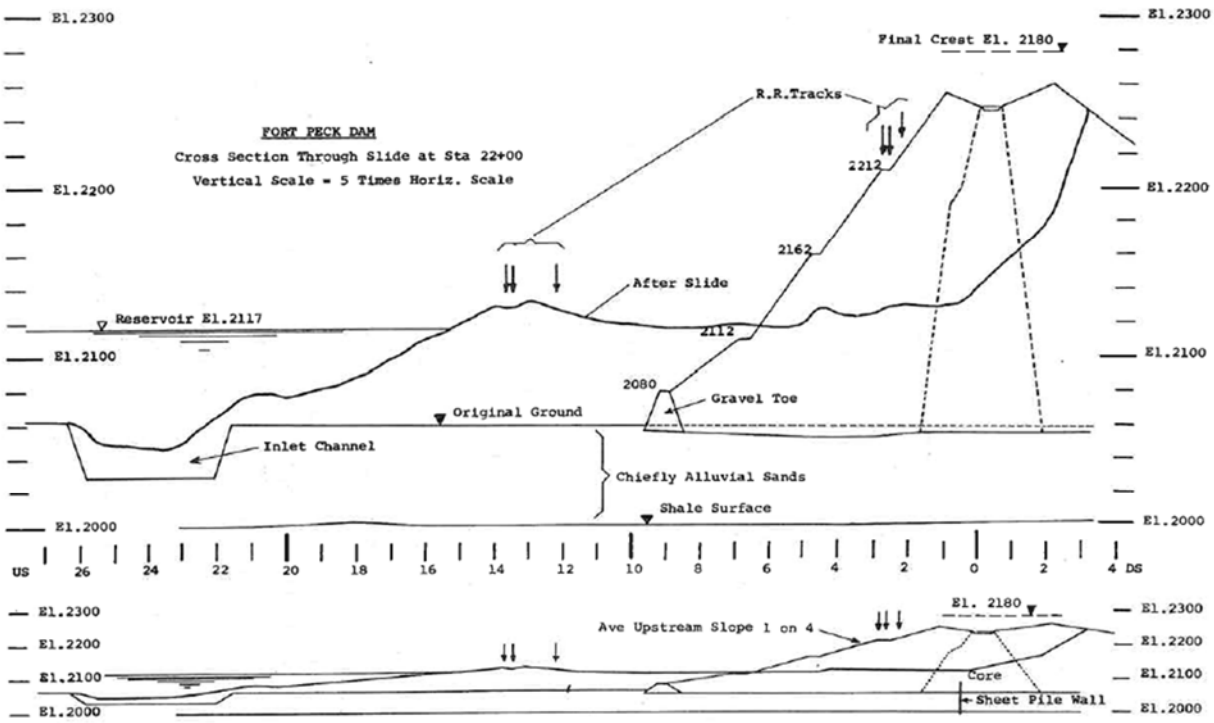

Figure 4. Cross-section of Dam at Station 22+00 before and after the slide at two different scales

Complete design reports are not available. However, much can be extracted from the plan of the right side of the dam shown on Figure 2A, as well as the cross section information shown on Figure 5. Other important details were obtained from a review of the site, information from the geology profile shown in Figure 3, as well as details from the design of the appurtenant structures.

Nearly all of the excavation materials at the site for the spillway, outlet works tunnels and shafts were described as firm shale, weathered shale and impervious clays. All derived from the clay shale of the Bear Paw formation. These materials would be difficult to construct with and would make poor embankment dam materials which is likely why they were not included in the embankment fill.

The valley floor in the vicinity of the dam site contained an ample supply of alluvium that supplied all the materials needed to engineer and construct a large zoned embankment, albeit most of the soils were saturated and below the river level. The abundant source of wet materials and the agency experience with dredging likely led to the selection of a hydraulic fill dam. The successful design and construction of Gatun Dam on the Panama Canal, of which William Gerig had worked on both the design and construction, may have factored into this decision as well. Using this method and the broadly graded alluvial borrows, a zoned embankment (considered a more robust cross section than a homogeneous embankment), could be constructed at the site.

The specifications required an average effective grain size and average clay content for each of the six zones. As shown on the design cross section (Figure 5), the design required a massive zoned embankment be constructed for such an important structure in the plan for the Missouri River basin development. An average downstream slope of 8.5:1(horizontal: vertical) or about a slope of 7 degrees and an upstream slope averaging about 4:1 or about 14 degrees were specified. By today's standards, both of these slopes are considered flat for a zoned embankment dam. The large difference in slopes is not typical and the reason for the difference has not been found by the authors. The sheet pile cutoff was included in the design to reduce under seepage.

Fundamental data on the embankment and foundation are included in the Embankment Operations and Maintenance (O&M) Manual issued in September 1957. The O&M Manual states*: "No major problems, other than the upstream face protection, can be anticipated for the near future."* This infers a high degree of confidence in the final constructed project. However, the opening paragraph also adds a precautionary statement: *"It is essential that nothing happens that would endanger the safety of the dam or its efficient operations. Forces or conditions that tend to weaken or destroy the dam must be detected and recognized years before they become serious. The proper operation of the dam will always require engineers who have a basic understanding of the engineering principles involved."*

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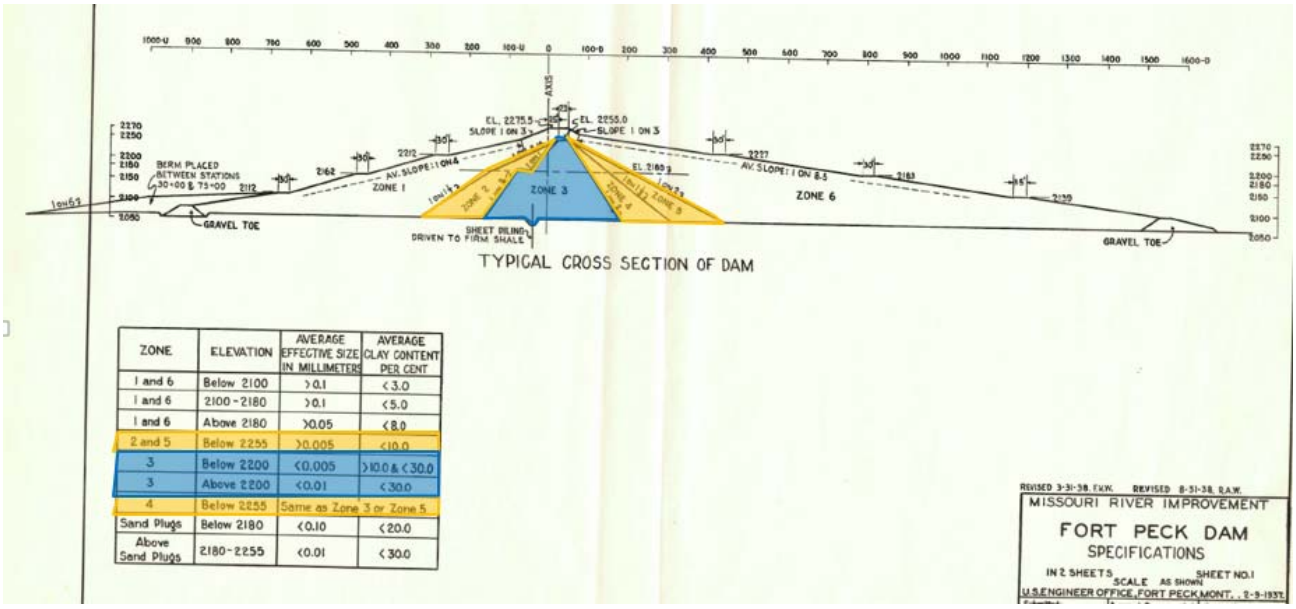

ZONE	ELEVATION	AVERAGE EFFECTIVE SIZE IN MILLIMETERS	AVERAGE CLAY CONTENT PER CENT
1 and 6	Below 2100	>0.1	<3.0
1 and 6	2100-2180	>0.1	<5.0
1 and 6	Above 2180	>0.05	<8.0
2 and 5	Below 2255	>0.005	<10.0
3	Below 2200	<0.005	>10.0 & <30.0
3	Above 2200	<0.01	<30.0
4	Below 2255	Same as Zone 3 or Zone 5	
Sand Plugs	Below 2180	<0.10	<20.0
Above Sand Plugs	2180-2255	<0.01	<30.0

Figure 5. Cross Section for Dam from Specifications dated 2-9-1937

The embankment dam is approximately four miles in length, including the two-mile dike section, with a maximum height of 250.5 feet, a maximum base width of 4,900 feet, and a crest width of 50 feet. The upstream face of the dam is protected by quarry stone placed on an 18-inch thick gravel blanket. A gravel toe was installed along both the upstream and downstream edges of the embankment fill to provide the initial dikes to contain the hydraulic fill and protection for through seepage at the downstream toe. Under seepage control was provided by a steel sheet pile cut-off wall located 37.5 feet upstream of the central dam axis. A system of relief wells was later installed along the downstream toe due to high pore water pressures that developed in the foundation during first filling.

The first construction contract was awarded to the E.J. Longyear Exploration Company on July 18, 1933 and completed on December 15, 1933. The work included preliminary site exploration for the dam and dike extension, emphasizing obtaining cores of the shale foundation. Drilling information did not address the competency of the shale in much detail. Construction of the bridges, dredging equipment, town buildings, and concrete lined diversion tunnels began in 1934. Stripping the surface beneath the footprint of the dam began in June and the sheet pile cutoff wall was started in August 1934. The term "stripping" was used but it entailed the removal of clays and silts down to the underlying pervious and stable sands due to concerns for construction stability. This concern was likely due to end of construction stability issues experienced at a couple of earlier dams built in Ohio by the USACE.

Placement of the first hydraulic fill between the "gravel toe" dikes was started on October 13, 1934, as soon as the first dredging unit was completed. Work on multiple features proceeded at a frenzied pace for the next several years, with dredging seasons from April through November (depending on weather). The dam reached an elevation of 2223 feet above mean sea level on May 24, 1938. (The end of dredging operations was November 5, 1939, post slide.)

Soil profiles from the Diamond Drill contract are identified as the best source of information on soil conditions beneath the dam, with soil profile drawings generated in 1935 or early 1936. A glimpse of the geotechnical basis of the design is provided in a paper by T.A. Middlebrooks (1936), then an Associate Engineer for the Fort Peck Engineer District located at Fort Peck, Montana. This paper describes a number of the explorations, sampling, testing and analysis methods used to design the closure section of the dam. The designers were particularly concerned about the behavior of the impervious clays within the foundation alluvium that can be seen on Figure 3.

Fort Peck Dam construction included the largest, deepest or "first of a kind" for many features (Nothing came easily!). For example, the sheet pile cutoff was the deepest constructed at the time. It was driven down through up to 160 feet of river alluviums to firm shale. This required splicing of sheets, a custom crane, custom leads to guide the piles and resulted in the invention of a water jet to install sheets to this depth. To be successful at this site, the hydraulic fill required the largest pumps ever installed in dredges up to this point in time. The materials were pumped over four miles with a lift of over 250 feet. The solids comprised only about 13% of the volume of the slurry. The rate of fill placement was amazing for that time in history, with over 28,000,000 cubic yards of fill placed in 1936, 54,000,000 cubic yards in 1937 and by fall of 1938 a total of nearly 100 million cubic yards had been completed. Routine testing was done as the fill placement was proceeding to verify the effective grain size of materials met specifications in the six zones designed into the dam.

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Figure 6. Aerial view of the dam construction in 1937 shortly after completion of the closure section and diversion through the outlet works tunnels.

Needless to say it was a difficult site to build a dam and the builders were tough people. In 1934, the swing from the hottest day at the site to the coldest day was over 160 degrees Fahrenheit (minus 40 to over 120.) About 60 fatalities total occur over the 7 years that the dam was being built. While safety was emphasized, the standards were not what they are today.

C. The Slide

Closure of the main river channel (Figure 6) with river diversion through the tunnels was completed on 24 June 1937, allowing more aggressive mass placement of the embankment fill. Monitoring for distress or problems with the hydraulic fill was a daily routine. As the fill rose, workers routinely walked the pipelines on the dam to look for sags and daily reporting was required due to concern for overtopping the u/s bank. Laborers walked the pipelines continuously looking for breaks in the pipe. It appears the safety of the dam was taken seriously by all.

The USACE report on the slide (1939) notes the following:

"*On the morning of September 22, 1938, the usual inspection was made by the principal engineer in charge of construction, his assistant, the fill superintendent, the associate superintendent in charge of levees and the fill inspection force.*

At about 10 am, their findings were discussed at the conference held on the crest of the upstream face near station 15+10. The fill inspectors and the assistant superintendent of construction stated that there did not appear to be sufficient freeboard. An immediate inspection of that point disclosed, by rough measurements that the height of the bottom of the pipe line above the core pool was only 30 inches, whereas it should have been 4.5 feet. …. At about 11:45 am, the survey crew submitted the following data:

Station 15 – Pipe line 3 feet above the core pool; (should have been 4 1/2 feet)
Station 16 – 3 feet; (should have been 4 1/2 feet)
Station 17 – 2.8 feet (should have been 4 1/2 feet)"

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At this point in time the elevation of the core pool was 2252 feet mean sea level, the elevation of the reservoir was 2117.5 and the dam was nearing completion. When additional observations confirmed that the water level of the core pool had not changed from the day before, it was realized that the upstream embankment near the right abutment (east) was apparently settling. Project leader, Clark Kittrell went to the site in the early afternoon. His driver, Eugene Tourlotte, approached from the west and arrived at the site about 1:15 PM. Tourlotte saw the upstream shell begin to move out beneath the car, slammed on the brakes and went in reverse at a high speed to successfully outrun the slide. During the next ten minutes railroad tracks, trains, boats, pipelines, and thirty-four men were on the 1700 foot-wide mass as it slid. Over 5 million cubic yards of material came loose from the dam, and five percent of the structure was destroyed. When it came to rest, some of the equipment was submerged and eight men were dead, buried in the slide. Twenty-six men successfully rode out the slide.

The following is an excerpt of a report describing eyewitness accounts of the slide, as well as observations of the slide area:

"Piecing together these fragmentary descriptions, it is believed that the following statements are essentially accurate: The core pool began to settle, slowly at first and then more rapidly. Immediately following this, or simultaneously, cracks generally parallel to the axis were observed on the upstream face about 30 feet below the crest. Then portions of the upstream shell nearest the pool began to slide into the sinking core pool. In a lesser degree, similar cracks and sliding and slumping were taking place on the upstream portion of the downstream beach. Simultaneously with these developments, the main mass of the upstream shell, almost intact, was moving out into the reservoir, in a swing similar to that of a gate hinged at the east abutment. The cleavage line where the west end of this mass broke away from the dam was at approximately station 27+00. When this cleavage took place, the core-pool waters from both east and west poured out through this gap with incredible speed. Many witnesses agreed that the flow of the core-pool water was particularly noticeable east of the cleavage line. One man working on the pump barge said that, looking westward, it appeared as if the fill at the west end (of slide area, presumably) was falling into a big hole and that this hole progressed rapidly toward the pump boat and, as it reached the boat, the forward end dipped down sharply and then the boat slid into the hole and disappeared.

A tender and a jetting barge were carried away and buried in the rush of water from the core pool. Two draglines, four loadmasters, six tractors and other miscellaneous equipment went down with the slide.

Apparently, none of the eyewitnesses observed the action of the slide at the cleavage line at the time the west end broke away from the dam and started its hinging swing previously described. However, the conclusion that this break must have occurred within 1 or 2 minutes after the mass began its movement is unavoidable and is borne out by the statements of numerous witnesses…"

The slide could only be described as enormous. The surface area of the damage was about 160 acres. It was difficult to get perspective of the size from the ground so aerial photographs (see Figure 1) were taken and used to provide a backdrop for some of the forensic explorations and evaluations to come. To put it in perspective the slide volume was more than the entire volume of concrete placed in the Hoover dam and power plant that had been completed two years earlier in 1936. After the sobering search for bodies had ended, the construction of a ring dike and haul roads began to allow re-construction in the damaged area as well as the start of field investigations. The slide area needed to be characterized quickly in order for a design to be developed for the portion of the dam to be founded upon the slide mass. The quickly approaching onset of winter, the enormous surface area and volume of materials needing to be characterized and the complexity of the slide mass left no time to waste.

D. Post Slide Evaluation

Immediately after the slide the original Board was expanded to include:

Dr. Arthur Casagrande -	Professor of Soil Mechanics at Harvard University
Mr. I.B. Crosby -	Consulting Engineering Geologist
Dr. Glennon Gilboy -	Consulting Engineer, former Prof of Soil Mech, MIT
Mr. Joel D. Justin	Chairman, Consulting Engineer Phil. PA, co-author of "Engineering for Dams"
Mr. William H. McAlpine	Office of the Chief of (USACE) Engineers
Mr. C. W. Sturtevant	Division Engineer

Dr. Ralph Peck reported to one of the authors how excited Dr. Casagrande became when he received a request to serve on this Board as he suspected it may have been a case of liquefaction of the hydraulic fill. Dr. Casagrande (1936) had been testing for liquefaction of sands in the Harvard soils lab and had recently published a ground breaking article (1936) on "critical density" later referred to as "critical void ratio" of soils, a key consideration in the evaluation of potential for liquefaction, and was excited for the opportunity to consult on this case. Dr. Gilboy had been educated/trained by Dr. Karl Terzaghi, the father of soil mechanics and had written a paper on the stability of hydraulic fill dams earlier in the year. His relationship with Dr. Terzaghi and the paper on hydraulic fill dams likely influenced his inclusion to the Board.

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Three of the newly added board members, Dr's Casagrande and Gilboy and Mr. Crosby had been in a meeting just about three months before the slide discussing the direction the USACE was going regarding many subjects related to soil mechanics and embankment dam design.

The augmented Board was requested to make such visits to Fort Peck as were necessary to inspect the work and consult the records. They were requested to submit, upon completion of their first visit, a memorandum setting forth:

1. Any subsurface investigations or tests which were deemed advisable, other than those already scheduled.
2. Their opinions on the emergency program then under way to take care of the river flow during the spring run-off in 1939.

The information obtained from the slide investigation was to be submitted to the members of the Board as it became available, so that a full meeting could be held as soon as practicable to consider the following points:

1. Final design for reconstruction of the slide area.
2. Recommendations as to whether changes or additions were necessary or advisable in any other portions of the dam.
3. Recommendations as to whether any additional precautions were necessary or advisable in bringing the dam to final grade.
4. Determination of the cause of the slide.

Included in the early information provided to the board were the interviews of eye witnesses. The board poured through these accounts to piece together the facts about the timing of movements and other observations made by the stunned on lookers. The combined account is provided above.

The footprint of the slide area had rather limited previous subsurface investigations prior to the slide. Most information was obtained from near the dam centerline and in the closure area at the maximum section. Construction information came from the line of sheet piles and data from the tunnel inlet portals provided information at the upstream limit of the slide on the abutment shales.

Explorations included: seven Calyx holes, 200+ core drill holes, exploratory tunnels, surface mapping of the slide, peg model of the slide and a glass model of dam and foundation. Everything was "geologically" mapped including the location of where the drag lines were found. The limited subsurface information obtained prior to the slide was included with the information assembled to investigate the slide/foundation for the redesigned portion of the dam. For expediency, the soils in the slide mass were classified according to effective grain size and the coefficient of uniformity. The drawings mapped the location of displaced railroad trestles. The most obvious movement markers were the trestles themselves. Some of the trestle piles were pulled and their condition noted on the maps to help estimate the depth of the slide surface. Unbroken piles suggested the failure surface was below the bottom of the pile, broken piles may have indicated where the slide plane had passed through the piles. The broken piles lined up with the reports of some witnesses who described the first warning as the sound of timber trestles straining, creaking and breaking.

The Bear Paw shale gets more attention at this point in time since it is obviously involved in the slide. It is characterized in much more detail than "weathered" or not. The terms include:

<u>Decomposed shale</u> or shale clay - thoroughly disintegrated

<u>Weathered shale</u> - less modified zone 15-50' thick, containing oxidation in joint cracks and deposits of selenite or gypsum in seams

<u>Sub-firm shale -</u> is the zone 10-50' thick showing modification by water penetration, but not weathered enough to open new joints

<u>Firm shale</u> – underlying hard blue and substantially unaltered

It is interesting to note that the liquid limit of bentonite samples obtained and tested as part of the recent spillway work are in the range of 100 to 150 and the plasticity index is in the range of about 90 to 120. Some of the lab testing for the characterization and the design for the failure area included:

- Typical lab testing (sieves, water content, void determination, field compaction, permeability, mineralogical examinations)
- Lab shear tests on foundation materials including: Bentonite seams, and weathered shale
- Field tests on bentonite materials in place

All the above information and more fed into the back analysis of the slide and then the redesigned cross section.

E. Findings from Evaluations and Board of Consultants

The explorations shown on the maps revealed the absence of the weathered shale in many locations and hydraulic fill in the slide mass lying directly upon the sub-firm shale. This finding strongly suggested that the base slide plane was seated in

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the clay shale in these areas. The geologists mention that the inlet channel was practically filled with material transferred with the slide at two of the cross sections. At Station 22, they mention "the clay mound plowed up in front of the slide and the absence of weathered shale from a large portion of the foundation." These findings also suggested shear failure in the materials characterized as shale well out from the toe of the dam. Based on a profile along the alignment of the sheet pile cutoff, they conclude the most core material was missing at sta. 14 and much was in place left of that station and below the slide.

After less than six months after the slide occurred, the Board of Consultants arrived at the following conclusions on the cause of the slide:

> *"After careful consideration of all the pertinent data the Board has concluded that the slide in the upstream portion of the dam near the right abutment was due to the fact that the shearing resistance of the weathered shale and bentonite seams in the foundation was insufficient to withstand the shearing forces to which the foundation was subjected. The extent to which the slide progressed upstream, may have been due, in some degree, to a partial liquefaction of the material in the slide." March 2, 1939 Board Report*

This brief conclusion about the cause of the failure from the Board emphasizes the shear strength of the shale and bentonite seams in the foundation. It may seem obvious given the right portion of the slide was the only location where the dam was likely founded directly upon a shelf in the shale materials and this is where the first movements were detected. As the top of shale dipped beneath the left portion of the slide the shale became buried to a maximum depth of about 40 to 60 feet of alluvium at the left limit of the slide, at least on centerline. The back analysis that formed the basis of the redesign focused on the strength of the bentonite shale materials while lab testing was completed to address the strength of the clay shales and to determine if liquefaction had occurred (U.S. Army Corps of Engineers, July 1939).

Two of the nine board members, Merriman and Mead, did not sign the report: One for technical reasons and one for more philosophical reasons. Merriman, who disagrees technically, in summary appears to agree that the shale was the root cause but also thinks that the weathered shale should be completely removed from the area during repairs. The remaining Board is asked to address these concerns by USACE and closes by suggesting that based on tests and analysis of the pre-slide and post-slide conditions ample factor of safety is provided by the extremely flat slopes adopted for the more massive cross section.

It is later reported, that a couple of the Board members (Drs. Casagrande and Gilboy) were convinced that liquefaction had occurred even though the results from the lab testing to estimate the critical void ratio of the hydraulic fill indicated that the materials would not have liquefied (Middlebrooks, 1942). As a result, a statement about the potential impact was mentioned in the Board's conclusion. Dr. Casagrande's background with lab testing for liquefaction and their observations at the site of the enormous translational movement, flattening of the slide mass and sand boils between the floating blocks of embankment still ejecting sand days after the failure likely fuel their desire to better assess this mechanism for some time to come. More on this is contained below in Part II.

F. Embankment Re-Design and Repair

The rate at which the embankment redesign work occurred was remarkable and paralleled the engineering geology work presented above. As previously mentioned within a day of the failure they start work to build a dike and roads to allow for access to the slide area. The occurrence of the failure in the early fall allowed for the exploration as well as some construction to be completed that fall and early winter. Explorations and testing continued throughout the stark winter months and the bulk of the re-design work was carried out through the winter when construction was typically halted due to the severity of the conditions any way.

The back analysis of the slide focused on the strength of the bentonite materials as sensitivity studies were performed to determine the strengths to apply in the redesign. The redesigned cross section was simple in concept and even more massive in appearance and more symmetrical when compared to the original as shown on Figure 7.

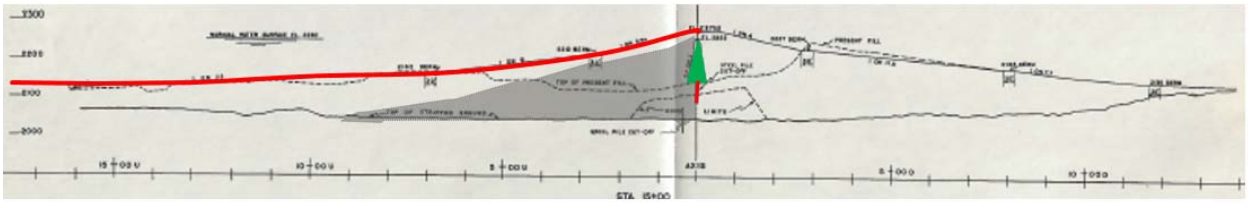

Figure 7. Re-designed slope in red over the original in gray. Green highlights the rolled till core around and above the sheet piles driven into the remnant core materials.

The Board of Consultants agreed that the shell of the reconstructed section could be built by either hydraulic fill or rolled-fill methods. The core materials in this area were to be of rolled fill obtained from the glacial till on the left abutment. The Board agreed with the plan for driving a single row of steel sheet piling (Figure 8) through the shell materials that moved in over the original core materials and into the underlying intact core, in effect, tying the three different cores together.

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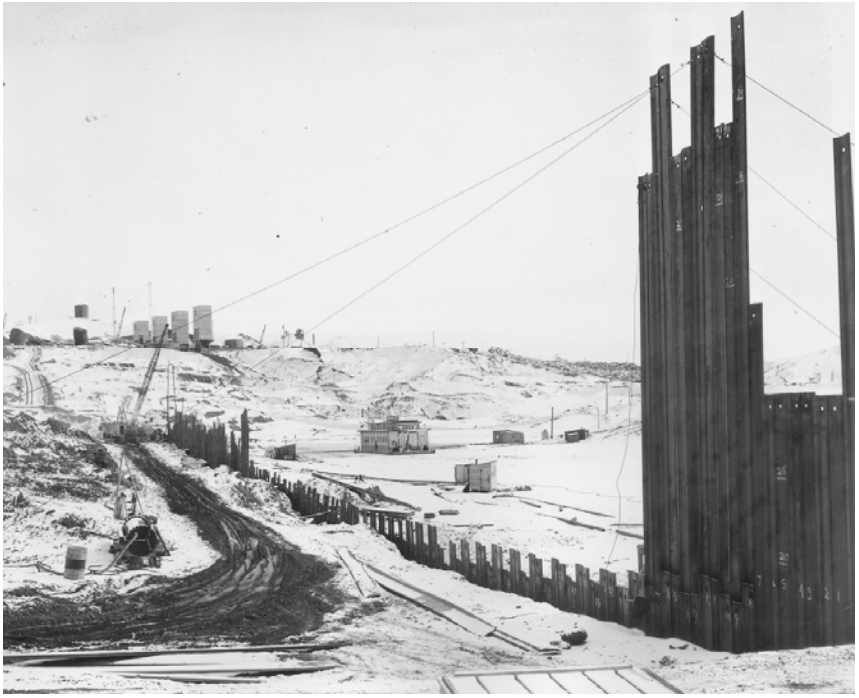

Figure 8. Winter installation of sheet piles into core

The portion of the dam unaffected by the slide would be strengthened by the addition of more fill in the upstream berm and the material in the added upstream fill be the same as or coarser than the original shell material. The board recommended these materials be compacted by dozer if it doesn't interfere with fill placement. The embankment fill above elevation 2,250 ft. was to be placed by rolled-fill methods (likely to improve short term stability over hydraulic fill) and that the placement of this fill be deferred until the recommended berms were completed. These precautions were apparently provided from the desire to "do no harm" by ensuring the temporary stability was increased over the existing condition with each step taken to finish the dam. Had they continued raising the unaffected portion of the dam with hydraulic fill before adding to the berm they would have approached the pre-slide conditions that existed at the location of the failure and may have triggered another slide. These precautions were likely born out of concerns related to liquefaction of the hydraulic fill as the shale was buried very deep across the remainder of the dam. Reconstruction of the dam was started immediately and the cross-section of the dam was redesigned for greater strength.

Material that had been hydraulically placed on the downstream slope was used for the outer shells. Glacial till from the left abutment was used for the core. The dam was raised to its design height elevation of 2275.5 mean sea level on October 11, 1940. By 1946 settlement had decreased sufficiently to allow final topping of the dam and crest road construction to a revised design elevation of 2280.5 mean seal level in 1948. Hydropower was added in 1943 and expanded with a second power plant in 1961.

G. Internal Erosion at Downstream Toe of Embankment Dam on First Filling

Another design change attributed to post construction operations at Fort Peck is the need to control seepage at dams with soil foundations. As noted previously and shown on the geologic and design cross section the designers were concerned with under seepage and installed a sheet pile cutoff wall to address this concern. A piezometer revealed high uplift pressures deep in the alluvium beneath the clay stratum at the downstream toe in 1942. The head is determined to be 45 feet above the ground surface and the depth to bottom of the clay stratum is about 90 feet at this piezometer. After a short time of high pressure readings, water began to flow up along the outside of the piezometer casing resulting in transportation and deposition of sand at the ground surface as shown in Figure 9.

This erosion had to be remediated immediately with an inverted filter consisting initially of sand, gravel, and cobbles. More piezometers were installed along the toe and pressures were predicted to rise a maximum of 70 feet above the ground surface at full reservoir. This performance and analysis of data led to the installation of relief wells which were observed to lower the pressures 40 feet to just above the ground surface at a discharge of about 4,000 gallons per minute. The Board was reconvened in the spring of 1943 at which time they agreed the temporary wells were adequate for full reservoir conditions but recommended that a system of more permanent replacement wells be planned as soon as materials were available after the war. The wells were later replaced and new wells were added in 2011 and 2012. The pressures have remained well controlled over the years. This was the first installation of relief wells at a USACE dam.

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Figure 9. Sand boils at toe with reservoir partially filled.

H. Conclusions – Part I

The slide mass was extremely complex as it moved over 1,200 feet upstream into the reservoir and parts of the mass moved left of its original station by as much as about 500 feet. Much of the mass was below the reservoir as the front of the slide was found to have been transferred into the diversion/outletworks channel nearly missing the tunnel intakes. The witnesses reported portions of the upstream shell nearest the pool began to slide into the sinking core pool and to a lesser degree the downstream shell was observed to do the same. Shale materials in the right abutment were found to have slid down and to the left likely into the sinking core pool area as well. On top of this, the geology appears to differ significantly from east to west across the area of the failure with the dam embankment being founded directly on clay shale and impervious clay to the east of about stations 19+00 to 20+00, and as much as 40 to 60 feet of semi-pervious sand alluvium on top of the shale west of these stations. Consequently, as previously noted, the controversies over the technical details of the failure is understandable.

This case history emphasized the importance of planned Corps' research into many issues related to shear strengths of natural soils, earth fill and the stability of embankment dams. It also emphasized the need for the following:

- Detailed characterization of dam foundations including detailed stratigraphy, fault and shear mapping, strength testing in labs and in the field, improved classifications of clay shale, etc.
- The evaluation of the "end of construction" stability of the embankment and foundation. Including the potential for pore pressures to develop in stiff clay shale.
- Designs of future embankment dams that addressed slope stability in many various ways:
    - Flat slopes
    - Compacted fill in core and shells
    - Drains at foundation/embankment contact
    - Foundation soil improvements by removal, blasting, even vertical sand drains
    - Staged construction to allow for pre-loading and drainage (consolidation)

Further safeguards included observations during construction of:

- Foundation conditions to verify design and geologic assumptions
- Foundation pore pressures
- Embankment pore pressures
- Inclinometers for shear movements
- Settlement of foundations as well as the dams using plates and monuments

Observations of these types resulted in a number of cases where construction operations were modified by the USACE to reduce the risk of a slope failure.

The subsequent internal erosion incident at the toe in 1942 also impacted on the USACE dam design practice related to under seepage:

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- It confirmed sheet pile cutoffs are imperfect and impacted design details and resulted in the adoption of multiple defenses for under seepage.
- Relief wells to protect against high uplift pressures and internal erosion at the toe became a relatively common practice.
- Other defensive design features such as blanket drains at the foundation contact were common downstream of all types of cutoffs and upstream impervious blankets were added to further reduce under seepage.

II. – A REVIEW OF TECHNOLOGY AND HUMAN FACTORS

A. Introduction

As noted in Part I, it was first revealed about three years after the completion of the project that two of the seven board members thought that liquefaction played a more substantial role in the slide than was captured in the conclusion of the report signed by all of them. Gilboy, in his discussion on the 1942 paper by Middlebrooks first articulated the view of the minority on the Board who concluded "that liquefaction was triggered by shear failure in the shale, and that the great magnitude of the failure was principally due to liquefaction." Casagrande remained silent on the subject until his Terzaghi lecture in 1965 where he reveals that the language of the Boards report was "a compromise wording to bridge the wide gap in the views of the consultants who signed the report." Casagrande goes on further to say that *"Gilboy and I shared the opinion that the liquefaction was centered principally in the fine sand zone of the shell next to the core, and that liquefaction may have spread into the underlying heavily loaded foundation sands."*

The following is a summary of the key finding from our review of the Technology and Human Factors associated with the design development and post failure evaluations:

- Key contributing factors related to the failure of the upstream slope of the dam were associated with the lack of understanding of
    - the shear strength of the shale foundation materials including what we now refer to as "**residual strength"**, and
    - the strength characterization of hydraulic fill materials and the mechanism of **"liquefaction".**
- Understanding the need to characterize the engineering geology information especially related to the pre-sheared clay shales and basic engineering properties of the foundation, was lacking
    - The classification of clay shales and the impact of slickensides, faults and other pre-sheared surfaces was not well understood.
    - The potential for high pore pressures to develop in the stiff fissured shales as a result of construction loads was also not anticipated.
    - The soil classification system they were using was of limited value as compared to the Unified Classification that would arrive nearly two decades later.
- The ability to characterize the shear strength of the foundation materials including the identified bad actor of the bentonite seams was minimal. While the issue of the strength of these materials and potential for failure had been identified, a full understanding of the strength properties, and the ability to characterize the strength for design and for post failure investigations was limited.
- While the concept of liquefaction (critical density/void ratio, Casagrande, 1936) was beginning to be understood from a number of case histories such as the failure of Calaveras Dam in CA (1918), the ability to properly characterize and assess the actual strength of the hydraulic fill materials either during design or as part of the post failure assessment was limited at best.

Consequently, the controversies over the assessment of the failure is understandable.

For example, with regard to liquefaction, it wasn't until 1958, that Roscoe reported on a series of S tests on cohesionless materials, performed with a "simple shear" device developed by him. Using this apparatus, he was able to reach the critical void ratio starting either from a loose or dense state. Tests on steel balls and on glass beads gave a better agreement on the final void ratios than those on sand, attributed to a certain amount of particle breakdown suffered by the sand during the tests. He confirmed that the critical void ratio decreased with increasing confining pressure. Hence, Roscoe was the first one to prove the existence of the critical void ratio as was hypothesized by Casagrande in 1936. Questions relative to the existence of a "flow structure" remained unanswered.

It took more than 20 to 30 years following the failure for the understanding of "residual" strength and "liquefaction" to begin to advance to a point where their impact on the development of the failure of the dam could be better understood. It is also appropriate to note that only within the last 10 years or so, have the analytical tools emerged that could more fully characterize the conditions leading to and triggering of the failure, and the contribution of both the residual strength and liquefaction mechanisms to the failure that developed.

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From a risk analysis standpoint, this failure points directly at the need for failure mode descriptions and event trees to include consideration of multiple contributing mechanisms to properly assess the system response.

B. Understanding of the Failure Mechanisms

While there were disagreements regarding the significance, the failure of Ft Peck Dam on September 22, 1938 appeared to involve two primary mechanisms: the residual strength of clay-shale in the foundation bedrock, and the liquefaction of hydraulically placed sands adjacent to the clay core of the dam.

The following subsections provide a summary of the understanding of these mechanisms at the time of design and when the failure occurred, as well as advances that occurred following the failure event. An understanding of the state of technology at that time is critical to understanding the struggles that the designers, and Board members experienced in reaching a consensus position on the cause of the failure and the appropriate modifications to the design needed to complete the dam. The authors wish to acknowledge that much of the information on early development of technology related to these failure mechanisms was obtained from Castro (1969), and LaGatta (1970).

C. Residual Strength of Clay Shale

At the time of the Fort Peck dam failure, the terminology and understanding associated with shear strength characterization of clays, and in particular, a clay shale such as the Bear Paw Shale in the right abutment of the dam where the failure occurred was very different than exists today.

Interest in the long term performance of excavated slopes in clay soils and the occurrence of landslides dates before 1846. Alexander Collin documented his research on this subject in his book entitled Experimental Research on Landslides in Clay Strata. As part of his research, Collin studied the location and shape of the slip planes in about fifteen slides where he concluded that the slides were the result of inadequate shear strength of the soil. Much of his research was related to stiff fissured clay. The cut slopes in these soils were stable during construction but failed at a later time. He concluded that the time effect associated with the failure was a result of softening of the clay due to entry of water into the slope. To support his conclusions, Collin performed relatively primitive shear strength tests (likely undrained although Collin did not refer to the tests as undrained nor did he recognize the importance of drainage conditions during the tests) by failing a clay prism in double-shear by adding a weight to a central, unsupported length of the clay prism. Collin found that the strength was greatly influenced by the water content of the clay. Times to failure were thirty seconds or less and Collin subsequently stated:

> "Knowledge of the absolute instantaneous resistance is of no use in construction practice. A *permanent resistance* is therefore the only one which it is necessary to know."

The research to find the "permanent resistance" of a clay took over a hundred more years to get to a place where engineers could adequately characterize and design embankment and cut slopes with a relatively complete ability to perform representative laboratory tests and understand the mechanics of clay soils and soft rock behavior.

The amount of shear strength testing performed between 1846 and the 1920's was meager with minimal improvement in the understanding of the stress-deformation properties of soils. In the 1920's however, research activities in this area began to increase significantly with the development of the direct shear device in Europe by Krey in 1926. Casagrande developed his direct shear apparatus while teaching at Massachusetts Institute of Technology between 1930 and 1932 (Hirschfeld and Poulos, 1973). The use of the term "residual strength" evolved through a series of research studies and technical articles as outlined below. Note, these studies were published after Fort Peck was designed and the dam was well into construction. The latter ones were published after the slide occurred.

Tiedemann, 1937 – "pure sliding resistance"
Haefeli, 1938 – "remaining shear strength"
Hvorslev, 1936 and 1939 – "ultimate minimum shear strength"
Haefeli, 1951 – "residual strength"

Tiedemann research beginning in the early 1930's focused on the study of stress-deformation characteristics of clays using a rotation shear device. Hvorslev (1936, 1939) and Haefeli (1938) also used rotation shear machines to investigate the strength of clay. Both Tiedemann and Hvorslev were particularly interested in investigating the shear strength of clays at large displacements. Hvorslev's 1939 publication describes a reduction in shearing resistance after the peak strength had been passed. Specifically, his tests on remolded Little Belt clay (L_w = 126, P_w = 36) indicated a drop in resistance to about 40% of the peak strength after a displacement of 9 cm. LaGatta (1970) suggests that the Hvorslev tests were likely not a drained test.

Tiedemann (1937) published results of rotation shear tests on remolded and undisturbed clay showing that at very large displacements, the shear strength was a fraction of the peak strength. This strength approached a constant value and Tiedemann referred to this approximately constant reduced strength as "Reinen Gleitwiderstand" (pure sliding resistance). Deformation on the order of 10 to 25 cm were required before "pure sliding resistance" was reached. It is possible that Tiedemann's tests on remolded clays were "S" tests (drained) with the tests lasting up to four weeks in some cases.

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The year of the slide, Haefeli (1938) called the last point of the stress deformation curve from his work with rotation shear tests "Restscherspannung" (remaining shear strength). LaGatta (1970) suggests from examination of the stress-displacement curves of Haefeli that "Restscherspannung" does not refer to a shear strength which is independent of displacement. In a later paper, Haefeli (1951) used the term residual strength as a translation for Restscherspannung. LaGatta notes that the tests reported by Haefeli were however, creep tests and they were not carried to sufficient displacement to achieve a constant shearing resistance we have come to associate with residual strength.

Hvorslev (1936, and 1939) introduced the term "ultimate minimum shear strength" to describe the approximated constant strength measured at large deformations.

Renewed construction activity on the existing Panama Canal provided the impetus for a significant increase in interest and research of clay slopes, particularly the re-evaluation of slides on the slopes along the existing canal. Binger and Thompson, 1949 describe the results of direct shear tests performed on a sample of undisturbed Cucaracha "shale" where two mating surfaces of the test specimens were polished to simulate slickensides. The slope of a strength envelope obtained from a series of tests on this sample was 10 degrees. The slope of the strength envelope was used for analyzing cuts in the Cucaracha formation in order to develop a relationship between depth of excavation and required slopes. ***This study appears to be the first use of strength smaller than peak strength for stability analysis for slopes in slickensided clay shale.***

In work summarized by Skempton, (1964), re-analysis of a number of slides showed that shear strength at failure may be close to the drained residual strength. Estimates of residual strength in Skempton's work were obtained by repeated shearing of a specimen in a direct shear machine.

In summary, at the time of the design and construction of Fort Peck Dam, the practice of characterizing the drained residual strength of clay shale materials containing slickensides and using this strength appropriately in design analyses did not exist. It was not until 1949, that the use of a reduced shear strength for this condition is noted as a basis of design in the literature, and not until 1964 (approximately 25 years after the failure), that the impact of the clay shale materials in the right abutment of the dam on stability during construction could have been fully characterized during explorations and laboratory testing for use in design. The redesign was, however, based on low shear strengths derived from a back analysis of the failure that they thought represented the existing conditions after the failure including pre-sheared failure planes.

D. Hydraulic Fill Dams and Liquefaction

At the time of the upstream slope failure, the understanding of the phenomenon of liquefaction was in its infancy. While there was evidence of this phenomenon in a number of case histories including the failure of Calaveras Dam in California, the community of practice was only beginning to wrestle with the subject. Most importantly, while there had been some ground breaking work published by Casagrade (see below), the understanding of effective stress and the shear behavior of materials at different density states was very limited. Most importantly, the ability to properly characterize the potential to trigger liquefaction and the undrained shear strength of the hydraulic fill materials in dams did not exist. The following is a summary of the general timeline associated with the understanding of "liquefaction" prior to, during and following the design and construction of Fort Peck Dam:

O. Reynolds (1885) was the first to discover that shear deformation of sand is accompanied by volume changes, and, in particular that dense granular materials increase in volume when sheared. This property of granular materials Reynolds termed "dilatancy."

The concept of effective stress begins to emerge in the U.S. from work by Hazen (1920) as part of his evaluation of the 1918 failure of Calaveras Dam in California, and Terzaghi (1925).

First use of the term liquefies (liquefaction) was by Hazen (1920) when describing the failure of the hydraulic fill Calaveras Dam. The slide at Calaveras was seated in the body of the embankment focusing attention on the strength of materials in hydraulic fill dams. The Fort Peck case likely included this type of mechanism as well as a foundation that was discovered after the failure to be extremely weak due to the presence of weathered shale and bentonite seams.

Perhaps the first explanation of the phenomenon of liquefaction was published by Terzaghi in 1925 and later in 1956: "Liquefaction can occur only on the condition that the structure of a large portion of the sedimentary deposit is metastable…. If the soil is saturated, at the instant of collapse the weight of the solid particles is temporarily transferred from the points of contact with their neighbors onto the water. As a consequence, the hydrostatic pressure at any depth z increases from its normal value of (z x gamma w) by an amount u_w which is close to the submerged weight of the sediment located between the surface and depth z." Terzaghi used the term "Beweglichkeit," which he later translated as "mobility," to describe the condition of sands during liquefaction failures.

50 years after the work by Reynolds related to "dilatancy" of dense sand, the concept of "Critical Density" (critical void ratio) is first described by Casagrande (1936) as the transition between dilatancy and contractive shear behavior. Casagrande hypothesized that at the critical void ratio, a condition that can be reached from either a loose (contractive) or dense (dilative) state where a "cohesionless soil can undergo any amount of deformation or actual flow without volume change."

During 1937, Casagrande made efforts to determine the critical void ratio in connection with design of the Franklin Falls Dam and noted that 1) the direct shear (DS) test was not suitable due to limited deformation possible, and challenges

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associated with determining both initial and the changes to void ratio during the tests, and 2) despite the DS test limitations, it was demonstrated that the critical void ratio was not a constant for a given sand, but decreased with increasing normal stress.

During 1937 – 1938 Casagrance focused on the triaxial S test for estimations of the critical void ratio. An S test is one in which the specimen is consolidated under a hydrostatic confining pressure, and then is subjected to axial straining with drainage such that practically no pore pressures are developed (drained test). In 1938, he further elaborated on the decrease in critical void ratio with increasing confining pressure, and proposed a mechanism to explain the strength reduction during liquefaction of sand with a void ratio above the critical. Specifically, Casagrande concluded that during a flow condition, the effective stresses are reduced to the effective confining pressure for which the critical void ratio is equal to the insitu void ratio of the sand. The shear strength of the sand would thus be reduced to a value which is only a function of its void ratio. If a slope or embankment is high enough so that the shear stresses are much greater than the reduced shear strength of the sand, a flow failure would result. Hence for what may have been the first time, it was proposed that the occurrence of liquefaction is not necessarily related to a zero or almost zero effective stress condition, but rather to a reduction of the effective stresses which is large enough so that the shear strength would drop substantially below the existing shear stresses in the soil mass. Hence in conditions of high shear stresses such as under a high dam, catastrophic liquefaction may develop while the effective stresses in the flowing mass remain relatively large.

For his doctoral thesis during the period of 1937-1939, J.D. Watson performed a comprehensive series of triaxial S tests under Professor Casgrande's guidance, to estimate the critical void ratio for several sands including samples obtained from Fort Peck Dam. He found that even during tests on loose specimens in which no failure planes developed, the volumetric and axial deformation of the specimens were not uniform at the large strains needed to reach the critical void ratio, nor when the maximum compressive stress was reached at which the "lower critical void ratio" was determined. He performed two R triaxial tests on loose specimens, during which he found a strength reduction of about one-half with respect to the S strength, due to the development of pore pressures. The triaxial R(bar) test is one in which a specimen is first consolidated under a hydrostatic confining pressure, and then, without permitting any further change in water content, is subjected to an increase in axial loading until failure is reached (undrained test).

Watson's tests of the samples of sand from the hydraulically deposited shells of Fort Peck Dam indicated that the critical void ratio as determined from the laboratory tests was higher than the estimated void ratio of the sand in the dam. Hence, he drew the conclusion that liquefaction could not have occurred.

Casagrande was not convinced and therefore concluded that the method used for determining the critical void ratio was faulty and led to unsafe results and warned against the use of that method. He subsequently developed the concept of the "flow structure" during a liquefaction slide where the relative position of the grains is constantly changing in a manner which maintains a minimum resistance. He explained that the change from a normal structural arrangement of the grains to the flow structure would start almost accidentally in a nucleus and then spread through the mass by a chain reaction; and that such a reaction could explain the spontaneous character of liquefaction slides.

As previously noted, the results of subsequent research by Castro (1969) and reported by Casagrande (1975) identified that the strain controlled laboratory tests performed on the Fort Peck dam samples erroneously identified the critical void ratio of these materials and that only a load controlled test procedure, available only after more than 30 years had passed, would have correctly identified the liquefaction potential of the hydraulic fill. As part of recent personal communication with Dr. Castro, subsequent research has modified the conclusion regarding the need for load controlled testing.

From the standpoint of liquefaction potential failure mechanisms, the Fort Peck failure case history remains of keen interest and the subject of additional study and research. The table below provides a summary of the efforts by researchers to characterize the undrained residual (steady state) strength of the hydraulic fill in Fort Peck Dam.

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TABLE 1. Summary of Estimated Undrained Residual (S_r) (also described as Undrained Steady State Strength (S_{us})) of Ft. Peck Hydraulic Fill materials

Reference	Estimated $N_{1,60}$ Blowcount	Back Calculated S_r (S_{us}) (psf)	Ave. Estimated Fines Content (%)	Estimated Vertical Effective Stress*(psf)	Comments
Casagrande (1965)					Reports Westergaard friction angle of 4.3 degrees from 1938 analysis.
Seed (1987)	11	600	-		S_r stated as reasonably conservative value
Davis, Poulos, and Castro (1988)	5.3	700	30	9,310	Method considered kinematics of failure. However, Fort Peck was estimated based on Westergaard friction angle times the average vertical effective stress.
Seed and Harder (1990)	8	350			Some dynamic effects considered but not well explained.
Stark and Mesri (1992)	10	350 +- 100	-	10,815	
Baziar and Dobry (1995)	10	350 +- 100	4	10,815	
Olson and Stark (2002)		550		7,341	Used Davis et al procedure to estimate S_{us}
Kramer and Wang (2015)	11.7	670	54	7580	Used Davis et al procedure with focus on configuration with zero inertia factor (ZIF)
Webber (2015)		760		7,258	Used procedure similar to Davis et al but included a sequence of slides rather than a large single mass moving simultaneously.

* at Critical Failure Surface Location

E. Epilog

The failure of Fort Peck Dam provided a significant case history for the dam engineering practice in the United States. Many lessons were learned through 1) the construction of this structure, 2) the partial failure that occurred, 3) the post failure assessments, redesign and completion of construction, and 4) first filling of the reservoir. In fact, the case history continues to be a source of learning and evolution of technology, particularly in the area of liquefaction.

Our review of the technology related to the primary mechanisms of failure including ***residual strength*** of the right abutment Bear Paw Formation clay shale, and ***liquefaction*** of hydraulic fill materials used in construction of the dam has reinforced the reasons for controversy amongst the members of the Consulting Review Board that were charged with assessing the root cause of the slope failure. The need for additional research in these topic areas as a result of the slide incident in 1938 would become a driving influence for research and technology development in the US, and even international community of practice related to dams. There appears to be even more lessons ahead as deeper technical questions emerge from continuing research on both failure mechanisms.

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