INDEPENDENT TECHNICAL REVIEW
SANDSTONE DAM PROJECT
NEAR SAVERY, CARBON COUNTY, WYOMING

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SUMMARY

The attached report summarizes an independent geological and geotechnical review of the Sandstone Dam and Reservoir Project near Savery, Wyoming. Stone and Webster Engineering Corporation (SWEC) of Denver, Colorado performed geological and geotechnical investigations at the site on behalf of the Wyoming Water Development Commission (WWDC) in late summer 1993 and reported draft results on December 17, 1993. These investigations, including drilling of two 121-foot deep core holes in the left dam abutment and excavation of borrow test pits in the reservoir basin, are an extension of previous feasibility studies conducted by SWEC at the Sandstone site. A major concern is the possible presence of weak clay seams in the dam foundation which could affect technical/economic feasibility of the site.

An RCC dam requires a high shear strength, high bearing capacity foundation to resist sliding and bearing failure. RCC dams are ideally constructed in high strength rocks without major structural discontinuities or zones of weakness. SWEC (1986) stated: "The foundation (at the Sandstone site) is too weak for a concrete arch dam and is considered marginal for any concrete gravity structure including roller compacted concrete [emphasis added]. In addition, SWEC concluded "the irregular topography is poor for a concrete gravity structure." WWC (1992) also recognized marginal geological conditions at the Sandstone site. An RCC dam is a marginal choice for the Sandstone site and will require additional engineering studies beyond those normally required for a more suitable site. Even so, significant uncertainty will exist, requiring conservative design assumptions to be made.

Emphasis in previous reservoir landslide studies has been on catastrophic failure, which could threaten the safety of the dam, and on sediment volumes contributed to the reservoir. However, the engineering implications of landsliding are far broader than these two issues.
alone. Current interpretations of landsliding in the project area are over-simplified. Landslide movement occurs in a variety of stratigraphic units, and failure mechanisms are complex. Bedrock is locally weak and susceptible to landsliding, especially in the presence of ground water.

Large landslides in the project area, previously interpreted to be 10,000± years old and having little engineering significance, are currently experiencing deep-seated and near surface movement. The Big Sandstone landslide is much younger than 10,000 years and may have been triggered only a few hundred years ago. Many other landslides show evidence of continuing movement, and a number of incipient slope failures in the project area indicate landsliding is a continuing geological process.

Impoundment of Sandstone reservoir will inundate many of the existing springs and seeps along the east side of the Savery Creek valley. Given the interpretation that confined and semi-confined units within the Mesaverde Group are recharged in higher terrain to the east and ground water moves downdip to the southwest, pore pressures must increase in existing landslides and adjacent rock masses on the east side of the reservoir. Existing landslides will be reactivated and new bedrock and surficial landslides will occur as the result of pore pressure increases. Uncertainty exists in the potential for new bedrock landslides in heretofore stable bedrock around the reservoir rim. The possibility exists that new landslide movements may be rapid as peak shear strength is reduced to residual strength along incipient failure surfaces. The volume of debris contributed to the reservoir may be a minimum value based on assumption of limited movement of existing landslide masses.

The number, variability of failure mechanisms, and stratigraphic range of landslides in the project area indicate that strength of the bedrock in the dam foundation is a legitimate concern, especially for an RCC dam. Assumptions regarding: (1) the presence of weak, incipient failure surfaces in the dam foundation and (2) shear strengths should be conservative. Any reduction in conservatism must be based on well-documented justification.

Low strength clay seams are continuous over several hundred feet, dip in a downstream direction from 0.4° to 4.5°, and daylight on the downstream face of the bluff which forms the left abutment of the dam site. Given the continuity and geometry of clay seams in the left abutment, shear strength of the clay seams is a critical issue. Laboratory direct
The shear strength of intact clay seams produced uncertain results. The peak shear strength of intact clay seams was not defined by the laboratory testing program. The interpretation of the residual shear strength envelope is reasonable given the available data. While differences of opinion may exist with regard to causes and interpretations of these laboratory results, the existence of low strength (friction angle = 10°), continuous clay seams must be considered in project design.

SWEC (1993) indicates that low strengths of the intact clay seams are inconsistent with the "published data", relating plasticity index to peak shear strength for normally consolidated clays. The clay seams in question, however, are overconsolidated and exhibit a 10° residual friction angle. The comparison made by SWEC to published data is not well drawn. A 10° residual friction angle is consistent with clay fraction and activity ratio.

The shear strength of remolded, composite samples is higher than naturally-occurring, intact clay seams due to the averaging of index properties and the destruction of geologic fabric in intact samples. Direct shear test data are consistent with this expectation. The higher friction angles of 20°-24°, obtained from composite samples, are not representative of the weakest intact clay seams in the dam foundation. Prudent engineering analyses requires use of a 10° friction angle in stability analyses.

The general criteria and geometry of the SWEC stability analyses are reasonable. Use of a 10° friction angle is appropriately conservative given the uncertainty in laboratory test results. The resulting factor of safety against sliding of 1.6 in the left abutment is less than the 2.0 recommended by the U.S. Army Corps of Engineers, 3.0 recommended by the Federal Energy Regulatory Commission, and 4.0 recommended by the U.S. Bureau of Reclamation for normal loading conditions. Seismic loads are not included in current stability analyses. Use of friction angles higher than the reported 10° in stability analyses will require additional study, documentation, and justification. Adequate documentation and justification has not been presented to dismiss a 10° friction angle in favor of exclusive use of higher values.

Irregular topographic conditions in the foundation of the proposed RCC dam could result in stress concentrations and potential for cracking in the RCC section. The recommendation that topographic irregularities be handled by vertical construction joints is expedient and probably not an ideal engineering solution.
SWEC (1986) conservatively assumes that the 14-mile long fault downstream of the dam is seismogenic (active). A Maximum Credible Earthquake (MCE) and resultant ground motions related to this fault are used in the preliminary seismic stability analysis. This analysis has not been updated for the proposed RCC dam using strength data generated from the 1993 site explorations. If the assumption is made that the fault is active and no information is available on segmentation, the entire 14-mile fault length should be used to estimate the MCE. A 14-mile rupture length is well within the range of documented surface ruptures in Wyoming and the intermountain west.

The right side of a proposed alternative dam alignment will abut a narrow bedrock spur rather than a declivity in the valley wall at the original site. Protruding bedrock spurs may not provide ideal abutment conditions because the rock mass tends to disintegrate due to stress relief. The net effect is that highly jointed and fractured bedrock must be over-excavated, negating the apparent topographic benefit of the site and cost savings. No studies have been performed to evaluate this condition.

Additional investigations will be required before reasonable conclusions can be drawn regarding the project as presently envisioned. Shifting the dam axis to the north fundamentally creates a new dam site with little reliable site-specific foundation data. Although existing subsurface explorations provide useful guidance at the new site, little if any reliable subsurface information is available. Existing USBR foundation data provide insufficient technical bases for feasibility designs and cost estimates.

Cost estimates in the WWC (1992) and SWEC (1993) reports are confusing. The WWC (1992) cost estimate, including engineering, permitting, and contingencies (15%), totaled $30,000,000. The SWEC (1993) cost estimate for Alignment A totaled $25,295,000 but does not include permitting, engineering, and contingencies. Costs were reevaluated based on the WWC (1992) approach. Because uncertainty exists in the amount of excavation required in the dam foundation and the resultant volume of RCC required, the contingency factor for these items was increased from 15% to 25%. The resulting total cost is $33,306,000 for Alignment A. Using the same approach for the alternative site (Alignment B), a cost of $31,121,000 was estimated. Cost estimates for the alternative site, however, are uncertain due to the relative absence of site-specific information.
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1. INTRODUCTION

This report summarizes an independent geological and geotechnical re­view of the Sandstone Dam and Reservoir Project near Savery, Wyoming. Stone and Webster Engineering Corporation (SWEC) of Denver, Colorado performed additional geological and geotechnical investigations at the site on behalf of the Wyoming Water Development Commission (WWDC) in late summer 1993 and reported draft results on December 17, 1993. These investigations, including drilling of two 121-foot deep core holes in the left dam abutment and excavation of borrow test pits in the reservoir basin, are an extension of previous feasibility studies conducted by SWEC (1986, 1987) at the Sandstone site. A major concern identified by SWEC and Western Water Consultants, Inc. (1992) is the possible presence of weak clay seams in the dam foundation which could affect technical/economic feasibility of the site. Independent technical review of studies performed to-date were requested by local landowners due to the controversial nature of the project.

1.1 OBJECTIVES AND SCOPE

The objectives of our independent geological and geotechnical review were: (1) to assess geological and geotechnical conditions at the Sandstone dam site and in the reservoir basin; and (2) to assess the impact of those conditions on the technical feasibility of dam construction as currently proposed. The scope of our geological and geotechnical studies included the following tasks:
1. Review of previous geological and geotechnical studies conducted at the Sandstone dam and reservoir site and any relevant literature/reports which may have been published since completion of earlier studies.

2. Geologic reconnaissance of the Sandstone site by the undersigned principal of our firm to allow development of independent professional opinions regarding geological and geotechnical conditions affecting dam design and construction.

3. Monitoring of core drilling and test pit excavation conducted in late August and early September 1993. Monitoring of the initial core hole and test pit excavations provided assurance that factual data was being collected in a manner consistent with accepted standards of practice. The SWEC supplemental exploration program was conducted in a thorough and professional manner which eliminated the need for full-time monitoring.

4. Analysis of the data reviewed and preparation of a report, signed by a Professional Engineer and Professional Geologist as defined by Wyoming state law, describing our professional opinions regarding geological and geotechnical conditions at the Sandstone dam and reservoir site and the potential impact on project design and construction.

As professional engineers and geologists, we offered no guarantees that our independent review would favor any party to the controversy or would disclose any "fatal flaws" or other serious deficiencies that might affect the ultimate feasibility of the project.

Time and available resources limited the depth of independent technical review summarized in this report. Accordingly, we identified key issues in our areas of expertise, geological and geotechnical engineering, that would most likely affect dam design and construction. We did not critically review other technical aspects of the project outside our expertise, including hydrology, economics, alternative development schemes, etc. This report is not represented and should not be construed as a comprehensive technical review of the proposed Sandstone Project. We are confident, however, that our technical review and comments on geological and geotechnical conditions are commensurate with the current level of investigation, reasonable, and technically defensible.
The practice of geological and geotechnical engineering is greatly affected by the uncertain nature of soil and rock materials. Our experience indicates that very little difference exists in analytical techniques employed by competent engineers and geologists. We believe that work performed by SWEC (1986, 1987, 1993) is consistent with the standard of engineering practice; and, therefore, we have expended little time in checking or reproducing analytical results or in questioning basic engineering data and procedures. Instead, we have focused our review on uncertainty and resulting assumptions in engineering analyses, which are far more likely, in our experience, to affect the outcome of analytical procedures. Because uncertainty and the assumptions that follow from uncertainty are subjective, reasonable and competent engineers/geologists can and often do disagree. Our individual and corporate experience is in characterization of uncertainty required in dealing with geologic materials in design and construction. These areas are where we focused our review. Although we may disagree with other workers, we caution that these disagreements are not a reflection on the professional competence or integrity of any individual or organization associated with the Sandstone Project.

1.2 ACKNOWLEDGMENTS

Our independent technical review benefited from the assistance of many people. Mr. John Boyer provided background information on the project and provided logistical support in the field during drilling operations and reconnaissance studies. Ms. Kate Fox, Esq., Burgess, Davis & Cannon, assisted with legal and procedural matters. Mr. Bob Reynolds, P.E., and Mr. David Jurich, P.E. of Stone & Webster Engineering Corporation provided access to existing data and allowed examination of core in SWEC offices; their assistance throughout our review is greatly appreciated. Mr. Michael Purcell and Mr. Michael Carnevale, Wyoming Water Development Commission, provided access to existing aerial photography of the dam/reservoir site and allowed us to examine core from previous drilling programs. Mr. C.S. Venable Barclay, U.S. Geological Survey (retired) discussed his previous scientific studies in the project area, and provided scientific observations and geologic interpretations to all parties. We discussed the geology of the dam site with a number of interested parties including
Mr. Sam Andrews, retired geologist, Marathon Oil Company; Mr. Mitch Reynolds, U.S. Geological Survey; Dr. Bob Schuster, U.S. Geological Survey; and Mr. Jim Case, Wyoming Geological Survey.

The independent technical review was conducted by Michael W. West, Ph.D., P.E., P.G., with assistance from Mr. James D. Gill and Ms. Carma A. San Juan, staff engineering geologists. This report was prepared by Dr. West with assistance from Mr. Gill and Ms. San Juan.
2. LOCATION AND PHYSIOGRAPHY

We refer the reader to previous reports on the Sandstone Project (SWEC, 1986, 1987, 1993; Naftz and Barclay, 1991; WWC, 1992) for detailed descriptions of the project including location, physiography, and project features. We provide a brief synopsis of these subjects to orient the reader.

2.1 LOCATION

The proposed Sandstone dam site is located on Savery Creek in the E-1/2, Section 2, Township 13 North, Range 89 West about eight miles north-northeast of Savery, Carbon County, Wyoming (Figure 1). Savery Creek flows to the south to confluence with the Little Snake River near the town of Savery. The proposed reservoir extends about 3.25 miles up the Savery Creek Valley and enters the mouths of the Little Sandstone and Big Sandstone valleys to the east. State Highway 70 east from Baggs and a local county road north from Savery provide access to the site.

2.1 PHYSIOGRAPHY

Sandstone Dam and Reservoir is proposed for construction in an incised, somewhat asymmetrical valley. Elevations range from about 7800 feet on the divides west of the reservoir to about 6960 feet at the dam site, total relief of about 840 feet. The west side of the Savery Valley is relatively steep and precipitous compared to the east side. Slopes on the west side are underlain by interbedded sandstones, siltstones, claystones, and shales dipping into the slope. These same units on the east side of the valley dip out of the slope, which has accentuated mass wasting (erosion, landsliding). Ancestral Savery Creek preferentially migrated to the west as it cut its valley, eroding the steep west valley wall and depositing terrace gravels at former stream levels on the east valley slope. The east-to-west migration of Savery Creek is
controlled largely by the westward dip of bedrock units and also ac-
counts for the asymmetry of the valley in cross-section.

The proposed Sandstone dam site is located in a narrow part of the
Savery Creek Valley created by an 20-foot high, alluvium-covered
sandstone bluff on the east side of the flood plain and the steep west
valley wall. The east valley slope above the sandstone bluff rises gently
across a series of older alluvial terraces. The west valley slope rises
steeply and somewhat irregularly to the ridge crest west of the dam
site. White to buff-colored sandstones, forming subvertical cliffs, out-
crop on the west valley slope above the dam site.

The Savery Creek flood plain at the dam site is about 170 feet wide
compared to 500-1000 feet upstream and 600-1700 feet downstream of
the site. Alluvial fans at the mouths of steep ravines on the west valley
wall partially cover and interfinger with flood plain alluvium in the
lower west abutment. These alluvial fans and the resistant sandstone
bluff on the lower east abutment at the site account for the constriction
in the flood plain at the dam site.

Little Sandstone Creek, one of two major tributaries of Savery Creek,
enters the main valley just upstream of the dam site. Big Sandstone
Creek, the second major tributary, enters the valley one mile upstream
of the dam site. Like Savery Creek, both valleys are deeply incised
(800-1000 feet) through similar sedimentary rocks, but are symmetrical
in cross-section probably due to orientation of the valleys subparallel to
bedrock dip.
3. GEOLOGIC SETTING

Barclay (1976, 1991), SWEC (1986, 1987, 1993), and WWC (1992) describe the geologic setting, stratigraphy, and structure of the Sandstone Project area, and we refer the reader to these maps and reports for detailed descriptions of regional and site geology. In addition, Barclay (Personal commun., 1993, 1994) examined selected core from the 1986 and 1993 SWEC drilling programs and summarized his interpretations on Figure 2 (this report) and in the Appendix. These interpretations were provided to SWEC and WWDC. We summarize key aspects of site geology below as an introduction for the reader.

3.1 STRATIGRAPHY

The project area is underlain by a thick sequence of predominantly late Cretaceous age and older rocks. Younger Tertiary age rocks unconformably overlie Cretaceous rocks on the upland surfaces in the project area. Unconsolidated surficial deposits, mainly of alluvial and colluvial origin, mantle the bedrock on lower valley slopes and the valley floors of major drainages. Large late Pleistocene to historic landslides in the reservoir basin involve bedrock and overlying surficial deposits, especially along Savery, Little Sandstone, and Big Sandstone creeks.

3.1.1 Bedrock

The principal bedrock units in the Sandstone project area are the Cretaceous Steele Shale, the Mesaverde Group consisting of the Haystack Mountains Formation, the Allen Ridge Formation, and rocks possibly equivalent to the Almond Formation; and the Oligocene-Miocene Browns Park Formation.

The Steele Shale in the project area consists of marine montmorillonitic clay shales with thinly interbedded shales, siltstones, and sandstones. The contacts between the underlying Niobrara Formation and the overlying Mesaverde Group are gradational. Thin, persistent bentonite
beds, probably resulting from devitrification of airfall volcanic ash, are also present within the formation. The Steele Shale outcrops in the upper reaches of the reservoir basin and in the headwaters of Little Sandstone and Big Sandstone drainages.

The Mesaverde Group consists of a 2825 to 3340-foot-thick sequence of marine transgressional, regressional, and coastal margin rocks deposited in the Cretaceous Western Interior Sea as it retreated to the east (Naftz and Barclay, 1991). These rocks include complexly interbedded sandstones, shales, siltstones, claystones, and coals. In the project area, the Mesaverde Group includes, in ascending stratigraphic order, the Haystack Mountains Formation, the Allen Ridge Formation and strata possible equivalent to the Pine Ridge Sandstone and the Almond Formation (Naftz and Barclay, 1991). The dam site is located in the middle part of the Haystack Mountains Formation.

Naftz and Barclay (1991) describe the stratigraphy of the Haystack Mountains Formation:

"The Haystack Mountains Formation was deposited in marine and marginal-marine environments and . . . is composed principally of two alternating lithofacies -- a sandstone lithofacies, which forms prominent persistent cliffs in the canyon of Savery Creek, and a shale lithofacies which forms the intervening slopes . . . The sandstone lithofacies primarily is composed of thick sequences of light-gray, grayish-orange to light-yellowish-gray weathering . . . very fine to medium-grained, crossbedded sandstone. This sandstone is composed mostly of detrital quartz, chert, and minor feldspar. The sandstone is cemented by quartz overgrowths and clay minerals, and less commonly by calcite . . ."

"The shale lithofacies consists predominantly of gray, generally noncalcareous clay-shale, mud-shale, and clayey silt-shale, which commonly contain thin layers of sparse, light-gray fossiliferous limestone and limy mudstone concretions and much less commonly, thin sandstone beds. Shale lithofacies generally are in sharp contact with underlying sandstone lithofacies, but at their top, generally grade into overlying sandstone lithofacies within 2- to 20-foot-thick sequences of thinly interbedded and interlaminated shale, siltstone, and very fine-grained sandstone."

"In Savery Creek canyon and adjacent areas, the Haystack Mountains Formation is 950 to 990 feet thick (Barclay, 1976; Gill, 1974) and is divided into three parts (Barclay, 1976), each of which contains both sandstone and shale lithofacies. The lower part consists, in ascending order, of (1) the Deep Creek Sandstone Member of Hale (1961), 120 to 140 feet thick, which is in gradational contact with the underlying Steele Shale . . . (2) an informal shale member, about 315 feet thick . . . and (3) the Hatfield Sandstone Member (Gill and others, 1970), 130 to 150 feet thick. The middle part of the Haystack Mountains Formation consists of a lower shale unit, about 114 feet thick, and an
upper sandstone unit, about 172 feet thick, which is informally referred to as the sandstone of Savery Creek. The upper part of the Haystack Mountains Formation consists of (1) a shale unit, about 70 feet thick, and (2) a sandstone unit, about 30 feet thick, which . . . is conformably overlain by coal and carbonaceous shale at the base of the Allen Ridge Formation."

"The Haystack Mountains Formation . . . was deposited during an overall eastwardly retreat of the Cretaceous Western Interior Sea. The Deep Creek Sandstone and each successive sandstone unit is interpreted to be a progradational marine shoreline deposit. Each intervening shale unit is an offshore deposit marking at least a local pause in shoreline progradation, a reversal in movement of the shoreline, and a marine transgression."

Naftz and Barclay (1991) characterized environments of deposition for individual lithofacies within the Haystack Mountains Formation based on depositional structures, grain size distribution, and (trace) fossils. Lithofacies and depositional environments are illustrated on the stratigraphic column, Figure 2. Principal depositional environments represented in the Haystack Mountains Formation include in increasing energy of deposition: (1) offshore; (2) transition; (3) lower shoreface; and (4) upper shoreface. Offshore units are predominantly fine-grained shales, claystones and siltstones deposited in a low energy environment. Lower shoreface units are typically higher energy deposits and consist of fine-grained sandstones with thin interbedded shales, claystones, and siltstones. Upper shoreface deposits are high energy deposits and tend to be massive fine- to medium-grained sandstones. The transition environment is intermediate between offshore and lower shoreface.

The engineering significance of these depositional environments lies in the probable presence of weak, laterally continuous clay-shales and/or claystone seams. The offshore, transition, lower shoreface units may contain weak beds, seams, or partings which could control the stability of a proposed dam foundation. The presence of similar weak beds, seams, or partings in the upper shoreface environment is less likely due to the higher energy of deposition and the resulting larger grain sizes and thicker beds.

Naftz and Barclay (1991) also describe the stratigraphy of the Allen Ridge Formation:

"The Allen Ridge Formation conformably overlies the Haystack Mountains Formation and . . . toward Rawlins . . . consists of a thick (1000-1200 feet) nonmarine, lower informal member and a much thinner (140-220 feet) marginal marine, upper informal member (Barclay, 1980a, 1980b). . . . the lower, nonmarine member crops out along the canyons of Savery Creek, Coal Gulch,
and Loco Creek, and is composed principally of four lithologies — sandstone, siltstone, mudrock, and coal. These rocks generally comprise graded, fluviatile channel-fill and overbank sequences of trough-crossbedded sandstone, ripple-laminated sandstone and siltstone, siltstone, and mudstone, which in some places are capped by thin beds of coal or carbonaceous shale. In addition, near the base of the Allen Ridge Formation, these rocks are found in reverse-graded, crevasse-splay sequences of mudstone, interbedded siltstone and sandstone, and crossbedded sandstone, also locally capped by carbonaceous shale or coal. The sandstone weathers yellowish-orange to yellowish-gray and is calcareous to noncalcareous, very fine to medium grained, and in lenticular units as thick as 40 feet that form resistant ledges and scarps. The siltstone is yellowish orange, clayey, massive to laminated, and flatbedded. Some outcropping siltstone is hard, calcareous, ferrugenous, concretionary, and has a nodular surface. The mudrocks include claystone, mudstone, and shale, and range in color and organic content from olive-gray rocks with no visible organic matter, to grayish-brown rocks with carbonized plant fragments, to blackish-brown carbonaceous rocks streaked with coal . . ."

"Most of the lower member of the Allen Ridge Formation probably formed in an alluvial-plain setting behind an eastwardly prograding shoreline. In the area of the proposed reservoir, coal beds about 1 to 4 feet thick, which directly overlie the highest shoreline sandstone of the Haystack Mountains Formation, and an overlying sequence of weakly bioturbated fine-grained beds are probably paralic, delta-plain deposits . . ." 

"North and west [of the project area], that part of the Mesaverde Group above the lower nonmarine member of the Allen Ridge Formation is composed of, in ascending order, (1) the upper part of the Allen Ridge, a marginal marine unit; (2) the Pine Ridge Sandstone, a nonmarine, fluviatil deposit, 40 to 80 feet thick; and (3) the Almond Formation, a marginal marine and marine unit (Barclay, 1980a, 1980b, 1984; Bryant, 1984). [In the project area,] the Pine Ridge Sandstone was not recognized (Barclay and Shoaff, 1978; Barclay, 1979), and the upper member of the Allen Ridge could not be differentiated from similar deposits in the Almond Formation. As a consequence, the upper member of the Allen Ridge, the Pine Ridge or its possible equivalents, and the Almond Formation are mapped as a single unit, herein referred to as the upper part of the Mesaverde Group."

In southern Wyoming, the Mesaverde Group is conformably overlain by the Lewis Shale of late Cretaceous age. This contact is not exposed in the project area, and may be covered by Browns Park Formation in the NW-1/4, Sec. 5, T.14N, R.89W (Naftz and Barclay, 1991).

The Browns Park Formation of Oligocene-Miocene age overlies the Mesaverde Group in the project area on an angular unconformity. The unit consists of a basal cobble-boulder conglomerate lithofacies, which unconformably overlies Cretaceous rocks of the Mesaverde Group, and an upper sandstone lithofacies. The upper lithofacies consists of
tuffaceous, clayey, very fine to coarse-grained sandstones of predominantly fluvial and eolian origin. Total thickness of the Browns Park Formation in the project area is 200 to 300 feet and includes mainly the basal conglomerate unit capping upland surfaces and drainage divides.

3.1.2. Surficial Deposits

Bedrock across the project area is mantled by variable thicknesses of unconsolidated surficial deposits ranging from probable Pleistocene to historic in age (<1.9 million years before present). These deposits include alluvium along the floors of major drainages and in strath terraces along the east side of the main Savery Creek valley, alluvial fan deposits, landslides, and residual soils/colluvium locally blanketing bedrock on the valley slopes.

3.2 STRUCTURE

Cretaceous sedimentary bedrock in the project area generally strikes northwest to north-northwest and dips to the southwest to west-southwest at 3°-9° away from the Precambrian-cored Sierra Madre uplift to the east. We infer that Cretaceous sedimentary units exposed at the dam site and in the reservoir area lap on to the Sierra Madre uplift and are present in subcrop beneath the Browns Park Formation.

Uplift of the Sierra Madre after the deposition of Cretaceous sedimentary rocks resulted in tilting, warping and local faulting of the originally horizontal beds. Generally low dip angles at the dam site and in the reservoir basin render true dip measurements difficult. At the dam site, we calculated that bedrock of the middle Haystack Mountains Formation strikes N63°W and dips 3.75° to the southwest. Our calculations (Figure 3) are generally consistent with SWEC's dip of 0.4° to 4.5° to the southwest. Barclay (Personal commun., 1993) indicates that flattening of dip may be present upstream of the dam site resulting from local warping of the bedrock. We noted evidence for some variation in dip (flattening) in our structural analysis at the dam site, but could not confirm it by direct observation in the field. In the reservoir area, bedrock generally strikes north-northwest and dips to the southwest at angles ranging from 3° to 4° (Barclay, 1976; Naftz and Barclay, 1991).
Barclay maps several down-to-the-west normal faults, striking north-west to north in the vicinity of the dam and reservoir (Barclay, 1976; Naftz and Barclay, 1991). In general, these faults are believed to be related to the uplift of the Sierra Madre to the east. Two of the faults have potential significance to the project. A down-to-the-west normal fault, approximately 14-miles long, passes about 1700 feet downstream of the Sandstone dam site. Exploratory trenching in Quaternary terrace deposits (SWEC, 1986) across the trace of the fault downstream of the dam site yielded inconclusive data regarding the age of latest fault movement. Nevertheless, this fault was selected as a local earthquake source in the SWEC (1986) seismic stability analysis.

A second, and possibly related down-to-the-west normal fault strikes northwesterly across the mouth of Big Sandstone Creek. This fault passes near a recently activated landslide complex on the north valley slope of the Big Sandstone Valley; but its role, if any, in controlling landsliding is unknown at this time.
4. PROPOSED CONSTRUCTION

As presently envisioned, the proposed Sandstone Dam will be a 130-foot high, membrane, roller-compacted concrete (RCC) dam, impounding a reservoir of 23,000 acre-feet (surface area of 463 acres) at normal water surface elevation of 6883 feet. The spillway will be an overflow, conventional concrete chute with ogee crest located at the maximum dam section. An outlet works will be constructed through the RCC section and will include a multi-level, gated intake tower located adjacent to the upstream dam face. The reservoir at normal water surface elevation 6883 feet will have a surface area of about 463 acres.

5. PREVIOUS WORK

Sandstone Dam has been studied since the 1940s as part of the U.S. Bureau of Reclamation's Savery-Pothook Project. USBR (1977) conducted site investigations at the Sandstone site including drilling in 1974-76, but the project was ultimately abandoned. WWDC initiated further study of the Sandstone site in 1979. In 1985-86, SWEC, under contract to WWDC, conducted geological, geotechnical, hydrologic, and economic studies of the proposed dam and reservoir. These studies included dam foundation investigations and evaluation of large landslides in the reservoir basin. As the result of public comment, SWEC (1987) performed supplemental geotechnical studies focused mainly on seepage losses through the dam foundation and reservoir basin and the possible impact of large landslides on dam safety and reservoir sedimentation.

In 1992, Western Water Consultants, Inc. (WWC), as part of a basin planning study for the Little Snake River, evaluated alternative reservoir sites. This study concluded an RCC dam with an upstream membrane would be "most suitable" (economically) for the Sandstone site. Additional field and laboratory investigations, however, were recommended to determine the suitability of the site for the proposed RCC dam.
In 1993, SWEC drilled two core holes in the left abutment at the dam site and excavated test pits in the reservoir area. Laboratory testing included index properties and direct shear tests on clay seams recovered from core drilling. The objectives of the core drilling and laboratory testing was to "evaluate the presence, continuity, and strength of clay seams in the foundation rock" and to "evaluate the suitability of on site material for use as RCC aggregate."
6. REVIEW AND ANALYSIS

Our technical review focused on geological and geotechnical conditions which would potentially impact design and construction of a 130-foot high RCC dam at the Sandstone site. We identified five topics where geological and geotechnical uncertainty is large. Each of these topics and related uncertainty could affect design and construction of the project as presently envisioned:

1. Dam type and suitability to site conditions.
2. Landsliding and relation to foundation strength and reservoir rim stability.
3. Foundation strength and stability.
4. Seismotectonics.
5. Other issues including a proposed alternative dam alignment, necessity for additional site investigations, and estimated project costs.

We discuss each of these topics below.

6.1 DAM TYPE AND SUITABILITY TO SITE CONDITIONS

SWEC (1986, 1987) evaluated the suitability of the proposed Sandstone site for a zoned, rolled-earth embankment dam 200 feet high and impounding a reservoir of 52,000 acre-feet at a water surface elevation 6932 feet. The project, as originally proposed, was not permitted by the U.S. Army Corps of Engineers, due in part to an approximately 20,000 acre-feet of unidentified storage. As a result, the project was down-sized to 23,000 acre-feet of storage, and the embankment dam was replaced by an RCC dam 130 feet high (Figure 3).

An RCC dam is fundamentally a concrete gravity dam which requires a high shear strength, high bearing capacity foundation to resist sliding
and bearing failure. Concrete gravity dams are ideally constructed in high strength rocks without major structural discontinuities or zones of weakness. The U.S. Bureau of Reclamation, in its manual on design of gravity dams (USBR, 1976), indicates that the dam foundation should be relatively free of faults or shears that may require expensive treatment. Hansen and Reinhardt (1991), in their book on RCC dams, also indicate that sound rock foundations, lacking major faults or shears, are the most suitable for RCC dams. Otherwise, treatment of flaws may be expensive to ensure an adequately safe foundation. Hansen and Reinhardt (1991) also state:

"... The conditions from the rock surface down to 30 to 60 feet (10 to 20 m) are considered to be of the greatest importance, as they have the greatest effect on the ability of the foundation to withstand loads without unacceptable short-term and long-term deformations."

"Foundation investigation and design is probably more important than the design of the dam section itself. History has shown that the potential for failure of a concrete gravity dam, while extremely remote, is more likely in the foundation than in the dam itself. Special attention should be given to identifying potential sliding planes in the foundation rock . . ."

The siting and design of an RCC gravity dam must consider the suitability of the foundation and the cost of remedial treatment to ensure an adequately safe foundation. With these thoughts in mind, SWEC (1986) stated:

"The foundation (at the Sandstone site) is too weak for a concrete arch dam and is considered marginal for any concrete gravity structure including roller compacted concrete. In addition the irregular topography is poor for a concrete gravity structure." [Emphasis added]

WWC (1992) also recognized the marginal geological conditions at the Sandstone site:

"Current data are not adequate to determine strength characteristics of the foundation (which appear to be marginal) . . ."

"The previous work completed at the dam site indicates that the bedrock foundation may be marginal in terms of the stability and strength necessary to support an RCC dam because there is some indication of lower strength clay seams within the sandstone foundation . . ."

"Vertical construction joints through the dam are necessary in locations where foundation conditions (material type, ground surface slope, etc.) change abruptly. The joints will provide a means of reducing the potential for
differential settlements across the length of the dam by allowing independent movement of each section of the dam.

We concur with SWEC (1986) and WWC (1992) that the Sandstone site is marginal for an RCC dam. Given the geology of the site, relatively weak interbedded sedimentary rocks, an RCC dam is a marginal choice and will require additional engineering studies beyond those normally required for a more suitable RCC site. Even so, significant uncertainty will exist in the most thorough studies, requiring conservative design assumptions to be made. Our experience indicates that, for marginal projects with complex foundation conditions, design modifications and additional costs are more likely than in less complicated projects.

6.2 LANDSLIDING

Much attention has been paid in previous studies to landsliding (SWEC 1986, 1987), especially in the reservoir area upstream of the proposed dam site. The emphasis in these studies has been limited to the potential for catastrophic failure of existing landslides into the reservoir, which might threaten the safety of the dam, and on sediment contributed to the reservoir. We agree that these two issues are important and deserve treatment. However, we also believe the engineering implications of landsliding are far broader than these two issues alone. Moreover, we differ in opinion with regard to some landslide interpretations presented by SWEC (1986, 1987). Our discussion of landsliding is divided into the following topics:

2. Age of landsliding.
3. Effect of reservoir impoundment.
4. Implications for strength and stability of dam foundation.
6.2.1 Mechanics of Landsliding

Landslides are widely distributed in the valleys of Savery, Little Sandstone and Big Sandstone creeks upstream of the proposed dam site. As part of our analysis, we cataloged 102 landslides on the Tullis Quadrangle (Barclay, 1976) and assessed the stratigraphic location of the failure surface, the landslide crown, and the intervening bedrock units involved in sliding. Our field reconnaissance indicated that many existing and incipient slope failures have not been portrayed on published geologic maps (Barclay, 1976; Naftz and Barclay, 1991) or in reports prepared by SWEC (1986, 1987, 1993). Landslide types include translatory block failures, rotational slumps, earthflows, and combinations of these types. We conclude that landslides in the project area represent a variety of failure mechanisms in a wide range of bedrock and surficial units. Virtually all of the stratigraphic section from the Steele Shale through the Mesaverde Group to the Browns Park Formation is involved in landsliding in the project area.

The largest landslides on the east side of the Savery Creek valley are controlled by weak, planar bedrock dipping out of the valley slope. Large landslides on north-facing slopes in the Little Sandstone and Big Sandstone valleys appear to be controlled by a combination of weak bedrock and slope aspect. Dip of weak bedrock units out of the valley slope does not appear to be as important a control as in the landslides along Savery Creek. We note that the Little Sandstone and Big Sandstone landslides show a direction of movement virtually perpendicular to the direction of movement in the Savery Creek landslide, suggesting a different failure mechanism. Moreover, the Big Sandstone landslide initially occurred much later in time than the Savery Creek slide based on geomorphic evidence discussed in Section 6.2.2. This opinion differs from SWEC's (1986, 1987) interpretation of landslide age. Ground water is a significant contributing factor in all landslides in the project area.

SWEC (1986, 1987) attributes failure of the Savery Creek and Big Sandstone landslides to a bentonite bed identified in core holes drilled into the Savery Creek slide mass. Boring SB-8, drilled at the toe of Savery Creek landslide, encountered a 3.6-inch thick bentonite seam 138' below the surface. Boring SB-7, drilled at the top of a ridge within the slide mass, encountered a 8.4-inch thick bentonite seam 334' below the surface. An inclinometer installed in SB-7 measured 0.9 inches of movement in 3.5 months at the level of the bentonite layer.
SWEC (1987) concluded that this bentonite bed was the principal failure surface for the Savery Creek landslide. After six months, the inclinometer apparently was sheared off at the depth of the bentonite bed.

While not explicitly stated, SWEC (1986, 1987) implies the bentonite bed is in the upper Steele Shale. SWEC projected the bentonite layer, using the two drill holes in the Savery Creek landslide, and concluded that the same bentonite layer was responsible for the Big Sandstone Creek landslide. The technical basis for this interpretation, however, is not clear to us. SWEC (1986, 1987) attributes other landsliding in the project area to "small surficial soil movement." Whether intended or not, SWEC seems to imply that all major landsliding in the project area is attributable to this single bentonite bed.

We would argue that this single bed is only one of many weak claystone/shales in the stratigraphic section exposed in the Savery Creek and tributary valleys that is prone to landslide movement. Geologically, we interpret movement in the Savery Creek slide to be occurring along multiple failure surfaces above the bentonite bed identified by SWEC. We agree with SWEC (1986) that the bentonite seam identified in the Savery Creek landslide is well below the depth of consideration at the dam site. But given the complex nature of bedrock stratigraphy from the upper Steele Shale through the Mesaverde Group to the Browns Park Formation, it is implausible to argue that this single bentonite bed is responsible for all landsliding and that no other zones of weakness are present above the bentonite bed in question. Insufficient studies have been performed to evaluate the mechanisms and causes of landsliding in the project area.

SWEC (1986) indicates that ground water recharges bedrock formations to the east and migrates southwesterly, down-dip to discharge points (springs) along the east side of Savery Creek and its tributaries. Ground water is confined or semi-confined in sandstones "sandwiched" between siltstones and claystones/shales of the Mesaverde Group. Significant artesian pressures were noted in boring SB-6 at the dam site and SB-8 in the Savery Creek landslide. We believe high ground water pore pressure associated with the ground water regime described by SWEC is a critical factor in continuing landsliding in the Savery Creek, Little Sandstone, and Big Sandstone valleys. Increases in pore pressure due to long-term climatic changes, short-term climatic cycles, seasonal variations, and cultural modifications are the probable immediate triggers of landsliding in the project area.
In our opinion, current interpretations (SWEC, 1987) of landsliding in the project area are over-simplified. We believe it is highly unlikely that the bentonite bed identified in the Savery Creek landslide is responsible for all or a majority of the landslides in the project area. The interpretation that the Big Sandstone landslide results from movement along the same bentonite bed is arguable based on stratigraphic position and direction of movement. The majority of other landslides in the project area are located stratigraphically above the identified bentonite bed and, therefore, are not associated with it. Landsliding in the project area is a result of many factors, including stratigraphy, ground water conditions, topography, and slope aspect, that cannot be easily or simply characterized. Landslide movement occurs in a variety of stratigraphic units, and failure mechanisms are complex. It is clear, however, that Mesaverde rocks are locally weak and susceptible to landsliding, especially in the presence of ground water.

### 6.2.2 Ages of Landsliding

SWEC (1986, 1987) interprets large landslides in the project area to have developed in late Pleistocene time (10,000± years before present). The implication appears to be that these are "ancient" landslides and have little or no engineering significance at present because they developed under significantly different climatic conditions. We fundamentally disagree with this interpretation for two reasons. First, the landslides described as ancient are currently moving as evidenced by shearing of the deep inclinometer installed in the Savery Creek landslide and surface deformation (ground cracks, heaves, slumps, earthflows, etc.) characteristic of active landslide movement. To imply the landslides are ancient, based on the age of initial movement, is misleading.

Second, the Big Sandstone landslide, in our opinion, is much younger than late Pleistocene and may have been triggered only a few hundred to a few thousand years ago, certainly much less than 10,000 years ago. Geologic evidence indicates that movement of the Big Sandstone landslide blocked Big Sandstone Creek and the stream has only recently re-established its course through the slide mass. Landslide debris is exposed in the streamcut to the level of the existing channel. This is compelling evidence that sliding is geologically recent. During our field reconnaissance we observed landslide debris resting on colluvial deposits on the north side of Big Sandstone Creek, implying that slide...
movement was rapid, perhaps catastrophic, during initial failure and/or that landslide debris filled the valley to a higher level than at present. Surface cracking and active movement is evident in various parts of the landslide mass. Surface movement was also detected by SWEC during monitoring studies.

Some question exists as to whether the blockage of Big Sandstone Creek occurred as the result of slide movement to the north on the south valley slope or movement to the south on the north valley slope. Our reconnaissance studies suggest the landslide on the north valley wall was much more limited in extent than the south slope failure and probably did not reach the location of the existing stream channel. Moreover, slide debris resting on colluvial debris on the north valley slope near the west toe of the landslide mass could only have been deposited by a landslide originating on the south valley slope and moving to the north.

A landslide in Sec. 21, T.14N, R.88W, the Savery Creek landslide and other large landslides in the Savery Creek valley have locally constricted stream channels at their toes (Naftz and Barclay, 1991). Many other landslides show evidence of continuing movement and a number of incipient slope failures were observed indicating landsliding is a widespread and continuing geologic process in the project area.

6.2.3 Effect of Reservoir Impoundment

As discussed in Section 6.2.1, SWEC (1986) indicates that ground water recharges bedrock formations east of Savery Creek on the flanks of the Sierra Madre uplift and migrates southwesterly, down-dip in confined or semi-confined units in the Mesaverde Group. Significant artesian pressures were noted in borings, and numerous springs issue along the east side of Savery Creek. High ground water pore pressures associated with the ground water regime, in our opinion, is a critical factor in continuing landsliding in the Savery Creek, Little Sandstone, and Big Sandstone valleys. Pore pressure increases due to long-term climatic change, short-term climatic cycles, seasonal variations, and cultural modifications are the probable immediate triggers of landsliding in the project area.
Impoundment of Sandstone reservoir to elevation 6883 feet will inundate many of the existing springs and seeps along the east side of the Savery Creek valley. The net effect of reservoir impoundment over the long-term will be to create a local ground water table or potentiometric surface at about the same elevation as the reservoir pool. Given SWEC's (1986) interpretation that confined and semi-confined units within the Mesaverde Group are recharged in higher terrain to the east and ground water moves downdip to the southwest, pore pressures will not be relieved by spring discharge along the east side of the valley after impoundment of the reservoir. Several mapped springs and many un-mapped seeps discharge below the normal reservoir water surface elevation of 6883 feet. Because these discharge points will be inundated, pore pressures must increase in existing landslides and adjacent rock masses. Increases in pore pressures will accelerate movement in existing landslides and destabilize adjacent slopes. New bedrock and surficial landslides will likely occur as a result.

Much attention has been paid to the possibility of catastrophic failure of landslides in the reservoir area, the related threat to dam safety, and reservoir sediment loads (SWEC 1986, 1987). We reviewed SWEC's evaluation of these issues and supporting opinions presented by Dr. Ralph Peck. We agree with SWEC and Dr. Peck that existing landslides, though accelerated, will experience slow movements and catastrophic failure of existing slide masses does not pose a credible hazard.

Uncertainty exists in the potential for creation of new bedrock landslides in heretofore stable bedrock around the reservoir rim. Dr. Peck allows the possibility that initial landslide movement may be rapid as peak shear strength is reduced to residual strength along the incipient failure surface. Other factors, however, are necessary to create a catastrophic failure including steep slopes and an adversely oriented failure surface. Field evidence outlined in the preceding section suggests that the Big Sandstone landslide may have failed rapidly, perhaps catastrophically, damming the valley temporarily. If this interpretation is correct, the risk of catastrophic failure, especially on the east side of the reservoir basin is not diminishingly small as suggested by SWEC (1986, 1987).

We conclude, based on the interpreted ground water regime and active landsliding in the reservoir basin, that existing landslides will be accelerated, albeit slowly, and new slope failures will develop in surficial and bedrock materials. The locations, mechanisms, rapidity, and volume of landsliding that might be triggered by reservoir impoundment is
speculative. The risk of additional landsliding in the reservoir basin, however, is significant and cannot be discounted. The potential for catastrophic failure of currently stable rock masses, in our opinion, is not as small as might have been assumed by SWEC, especially in view of geologic evidence from the Big Sandstone landslide. We consider the volume of debris contributed by landsliding to the reservoir calculated by SWEC (1987) to be a minimum value based on reactivation and limited movement of existing landslide masses.

The environmental impact of reservoir basin landsliding potentially reaches far beyond the reservoir shoreline. Many of the landslides in the Savery, Little Sandstone, and Big Sandstone valleys encompass hundreds of acres and extend more than a mile from crown to toe. If marginally-stable existing landslides are reactivated/accelerated by reservoir impoundment, the effects potentially will be manifested over hundreds of acres extending of thousands of feet from the reservoir rim. If new landslides are triggered by reservoir impoundment, environmental disturbance will be compounded. While we believe landsliding is a natural geologic process in the Savery Creek valley and its tributaries, acceleration and enlargement of landsliding by reservoir impoundment is clearly an environmental impact of significant proportions.

6.2.4 Implications of Landsliding for Strength and Stability of Dam Foundation

The number, variability of failure mechanisms, and stratigraphic range of landslides in the project area indicates that strength of the bedrock in the dam foundation is a legitimate concern, especially for an RCC dam. These factors also indicate that assumptions regarding: (1) the possible presence of weak, incipient failure surfaces in the dam foundation and (2) shear strengths should be conservative. Reduction in conservatism should be based on well-documented technical data collected from site explorations, laboratory testing, and applicable experience from technically comparable projects.

We do not believe that landsliding can or should be severed from dam foundation strength and stability issues. Sandstone Dam is proposed for construction in bedrock of the Mesaverde Group. These same rocks are susceptible to massive landsliding elsewhere in the Savery Creek valley. Until a better understanding of landslide stratigraphy,
failure mechanisms, and local/regional ground water regimes is avail­
able, Mesaverde bedrock should be treated conservatively from a de­
sign viewpoint.

6.3 FOUNDATION STRENGTH AND STABILITY

SWEC (1986) and WWC (1992) identified two issues that, in their
opinion, rendered the proposed site marginal for RCC dam construc­
tion: (1) low strength clay seams in the foundation; and (2) irregular
topographic conditions in the dam footprint that would tend to concen­
trate stresses and induce cracking in the dam. We discuss these issues
below in light of the 1993 SWEC site investigations. Figures 4, 5, and
6 summarize our interpretations of foundation geology and geometry.

6.3.1 Foundation Clay Seams and Shear Strengths

A primary objective of the SWEC 1993 site explorations was to evalu­
ate the presence, continuity, and strength of clay seams in the founda­
tion rock. These clay seams were identified by SWEC (1986) and by
WWC (1992) as a design issue for either an embankment or an RCC
dam. The continuity and shear strength of clay seams in the foundation
of a proposed RCC dam is especially critical.

The 1993 site investigations performed by SWEC included drilling of
two core holes offset perpendicular to the dam axis from previous
SWEC boring SB-25. The two additional holes, SB-26 and SB-27,
were spaced approximately 23 and 48 feet, respectively, north from
SB-25. Investigations also included limited geologic mapping of out­
crops in the lower left abutment, laboratory testing of clay seam index
properties and shear strengths, and dam stability analyses.

Based on these studies, SWEC (1993) concluded that "some of the clay
seams are definitely continuous over several hundred feet and that the
clay seams dip in a downstream direction from 0.4° to 4.5°. Our inde­
pendent analysis indicates a downstream dip of 3° and that some of the
clay seams probably "daylight" on the downstream face of the bluff
which forms the left abutment of the dam site (Figures 4 and 5). SWEC's
interpretation and our analysis are consistent. Therefore,
given the continuity and geometry of clay seams in the left abutment area, the critical question becomes what shear strength should be used in stability analysis. SWEC attempted to address this question by laboratory testing of clay seams believed to be representative of actual foundation conditions.

Before continuing our discussion of SWEC test results, we observe that recovery and testing of representative soil rock/samples, including clay seams in bedded sedimentary rock, is difficult and fraught with problems. A significant and perhaps overriding question is whether samples recovered and tested in the laboratory are truly representative of actual foundation conditions. In other words, what is the range of uncertainty in the test results? No simple answer exists. The relative importance of the question lies in the sensitivity of critical engineering analyses to the test results. In some cases, uncertainty in test results has little significant impact. In other cases, for example a marginal site, the range of uncertainty may encompass a potentially "fatal flaw". Uncertainty takes on added importance for design of Sandstone Dam.

### 6.3.1.1 Sampling

Samples of intact clay seams were obtained by "PQ" (3-inch nominal diameter) core drilling in late August-September 1993. Clay seam specimens were waterproofed in the field using wax and cheese cloth, and samples were stored in a humid room on arrival at the laboratory. SWEC (1993) indicates that about half of the samples received were intact and the other half were "parted."

### 6.3.1.2 Intact Shear Strength Tests

SWEC attempted to develop a Mohr failure envelope using 3 intact clay seam specimens chosen from hole SB-27 at depths of 18.7, 23.0, and 79.1 feet; these specimens arrived at the lab in an intact state. Normal stresses of 50-52 psi, 101-102 psi and 210 psi were used to develop the envelope. For the 50 and 100 psi (normal stress) points, two residual strength tests were made by resetting the shear box after obtaining peak values. For the 210 psi normal stress, the peak strength test gave an anomalously high reading (SWEC, 1993, Fig. 13).
A reasonable Mohr envelope cannot be interpreted from this data for the peak shear strength of intact clay seams for the following reasons:

1. Test 2718.7 at 210 psi normal stress failed through siltstone-sandstone giving an anomalously high shear stress value. This test is not representative of the clay seam and cannot be used in evaluating the peak Mohr envelope for intact clay seams. This test was not used in SWEC's interpretation.

2. A line drawn through the other two peak strength points (tests 2723_OA and 2779_1 at 50 and 100 psi normal stress, respectively) results in a negative value of the angle of internal friction, $\Phi$. This result is not unexpected, however, since the materials used to develop these points come from different depths and stratigraphic units.

We conclude that the peak shear strength of the intact clay seams has not been defined by the laboratory testing program. The interpretation of the residual shear strength envelope (SWEC, 1993, Figure 13) is reasonable given the available data. We do not agree with SWEC that this test data is "inconsistent" as indicated on SWEC Figure 13. An examination of the performance of the intact clay seams during direct shear testing is instructive as discussed in the following paragraphs.

The set of points at 50 psi normal stress was derived from SB-27 at a depth of 23.0 feet. The specimen arrived at the SWEC laboratory intact. Shear stresses developed during shear testing were 25 psi, 9 psi and 1.2 psi for the peak and the two residual runs respectively. This shows a dramatic decrease in shear strength with increasing strain. A 64% loss in strength was measured during the first residual run and an additional 31% loss in strength was measured during the second residual run for a total of 95% strength loss with increasing strain. SWEC (1993) did not report the results from the final residual run on either the summary table (SWEC, 1993, Table 5) or on the shear strength plots (SWEC, 1993, Figure 13). Complete test results are included in the Appendix.

The set of points at approximately 100 psi show a different story. This set of points was derived from SB-27 at a depth of 79.1 feet. This specimen was also recorded as arriving at the Boston SWEC lab intact and in good shape. Shear stresses developed during shear testing (at normal stresses from 101-102 psi) were 24 psi, 20 psi, and 22 psi for the peak, first residual run and second residual runs, respectively. This
data shows only a slight drop in strength for the first residual run of 17%. During the second residual run, the shear stress developed was actually higher than for the first residual run showing only an 8% strength loss from the "peak" value. The nearness of these points to each other may be indicative that the intact shear strength of this clay seam may have already fallen to residual values in the field.

Clay seams in the dam foundation may have reached residual strengths through two processes: (1) movement related to landsliding; and (2) movement related to tectonic deformation. Landsliding is well-documented in Mesaverde Group rocks in the project area. No landslides are known to be present in the left abutment area at the proposed site. Minor slippage, however, may have occurred along weak zones (clay beds, seams, or partings) in the left abutment, which would produce residual strength values. It is also conceivable, however unlikely, that unrecognized, larger-scale, translatory landslide movement is present in the left abutment area. Enlargement of Black Lake Dam near Vail, Colorado, for example, disclosed an old translatory bedrock landslide overlying alluvium in the right abutment. The presence of this landslide was not detected by surface mapping or by pre-construction drilling. Bedrock exposed in the right abutment at Black Lake Dam appeared to be intact and in-place.

Tectonic deformation may induce slippage along bedding planes due to folding and warping of strata. Bedrock at the Sandstone site has been subjected to folding associated with uplift of the Sierra Madre to the east. Several faults have been mapped in vicinity of the Sandstone site, including a 14-mile long fault 1600 feet downstream. Folding and faulting in the vicinity of the site conceivably could reduce shear strength from peak to residual values along clay beds, seams, or partings in foundation bedrock.

6.3.1.3 Reported Inconsistencies with Published Data

SWEC (1993) indicates that the low strengths of the intact clay seams are inconsistent with the "published data." The published data SWEC refers to is a chart in Lambe and Whitman (1969, page 307). This chart relates plasticity index (PI) to peak shear strength for normally consolidated clay soil. The clay seams in question, however, are overconsolidated sedimentary bedrock and may be reduced to residual strength
values as discussed in Section 6.3.1.2. We do not believe that comparison of SWEC (1993) shear test results to Lambe and Whitman (1969) is necessarily valid or especially compelling, given the differences in the types of materials being compared and geologic origins.

Mitchell (1993, page 371) relates clay fraction and activity ratio to residual shear strength. Using this relation, a residual friction angle would be estimated at $13^\circ$-$14^\circ$ which is not much larger than the $10^\circ$ angle reported by SWEC (1993). We also note that SWEC (1987) estimated a $10^\circ$-$12^\circ$ friction angle for the bentonite bed believed to be the failure surface in the Savery Creek landslide. This would lend additional credibility to the presence of low strength clay seams in the sedimentary section in the Savery Creek valley. Bentonites are not stratigraphically restricted to the Steele Shale and may occur throughout the late Cretaceous section including the Mesaverde Group (Barclay, Personal commun., 1993). The friction angle of $10^\circ$ may represent a residual value in a CH clay and is not inconsistent with material type.

SWEC (1993) reports that clay fractions of samples at depths of 23.1 feet and 79.1 feet in SB-27 ranged from 41% to 42%. Our interpretation of the gradation results in Appendix C-2 (SWEC, 1993) indicates clay percentages ranging from 45% to 48% for these samples. Both samples classify as a plastic clay (CH). Higher percentages of plastic clay (CH) versus low plasticity clay (CL) will result in lower observed shear strengths.

6.3.1.4 Remolded Composite Samples

SWEC (1993) tested a set of remolded, composite samples to evaluate shear strength (SWEC, 1993, Figure 13). These specimens were remolded and consolidated to the same stress used during shear testing of intact clay seam samples; therefore, they are normally consolidated. These tests show consistently higher strength values than for intact specimens. Composite friction angles ranged from $20^\circ$ to $24^\circ$. Index tests on composite samples show clay fractions ranging from 33%-40% compared to 45%-48% for intact clay seams. Atterberg limits are reported for one of the composite samples as LL = 40.8 and PI = 23.3. This is a 10% lower liquid limit than the intact clay seam specimens. Generally, lower liquid limit (-10%) and lower clay fraction (-5%) to
-15%) would be reflected by a higher friction angle, which is indeed the case.

An important question is whether remolded, composite samples are representative of intact clay seams in the dam foundation. The composite samples are obtained by combining materials from several clay seams. Accordingly, index properties reflect all the materials combined. The composite samples are man-made and cannot be representative of the weakest clay seam except perhaps under the most ideal and fortuitous conditions. Index test data presented by SWEC (1993) show that the composite samples vary significantly from the weakest material identified through intact clay seam testing.

The geologic fabric of the natural undisturbed clay seam material is destroyed by remolding. Anisotropy in the natural clay seam (sedimentary partings, bedding, laminations, joints, slickensides, etc.), which can directly affect shear strength, is lost; and a homogenous sample is created. The strength of remolded samples should be higher than intact clay seams due to the averaging of index properties of different materials and the destruction of geologic fabric present in natural, intact samples. SWEC (1993) direct shear testing data are consistent with this expectation.

The use of composite samples in direct shear testing is common; we have used it in our own work. The results, however, must be used with judgment and caution for the reasons outlined above. The SWEC intact and composite test data show a range in friction angles of 10° to 24°. We contend that a friction angle of 10° is probably most representative of the weakest, intact clay seams in the dam foundation. The higher friction angles of 20°-24°, obtained from composite samples, are not representative of intact clay seams in the dam foundation. Moreover, we do not see the inconsistency in low strength data (friction angle of 10°) that SWEC (1993) observes. If problems exist with test data that produced a 10° friction angle, SWEC (1993) has not presented compelling evidence to that argument. In any case, prudent engineering analyses would require use of a 10° friction angle in stability analyses.
6.3.2 Stability Analyses

We concur with the general criteria and geometry of the SWEC stability analyses. Use of a 10° friction angle is appropriately conservative given the uncertainty in test results. The resulting factor of safety against sliding of 1.6 in the left abutment is less than the 2.0 recommended by the U.S. Army Corps of Engineers, 3.0 recommended by the Federal Energy Regulatory Commission, and 4.0 recommended by the U.S. Bureau of Reclamation for normal loading conditions. Use of a higher friction angle, 24°, derived from composite samples and reduced to 22° for strain compatibility results in higher factors of safety. Seismic loads, used by SWEC (1986), are not included in current stability analyses. USBR (1976) indicates that the factor of safety against sliding applies to "... any section of the structure ..." We interpret this to mean that all parts (sections) of the dam and foundation must meet or exceed the required factor of safety.

In our opinion, any decision to use friction angles higher than the reported 10° will require additional study, documentation, and justification. We do not believe adequate documentation and justification has been presented to dismiss a 10° friction angle in favor of exclusive use of higher strength values.

6.3.3 Irregular Foundation Geometry

Irregular topographic conditions (Figure 5) in the foundation of the proposed RCC dam could result in stress concentrations and potential for cracking in the RCC section. The recommendation that topographic irregularities be handled by vertical construction joints is expedient and, in our opinion, probably not an ideal engineering solution. In general, all abrupt grade changes in the dam foundation should be removed or shaped to accept the dam. In this regard, the quantity of rock excavation, as presently envisioned by SWEC (1993), may be underestimated.
6.4 SEISMOTECTONICS

SWEC (1986) conservatively assumes that the 14-mile long fault downstream of the dam is seismogenic. A Maximum Credible Earthquake (MCE) and resultant ground motions are assigned for use in the seismic stability analysis. This analysis has not been updated for the proposed RCC dam using strength data generated from the 1993 site explorations. Moreover, we question use of the half-fault length method to estimate the MCE for a short, local fault. This concept is generally applied to long, regional faults where either segmentation data is absent or certainty exists that the entire fault would not rupture in a single earthquake. If the assumption is made that the fault is active and no information is provided on segmentation, then the entire 14-mile fault length should be used to estimate the MCE. A 14-mile rupture length is well within the range of documented surface ruptures in Wyoming and the intermountain west.

6.5 OTHER ISSUES

6.5.1 Alternative Alignment

SWEC (1993) identified a possible alternative (Alignment B) to the dam alignment studied (Figure 3). The alternative alignment rotates the right abutment about 450 feet upstream. The benefits of the alignment shift are believed to be twofold: (1) the thick alluvial fan deposit in the lower right abutment and foundation area, which requires excavation, would be avoided; and (2) the shift upstream would mobilize additional shear strength along potential failure planes in the left abutment.

The right side of the dam in the proposed alternative will abut a narrow bedrock spur rather than the declivity in the valley wall at the original site. Our experience indicates that protruding bedrock spurs may not provide ideal abutment conditions because the rock mass tends to disintegrate due to stress relief. The net effect is that highly jointed and fractured bedrock in the protruding spur must be overexcavated to sound rock, negating the apparent topographic benefit of the site and apparent cost savings. No studies have been performed by SWEC to evaluate this condition.
6.5.2 Site Explorations

Studies conducted by SWEC (1986, 1987, 1993) and our independent review (this report) disclosed several potentially significant technical issues. Additional studies are required, in our judgment, to more fully address these issues and to evaluate that actual significance to project design and operation. These issues and additional studies include:

1. The continuity and physical characteristics of clays seams and other potentially weak beds in the dam foundation.

2. The shear strength of clays seams in the dam abutment and foundations given the uncertainty in 1993 results.

3. The characteristics of the alternative dam foundation in terms of items 1 and 2 above and the competency of the rock mass in the proposed alternative right abutment.

The footprint of the dam foundation is 1830 feet long by 200 feet wide at its widest point in the valley bottom. The 1993 site explorations, intended to provide detailed information on stability of the dam foundation, encompassed a section of the left abutment perpendicular to the axis over a distance of 48 feet between boring SB-25 and SB-27. Confidence in interpretation was extended laterally to the northwest by geologic mapping of outcrops. Nevertheless, this represents a very small sample of the overall dam foundation, especially considering the significance of the design issues and uncertainty in results. Given the variability of the bedrock in the dam foundation (Figure 5), additional investigations will be required before reasonable conclusions can be drawn regarding the project as presently envisioned.

Shifting the dam axis to the north as proposed in the alternative fundamentally creates a new dam site with little reliable, site-specific, foundation data. Although existing subsurface explorations (SWEC 1986, 1993) provide useful information and guidance at the new site, little if any reliable subsurface information is available within the footprint of the alternative dam foundation. Test holes drilled by USBR in 1976 are neither detailed nor useful in addressing foundation stability issues, especially in comparison with subsequent drilling and logging performed by SWEC (1986, 1993). In our judgment, USBR foundation data within the footprint of Alignment B provide insufficient technical bases for feasibility designs and cost estimates.
Cost estimates in the WWC (1992) and SWEC (1993) reports are confusing and potentially misleading to a casual reader. The WWC (1992) cost estimates include engineering, permitting, and contingencies (15% across the board) normally associated with dam construction. All of these costs are real and must be paid to construct Sandstone Dam. The WWC (1992) cost estimate for Sandstone Dam totaled $30,000,000. The SWEC (1993) cost estimate totaled $25,295,000 for Alignment A but does not include permitting, engineering, and contingencies. We understand SWEC was directed by WWDC to estimate only "hard" construction costs without permitting, engineering, and contingency costs. The difference in cost between the WWC (1992) and SWEC (1993) estimates is minus $4,705,000. We believe this apparent reduction in cost is potentially misleading because of the different basis used in arriving at the cost estimates.

To arrive at a more directly comparable estimate using SWEC (1993) data, we reevaluated costs based on the WWC (1992) approach (Appendix II). SWEC's 1993 volumes and cost estimates were used with the addition of permitting, engineering, and contingency costs (15%) from WWC (1992). Because we believe that uncertainty exists in the amount of excavation required in the foundation of the dam and the resultant volume of RCC required, we increased the contingency factor for these items from 15% to 25%. We believe this is a realistic contingency factor for these items. The resulting total cost is $33,306,000 for Alignment A, $3,306,000 more than the WWC (1992) estimate and $8,011,000 more than the SWEC (1993) estimate.

Using the same approach for the SWEC (1993) alternative site (Alignment B), we estimate a cost of $31,121,000 compared to the SWEC estimate of $23,285,000, an increase of $7,836,000. We consider cost estimates for the alternative site, however, to be uncertain due to the relative absence of site-specific information, especially the nature of the rock mass and amount of required overexcavation in the right abutment.
7. CONCLUSIONS

1. An RCC dam requires a high shear strength, high bearing capacity foundation to resist sliding and bearing failure. RCC dams are ideally constructed in high strength rocks without major structural discontinuities or zones of weakness.

2. SWEC (1986) stated: "The foundation (at the Sandstone site) is too weak for a concrete arch dam and is considered marginal for any concrete gravity structure including roller compacted concrete [emphasis added]. In addition, SWEC concluded "the irregular topography is poor for a concrete gravity structure." WWC (1992) also recognized marginal geological conditions at the Sandstone site.

3. An RCC dam is a marginal choice for the Sandstone site and will require additional engineering studies beyond those normally required for a more suitable site. Even so, significant uncertainty will exist, requiring conservative design assumptions to be made.

4. The emphasis in previous reservoir landslide studies has been on catastrophic failure, which could threaten the safety of the dam, and on sediment volumes contributed to the reservoir. However, the engineering implications of landsliding are far broader than these two issues alone.

5. Current interpretations of landsliding in the project area are oversimplified. Landslide movement occurs in a variety of stratigraphic units, and failure mechanisms are complex. Bedrock is locally weak and susceptible to landsliding, especially in the presence of groundwater.

6. Large landslides in the project area, previously interpreted to be ancient (10,000± years old) and having little engineering significance, are currently experiencing deep-seated and near surface movement. The Big Sandstone landslide is much younger than 10,000 years and may have been triggered only a few hundred years ago. Many other landslides show evidence of continuing movement, and a number of incipient slope failures in the project area indicate landsliding is a continuing geologic process.
7. Impoundment of Sandstone reservoir will inundate many of the existing springs and seeps along the east side of the Savery Creek valley. Given the interpretation that confined and semi-confined units within the Mesaverde Group are recharged in higher terrain to the east and ground water moves downdip to the southwest, pore pressures must increase in existing landslides and adjacent rock masses on the east side of the reservoir. Existing landslides will be reactivated and new bedrock and surficial landslides will occur as the result of pore pressure increases.

8. Uncertainty exists in the potential for new bedrock landslides in heretofore stable bedrock around the reservoir rim. The possibility exists that new landslide movements may be rapid as peak shear strength is reduced to residual strength along incipient failure surfaces. Geologic evidence indicates that the Big Sandstone landslide may have failed rapidly, perhaps catastrophically, damming the valley temporarily.

9. The volume of debris contributed by landsliding to the reservoir may be underestimated based on assumption of limited movement of existing landslide masses.

10. The number, variability of failure mechanisms, and stratigraphic range of landslides in the project area indicate that strength of the bedrock in the dam foundation is a legitimate concern, especially for an RCC dam. Assumptions regarding: (1) the presence of weak, incipient failure surfaces in the dam foundation and (2) shear strengths should be conservative. Any reduction in conservatism must be based on well-documented justification.

11. Low strength clay seams are continuous over several hundred feet, dip in a downstream direction from 0.4° to 4.5°, and daylight on the downstream face of the bluff which forms the left abutment of the dam site. Given the continuity and geometry of clay seams in the left abutment, shear strength of the clay seams is a critical issue.

12. Laboratory direct shear testing of intact clay seams produced uncertain results. The peak shear strength of intact clay seams has not been defined by the laboratory testing program. The interpretation of the residual shear strength envelope is reasonable given the available data. We do not agree with SWEC that this test data is "inconsistent." While differences of opinion may exist with regard to causes and interpretations of these laboratory results, the existence of low strength
friction angle = 10°), continuous clays seams must be considered in project design.

13. SWEC (1993) indicates that low strengths of the intact clay seams are inconsistent with the "published data", relating plasticity index to peak shear strength for normally consolidated clays. The clay seams in question, however, are overconsolidated and exhibit a 10° residual friction angle. The comparison made by SWEC to published data is not well drawn. A 10° residual friction angle is consistent with clay fraction and activity ratio.

14. The shear strength of remolded, composite samples is higher than naturally-occurring, intact clay seams due to the averaging of index properties and the destruction of geologic fabric in intact samples. Direct shear test data are consistent with this expectation. The higher friction angles of 20°-24°, obtained from composite samples, are not representative of the weakest intact clay seams in the dam foundation. Prudent engineering analyses requires use of a 10° friction angle in stability analyses.

15. The general criteria and geometry of the SWEC stability analyses are reasonable. Use of a 10° friction angle is appropriately conservative given the uncertainty in laboratory test results. The resulting factor of safety against sliding of 1.6 in the left abutment is less than the 2.0 recommended by the U.S. Army Corps of Engineers, 3.0 recommended by the Federal Energy Regulatory Commission, and 4.0 recommended by the U.S. Bureau of Reclamation for normal loading conditions. Seismic loads are not included in current stability analyses. Use of friction angles higher than the reported 10° in stability analyses will require additional study, documentation, and justification. Adequate documentation and justification has not been presented to dismiss a 10° friction angle in favor of exclusive use of higher values.

16. Irregular topographic conditions in the foundation of the proposed RCC dam could result in stress concentrations and potential for cracking in the RCC section. The recommendation that topographic irregularities be handled by vertical construction joints is expedient and probably not an ideal engineering solution.

17. SWEC (1986) conservatively assumes that the 14-mile long fault downstream of the dam is seismogenic (active). A Maximum Credible Earthquake (MCE) and resultant ground motions related to this fault are used in the preliminary seismic stability analysis. This analysis has
not been updated for the proposed RCC dam using strength data generated from the 1993 site explorations. If the assumption is made that the fault is active and no information is available on segmentation, the entire 14-mile fault length should be used to estimate the MCE. A 14-mile rupture length is well within the range of documented surface ruptures in Wyoming and the intermountain west.

18. The right side of the proposed alternative alignment will abut a narrow bedrock spur rather than a declivity in the valley wall at the original site. Protruding bedrock spurs may not provide ideal abutment conditions because the rock mass tends to disintegrate due to stress relief. The net effect is that highly jointed and fractured bedrock must be overexcavated, negating the apparent topographic benefit of the site and cost savings. No studies have been performed to evaluate this condition.

19. Given the variability of the sedimentary bedrock in the dam foundation and critical design issues, additional investigations will be required before reasonable conclusions can be drawn regarding the project as presently envisioned. Shifting the dam axis to the north fundamentally creates a new dam site with little reliable site-specific foundation data. Although existing subsurface explorations provide useful guidance at the new site, little if any reliable subsurface information is available. USBR foundation data provide insufficient technical bases for feasibility designs and cost estimates.

20. Cost estimates in the WWC (1992) and SWEC (1993) reports are confusing. The WWC (1992) cost estimate, including engineering, permitting, and contingencies (15%), totaled $30,000,000. The SWEC (1993) cost estimate for Alignment A totaled $25,295,000 but does not include permitting, engineering, and contingencies. Accordingly, costs were reevaluated based on the WWC (1992) approach. Because uncertainty exists in the amount of excavation required in the dam foundation and the resultant volume of RCC required, the contingency factor for these items was increased from 15% to 25%. The resulting total cost is $33,306,000 for Alignment A. Using the same approach for the alternative site (Alignment B), a cost of $31,121,000 was estimated. Cost estimates for the alternative site, however, are uncertain due to the relative absence of site-specific information.
8. GENERAL INFORMATION

Information presented in this report is intended to provide an independent technical review of the proposed Sandstone Dam and Reservoir Project; no other use is intended. It is based on review of the proposed construction; previous work and reports prepared by Stone & Webster Engineering Corporation, Western Water Consultants, the U.S. Bureau of Reclamation and other individuals/entities; observation of 1993 site investigations; a reconnaissance of the dam and reservoir site; air photo interpretation; and our general experience with similar projects and geological/geotechnical conditions. The opinions, conclusions, and recommendations presented are subject to the limitations and explanations contained herein.

MICHAEL W. WEST & ASSOCIATES, INC.

By: Michael W. West, Ph.D., P.E., P.G.
    Wyoming Professional Engineer (Civil)
    Wyoming Professional Geologist
9. REFERENCES CITED


Sandstone Dam Location Map

By J.D.G. January, 1994
Proj 93185 FIGURE 1

Stratigraphic Column from Naftz and Barclay, (1991)
Modified by Barclay, personal communication, 1994, in this report.
NOTES

1. Topography from SWEC (1986).
2. Complete geology not shown, upper part of Haystack Mountains Formation shown for stratigraphic correlation purposes only.
4. Upper part of Haystack Mtns. Fm. denoted by hatch pattern.
5. Refer to Geologic Cross Sections in Figures 4, 5, and 6.
Geologic Cross Section A–A’ through left abutment in line with SWEC BORING SB–26

**NOTES**

1. Dam cross section shown with 20 feet of overexcavation into bedrock.
2. Refer to Figure 3 for location of cross section.
3. Component of dip calculated to be 3° downstream from three point data presented on Figure 3.
4. Khm refers to middle part of Haystack Mountains Formation.
Geologic Cross Section B–B’ through left abutment in line with Bureau of Reclamation DRILL HOLE DH–2.

NOTES
1. Dam cross section shown with 20 feet of overexcavation into bedrock.
2. Refer to Figure 3 for location of cross section.
3. Component of dip calculated to be 3° downstream from three point data presented on Figure 3.
5. Khm refers to middle part of the Haystack Mountains Formation.
6. Refer to SWEC (1986) for drill log of hole DH–2.

Michael W. West & Associates, Inc. Geologic Cross Section B–B’

By: J.D.G. Jan. 1994
Proj: 93185 Fig. 5
Geologic Cross Section C–C’ along proposed ALIGNMENT "A".

NOTES
1. Stratigraphic column from (Naftz and Barclay, 1991). refer to Figure 2 for detailed description.
2. Correlations for SB-26 and SB-23 from (Barclay, personal communication, 1994, in this report.), exact correlation points indicated by arrow "-".
3. Refer to Figure 3 for location of Cross Section C–C’.
4. Khu, Khm, and KhL refer to upper, middle and lower parts of Haystack Mountains formation.
6. Alluvium overlying bedrock on abutments not shown here.

Proj: 93185  Fig. 6
APPENDIX I
January 17, 1994

Dr. Michael W. West  
Michael W. West and Associates  
290 Bank Western Building  
8906 west Bowles Avenue  
Littleton, CO 80123

Dear Dr. West:

Many thanks for arranging for me to look at four cores from the Haystack Mountains Formation, Mesaverde Group(Upper Cretaceous), which were drilled by Stone and Webster as part of a study of the proposed Sandstone Reservoir damsite near Savery, Carbon Co., WY, for the Wyoming Water Commission. My conclusions as to what lithofacies of the Haystack Mountains Formation are represented in the core are summarized below.

The two cores I looked at in Stone and Websters’ offices on November 9, 1993, are from holes SB-26 and SB-27, which were drilled about 25 ft apart in the NW/4 NE/4 SE/4 of section 2, T. 13 N., R. 89 W., 6th P. M., on a terrace east of Savery Creek and near the proposed location of the dam axis. From an inspection of the geologic map of the proposed Sandstone Reservoir (Naftz and Barclay, 1991, Pl. 1) and a cursory examination of the two cores, I conclude (1) that the drill holes penetrated about 113.5 ft of the Haystack Mountains Formation below 8 ft of Quaternary terrace gravel; (2) that the top of the cored-interval of the Haystack Mountains Formation begins near the base of a lower shoreface deposit, which comprises the middle, cliff-forming sandstone of the sandstone of Savery Creek; and (3) that the bottom of the drilled-interval—and of the drill holes—is in thinly interbedded sandstone and siltstone-mudstone of the transition zone below the lower shoreface deposit in the lower part of the lower, cliff-forming sandstone of the sandstone of Savery Creek. Tables 1 and 2 are logs of the cores from the two drill holes. The logs lack the sedimentologic detail which might have supported a more accurate stratigraphic assignment of the cored intervals than I have given and which would be useful in evaluating the physical properties of the rock at the proposed damsite. I was unable to obtain that detail in the time available for logging because of the poor lighting and space conditions in the highly trafficked corridor in which the cores were displayed.

The two cores that Jim Gill of your office and I looked at on December 17, 1993, in the basement of a State of Wyoming warehouse on Pacific Ave. at Parsley Dr., Cheyenne, WY, are from drill-holes SB-22 and SB-23 in the NE1/4 NW1/4 SE1/4 of section 2 in the Savery Creek flood plain and near the axis of the proposed dam. Both holes were drilled at an angle of 60 degrees from horizontal, SB-22 on the west side of Savery Creek with a north-northwesterly plunge, and SB-23 on the east side with a southeasterly plunge. Both holes were drilled through thick, unconsolidated, alluvial deposits of Quaternary age into sandstone beds of the lower shoreface deposit in the lower part of the lower, cliff-forming sandstone of the sandstone of Savery Creek, and bottomed in the thinly interbedded sandstone and siltstone-mudstone of the underlying
transition zone. Jim Gill and I worked together in describing these cores. Table 3 is a log of the core from SB-23. No log was prepared of SB-23 other than to note that the contact between lower shoreface deposits and deposits of the underlying transition zone is at a depth of about 113.7 ft in SB-22.

The interval in the Haystack Mountains Formation that was drilled in the 4 cores is indicated on a copy (fig. 1) of the columnar section of Mesaverde Group rocks from Naftz and Barclay (1991) and on outcrop pictures of the sandstone of Savery Creek (Figs. 2-4).

As we agreed, I am sending copies of this letter with tables and figures to David Jurich of Stone and Webster in Denver.

Sincerely yours,

C. S. Venable Barclay
In all logs sandstone color is designated as "light gray" (or "lighter gray" where comparing to siltstone-mudstone), but actually ranges from light gray, to yellowish-gray, to pale orange. Siltstone-mudstone color, described as "gray" or "darker gray", ranges from gray, to dark gray, to olive gray, to brownish gray.

In logs of SB-26 and SB-27, estimates of grain size are not more accurate than plus or minus one-half of a Wentworth size-class.

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2In logs sandstone color is designated as "light gray" (or "lighter gray" where comparing to siltstone-mudstone), but actually ranges from light gray, to yellowish-gray, to pale orange. Siltstone-mudstone color, described as "gray" or "darker gray", ranges from gray, to dark gray, to olive gray, to brownish gray.

3In logs of SB-26 and SB-27, estimates of grain size are not more accurate than plus or minus one-half of a Wentworth size-class.
Table 2. Log of drill-hole SB-27

<table>
<thead>
<tr>
<th>Unit</th>
<th>Depth(ft)</th>
<th>Thickness(ft)</th>
<th>Lithologic description</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0-8</td>
<td></td>
<td>Gravel. [No core; description from Jurich.]</td>
<td></td>
</tr>
<tr>
<td>2a</td>
<td>8-12.1</td>
<td>4.1</td>
<td>Sandstone, weathered. [No core; description from Jurich]</td>
<td></td>
</tr>
<tr>
<td>2b-3</td>
<td>12.1-35.6</td>
<td>23.5</td>
<td>Sandstone, light gray, mostly very fine-grained, and darker gray shaly mudstone-siltstone, thinly bedded and laminated. Sandstone is commonly finet-bedded, ripple-laminated, and cross-bedded.</td>
<td>Probably part of lower shoreface deposit comprising middle, cliff-forming sandstone of the sandstone of Savary Creek (Naftz and Barclay, 1991, Pl. 1).</td>
</tr>
<tr>
<td>4a</td>
<td>35.6-42.1</td>
<td>6.5</td>
<td>Sandstone, light gray, mostly medium-grained, few shaly partings below 39.6 ft; cross-bedded. Contains horizontal and inclined Ophiomorpha.</td>
<td></td>
</tr>
<tr>
<td>4b</td>
<td>42.1-73.4</td>
<td>31.3</td>
<td>Sandstone, light gray, probably fine- and medium-grained, coarsens up; contains few shaly streaks. Crossbedded, few thin, ripple-laminated, flaser-bedded intervals. Contains few thin (&lt;0.03 ft), gray, shaly partings below 67 ft.</td>
<td></td>
</tr>
<tr>
<td>5a</td>
<td>73.4-81.7</td>
<td>8.3</td>
<td>Sandstone, light gray, probably fine- and very fine-grained, crossbedded. Not as massive, contains more gray, shaly streaks than sandstone above. Crossbedding at lower angles and in thinner sets than in interval above, and some sets contain thin, ripple-laminated flaser-bedded intervals. Basal 2.5 ft mostly parallel laminated and sub-horizontally bedded. Interval contains Ophiomorpha.</td>
<td></td>
</tr>
<tr>
<td>5b</td>
<td>81.7-91.2</td>
<td>9.5</td>
<td>Sandstone, light gray, probably very fine- and fine-grained, with darker gray shaly streaks and laminite. Mostly sub-parallel laminated and sub-horizontally bedded (&lt;1 ft) bedded. More Ophiomorpha – most horizontal – than above. Lower shoreface deposit in lower part of cliff-forming sandstone of the sandstone of Savary Creek (1991). On outcrop, some hummocky cross-stratified sandstone beds (1-2 ft thick) and thin (0.05-0.4 ft) beds of interlaminated shaly siltstone-mudstone and ripple-laminated sandstone in this lithofacies are laterally persistent (see fig. 4). [Core removed 77.4-79.2 ft].</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>91.2-121.3</td>
<td>30.1</td>
<td>Sandstone, light gray, mostly very fine-grained, and darker gray shaly mudstone-siltstone, thinly bedded and pervasively &quot;mottled.&quot; Strong bioturbation generally obscures bedding structures. Interval coarsens up, and sandstone beds thicken up, from shaly mudstone with lenticular sandstone laminite, in lower part, to sandstone in thin (most &lt;0.1 ft thick), ripple-laminated beds, containing thin (mm-scale) streaks and laminites of shaly mudstone, in upper part. Sandstone dominant above 106 ft, siltstone-mudstone below! White-walled burrows are numerous. Bacculite fragment at 111.3 ft. Transition zone deposit below lower shoreface in lower part of lower, cliff-forming sandstone of the sandstone of Savary Creek (1991). Interval may include part of transition zone deposit between the sandstone of Savary Creek and underlying offshore deposit.</td>
<td>Transition zone deposit below lower shoreface in lower part of lower, cliff-forming sandstone of the sandstone of Savary Creek (1991). Interval may include part of transition zone deposit between the sandstone of Savary Creek and underlying offshore deposit.</td>
</tr>
<tr>
<td>Unit</td>
<td>Depth(ft)</td>
<td>Thickness(ft)</td>
<td>Lithologic description</td>
<td>Remarks</td>
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<td>------</td>
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<td>---------</td>
</tr>
<tr>
<td>1</td>
<td>0-24.2</td>
<td>24.2(21)</td>
<td>Mud, silt, sand and gravel. [No core: based on information from Stone and Webster report.]</td>
<td>Quaternary valley-fill, including modern floodplain deposits.</td>
</tr>
<tr>
<td>2</td>
<td>24.2-39.6</td>
<td>15.4(13.3)</td>
<td>Sandstone, light gray, mostly very-fine-grained and lower fine-grained. Coarsens up: mostly upper very fine-grained and lower fine-grained, ranging to upper fine-grained, above 32.8 ft; mostly massive, but contains some coaly and dark gray, shaly streaks, ripple-laminated flasers, and <em>Ophiomorpha</em>. Below 32.8 ft, more shaly streaks and laminae, more <em>Ophiomorpha</em>, and some &quot;mottled&quot;, strongly bioturbated intervals.</td>
<td>Lower shoreface deposit in lower part of lower, cliff-forming sandstone of the sandstone of Savery Creek(1991). Correlate with lower part of unit 5 in SB-26 and in SB-27.</td>
</tr>
<tr>
<td>3a</td>
<td>39.6-54</td>
<td>14.4(12.5)</td>
<td>Sandstone, light gray, mostly very-fine-grained, with numerous coaly and dark gray shaly streaks and laminae, flute-beded and ripple-laminated. Sandstone fines and amount of shaly siltstone-mudstone increases with depth, so that near base, some intervals consist of wavy-bedded siltstone-mudstone and sandstone. <em>Ophiomorpha</em> and smooth white-walled burrows, commonly 9-13 mm in diameter, are numerous.</td>
<td>Transition zone deposit below lower shoreface deposit in lower part of lower, cliff-forming sandstone of the sandstone of Savery Creek(1991). Correlate with upper part of unit 6 in SB-26 and in SB-27.</td>
</tr>
<tr>
<td>3b</td>
<td>54-62.2</td>
<td>8.2(7.1)</td>
<td>Gray siltstone-mudstone with lenticular laminae and thin beds of light gray, very-fine-grained silty sandstone and coarse siltstone, wavy, high-index ripple-laminations. Contains some distinct white-walled burrows. Most of interval &quot;mottled&quot;, strongly bioturbated.</td>
<td>Transition zone deposit(continued). Correlate with part of unit 6 in SB-26 and in SB-27.</td>
</tr>
<tr>
<td>3c</td>
<td>62.2-80</td>
<td>17.8(15.4)</td>
<td>Siltstone, clayey, to shaly mudstone, gray, containing lighter gray, coarse siltstone and silty, very-fine-grained sandstone laminae, lenticular-bedded and rippled, to wavy-bedded. Some parts of interval are &quot;mottled&quot;, but much less bioturbation and fewer large(10-mm)-diameter white-walled burrows than in interval above; mud-filled burrows, 1-2 mm. diameter, are numerous in siltstone and sandstone laminae.</td>
<td>Transition zone deposit(continued). Most of interval correlates with lower part of unit 6 in SB-26 and in SB-27. Basal 7 ft (vertical) is probably from stratigraphically lower interval than unit 6 and part of transition zone deposit between sandstone of Savery Creek and underlying offshore deposit.</td>
</tr>
</tbody>
</table>
Figure 1. Partial section of Haystack Mountains Formation, showing intervals cored in SB-23, SB-26, and SB-27 (modified from Naftz and Barclay, 1991; pl. 1).
Partial section of Haystack Mountains and Allen Ridge Formations measured in the northwestern part of sec. 31, T. 14 N., R. 88 W., 6th P.M., Carbon County, Wyoming. Modified from Barclay (1976)
Figure 2. Outcrops of Haystack Mountains Formation on north side of Big Sandstone Creek, where section shown on figure 1 was measured. [Tbp = Browns Park Formation; Karl = lower, nonmarine part of Allen Ridge Formation; Khu = upper part of Haystack Mountains Formation; Khm = middle part of Haystack Mountains Formation, with sandstone of Savery Creek in the upper part; Khl = lower part of Haystack Mountains Formation.]
Figure 3a-b. Lithofacies in the sandstone of Savery Creek shown on figure 1 and exposed in cliffs on right side of figure 2. [0 = offshore shale above and below sandstone of Savery Creek; 1 = transition zone; 2 = lower shoreface; 3 = upper shoreface.]
Figure 4. Close view of outcrop on right side of figure 3A of part of lower shoreface deposit in lower, cliff-forming sandstone of sandstone of Savery Creek. The outcrop is a section, about 17 ft thick, of well-beded sandstone, which coarsens up from very-fine-grained and lower fine-grained, to fine-grained, and near the top, to upper fine-grained and lower medium-grained. Bedding thickens from 0.1 to 0.5 ft in lower part to mostly 0.5-2.0 ft in upper part. Beds are sub-parallel and sub-horizontally laminated, and hummocky cross-stratified, and are typically separated by thin (0.05-0.3 ft) beds of shaly mudstone-siltstone and ripple-laminated sandstone. Some shaly intervals can be traced several tens to a few hundreds of feet along outcrop. Ophiomorpha and Thalassinoides are the principal trace fossils in sandstone beds and shaly beds are typically strongly bioturbated. Rock hammer, which is about 11 inches long from top of head to end of handle, is head-down on top of 1.7-ft-thick, hummocky-crossbedded sandstone, which is overlain by about 0.3 ft of interlaminanted, shaly siltstone-mudstone and ripple-laminated sandstone. [US = upper shoreface; LS = lower shoreface; HCS = hummocky crosstratified sandstone bed.]
APPENDIX II
Cost Estimates for Alignments A and B
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