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Feasibility Level Geological and Geotechnical Investigation for Union Park Dam

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ABSTRACT: Union Park Dam, a modern roller-compacted concrete (RCC) gravity dam, with its maximum height of 575 feet and crest length of 2,050 feet, will be the key feature of a major Colorado water supply and hydroelectric power project, the Central Colorado Project (CCP) also known as the Union Park-Aspinall Pool Project, proposed by Natural Energy Resources Company (NECO) in 1982. The dam site is located at an altitude of 10,000 feet on Lottis Creek in Union Canyon approximately 35 miles northeast of Gunnison, Colorado in the upper Taylor River drainage. When completed, Union Park Dam and Reservoir will store up to 1.2 million acre-feet of Colorado's currently undeveloped Aspinall Pool and Colorado River Compact entitlements. This water can then be efficiently delivered by gravity tunnels and pipelines to urban and rural areas east and west of the Continental Divide. The project will be operated as a pumped-storage facility generating high value peaking power using the Bureau of Reclamation's existing Taylor Park and Blue Mesa Reservoirs as lower pools. This paper presents the results of the preliminary geological and geotechnical investigations and engineering analyses thus far completed for Union Park Dam, to confirm the technical feasibility of the project.

The main objective of these investigations was to determine the location, size and type of dam required to safely and economically store up to 1.2 million acre-feet of water in Union Park Reservoir and develop recommendations for the more detailed field investigations and engineering analyses to follow. The reservoir water surface elevation of 10,120 feet and topography of Union Canyon were governing factors in locating the axis for the required 575-ft.-high dam. The excellent quality and strength of the rock in the foundation and abundant availability of construction material in close proximity to the site led to the selection of an RCC gravity dam. The final configuration of the dam was determined from finite element stress analyses performed on a number of two-dimensional models of the dam and foundation subjected to the governing seismic or extreme loading. To avoid cracking, the resulting tensile stresses in the dam body needed to be below the tensile strength of the concrete. The sliding stability of the dam was evaluated for static, hydrologic and seismic loading making use of strength and deformation properties of the rock mass determined from geological field mapping (discontinuity surveys), rock testing, and geophysical measurements (seismic refraction surveys). A simplified fault model was used to evaluate ground motion parameters (horizontal and vertical components of peak ground acceleration), acceleration response spectra, and anticipated seismic loads from a maximum credible earthquake (MCE).

A preliminary hydrologic evaluation of the drainage basin of Union Park Reservoir was completed to determine the probable maximum flood (PMF) and hydrologic loading of the dam. An opinion of the probable construction/project costs was prepared for the main 575-ft.-high RCC gravity dam and three smaller RCC (saddle) dams to be constructed at the north end of the reservoir. A core drilling/rock testing program and cost estimate was developed for implementation during the next phase of the feasibility study.

1 DESCRIPTION AND HISTORY OF PROJECT

The proposed Central Colorado Project (Figure 1) is an innovative water storage alternative that can satisfy most of Colorado's future drought and growth needs. CCP's high altitude Union Park pumped-storage site can economically save up to 1.2 million acre-feet of Colorado's undeveloped Colorado River Compact and Aspinall Pool entitlements during normal and wet cycles. These conserved spring flood waters will then be available for responsive gravity deliveries, when and where needed, throughout Colorado's five major river basins (Gunnison, Upper Colorado, South Platte, Arkansas, and Rio Grande).

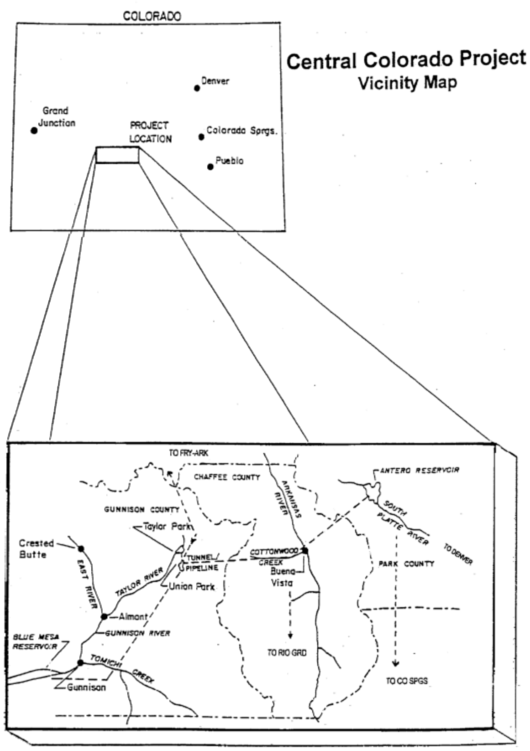


Figure 1. Major features of CCP.

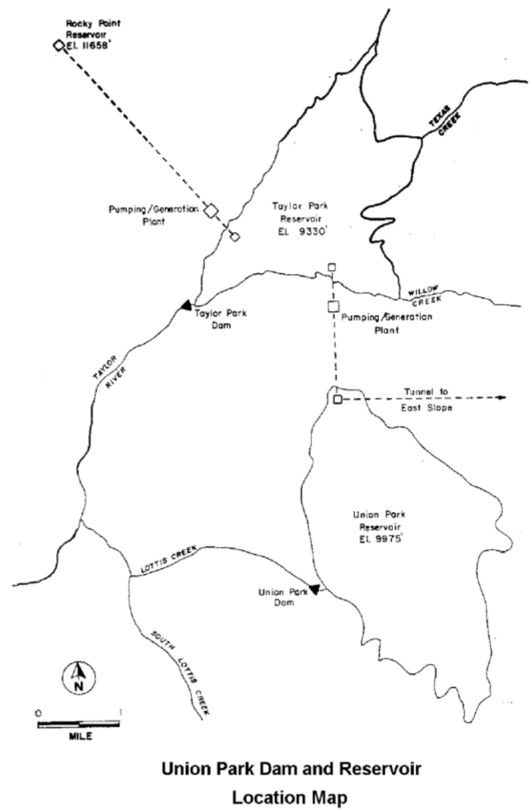


Figure 2. Original project layout.

As reported by EBASCO (1986) [1] and WRC Engineering, Inc. (1989) [2], the original project layout (Figure 2) consisted of an earth-core rock fill dam located on Lottis Creek at the east entrance to Union Canyon with crest elevation 9,989 feet, about 370 feet above stream level, impounding a total of 600,000 acre-feet of water. A RCC gravity dam would be investigated in later stages and would be the probable choice if no sources of impervious core material were identified. The original project layout also included an 11 ft.-diameter power tunnel connecting Union Park Reservoir to Taylor Park Reservoir, an underground 60 MW pumping plant with reversible units for pumped-storage operation, a spillway, an outlet works to Lottis Creek and an intake/outlet at both Taylor Park and Union Park Reservoirs. For supply to the Denver metropolitan area, the tunnel/pipeline conduit to Antero Reservoir on the South Platte River east of the Continental Divide would be about 42 miles long.

UEBLACKER ASSOCIATES (1989) [3] completed a reconnaissance level geotechnical investigation of the proposed project which included a geologic evaluation of the dam site, reservoir area and tunnel/pipe line route to Antero Reservoir, and the conceptual design and construction cost estimate for a 460-ft.-high RCC gravity dam. It was determined that the site of the proposed RCC gravity dam should be located on Lottis Creek in Union Canyon approximately 2,000 feet downstream from the site of the originally proposed rock fill dam. The 460-ft.-high dam provided storage for 900,000 acre-feet at a reservoir water surface elevation of 10,052 feet.

In 2001, the Colorado Supreme Court ruled that the waters of the U.S. Bureau of Reclamation's Aspinall Pool, which are mainly stored in Blue Mesa Reservoir, are available for development. The Aspinall Pool was authorized by Congress in 1956 to help Colorado develop 300,000 acre-feet of its unused Colorado River Compact water for statewide consumptive needs. The proposed CCP includes a 35-mile tunnel/pipeline route connecting Blue Mesa Reservoir with Union Park Reservoir and a pumping station at Blue Mesa Dam. These facilities will permit pumping the water to a higher elevation where it can be stored in Union Park Reservoir before being released to drainage basins east and west of the Continental Divide when needed. It is therefore desirable to increase the storage capacity of Union Park Reservoir to 1.2 million acre-feet (water surface elevation 10,120 feet). This can be accomplished by building a 575-ft.-high RCC gravity dam at the above referenced location on Lottis Creek in Union Canyon and three additional smaller RCC (saddle) dams at the north end of the reservoir.

It is presently proposed to construct the CCP in four phases over a period of 21 years for a total estimated cost of $2.5 billion. Phase I, a 60,000 acre-ft.-diversion, which includes the construction of Union Park Dam, the power facility at Tailor Park Reservoir, and the tunnel/pipeline conduit to Antero Reservoir on the South Platte River, would be completed in 6 years for approximately $1 billion.

In 2003 and 2004, NECO authorized UEBLACKER ASSOCIATES to proceed with the feasibility level geological and geotechnical investigation for Union Park Dam. The initial phase of this investigation has been completed (UEBLACKER ASSOCIATES, 2004a & 2004b) [4-5], and included a preliminary hydrologic evaluation of Union Park Reservoir conducted by WRC Engineering, Inc.(2004) [6]. These studies indicate that a large RCC gravity dam can be constructed in Union Canyon to safely and economically store up to 1.2 million acre-feet of water without requiring a spillway. The estimated construction cost for the dam and reservoir is only $329 per acre-foot of storage.

2 ADVANTAGES OF ROLLER-COMPACTED CONCRETE (RCC)

Since the construction of the first large RCC dam in 1980, this technique has gained worldwide acceptance within a relatively short time because of its low cost, derived in part from its rapid method of construction. Throughout the world, numerous dams over 300 feet high are presently either in operation or under construction. The highest RCC gravity dam, Miel I Dam, Colombia (Marulanda, A., et. al., 2002 [7]) with 2.29 million cubic yards of RCC is 618.5 feet tall. It was completed in 2002 in only 25 months.

2.1 Costs

Construction cost histories of RCC and Conventional Mass Concrete (CMC) dams show that the unit cost per cubic yard of RCC is considerably less than conventionally placed concrete. Approximate costs of RCC range from 25 to 50% less than conventionally placed concrete. The difference in percentage savings usually depends on complexity of placement and on total quantities of concrete placed. Savings associated with RCC are primarily due to reduced forming, placement, and compaction costs, as well as reduced construction times.

Table 1 includes a preliminary construction cost estimate for placement of 6,161,669 cubic yards of RCC in the proposed Union Park main dam and 622,986 cubic yards in the saddle dams. This estimate, which is based on U.S. Army Corps of Engineers year 2000 figures (Engineer Manual EM1110-2-2006 [8]) does not include contingencies to account for variations in prices due to possible changes in quality of fly ash, cement, and aggregate which affect RCC mix designs. Nor do these costs include any contingencies for foundation drilling and grouting. To account for these contingencies a more detailed construction cost estimate, based on core drilling, borrow source evaluation, and construction materials testing, can be prepared during later stages in the design.

2.2 Rapid Construction

Rapid construction techniques (compared to both CMC and embankment dams) and reduced material quantities (compared to embankment dams) account for major cost savings in RCC dams. Maximum placement rates of 11,000 to 12,000 cubic yards per day have recently been achieved (Steele, A. K., et. al., 2003 [9]). These production rates make dam construction in one construction season readily achievable for even large structures. When compared to embankment or CMC dams, construction time for large projects can be reduced by 1 to 2 years. Applying these RCC placement rates to Union Park Dam, construction of the main RCC gravity dam could be completed in approximately 560 days or 18 months. Other benefits from rapid construction include reduced administration costs, and earlier project benefits. Basically, RCC construction offers economic advantages in all aspects of dam construction that are related to time.

Table 1. Preliminary construction cost estimate for Union Park Dam and Reservoir

OPINION OF PROBABLE CONSTRUCTION/PROJECT COSTS **Main Roller Compacted Concrete (RCC) Gravity Dam on Lottis Creek** **and RCC Saddle Dams located South of Lakeview Campground**						***Proposed Storage Facility:*** ***Union Park Reservoir*** ***(1,200,000AF)***	
Dimensions of Dams:		Height (feet)		Base Width (feet)		Crest Length (feet)	
Main Dam (N35.5E):		575.0		612.5		2,050.0	
East Saddle Dam (N47W):		160.0		137.6		2,750.0	
North Saddle Dam (N84E):		70.0		60.2		1,650.0	
West Saddle Dam (N74E):		70.0		60.2		1,300.0	
Prepared by Horst Ueblacker, P.E., *UEBLACKER ASSOCIATES, Consulting Engineers, Geologists, Constructors, Lakewood, CO*						*1/25/2004*	
Item No.	**Description**			**Quantity**	**Unit**	**Unit Price**	**Total Cost**
1	**Reservoir**						
2	*Land Acquisition*				Acres		
3	*Access Roads*			5.00	Miles	$ 500,000.00	$ 2,500,000.00
4	*Reservoir Cleaning*			4,850.000	Acres	$ 1,500.00	$ 7,275,000.00
5	*Reclamation of Disturbed Areas*				Acres		
6	**Main RCC Gravity Dam**			6,161,668.98	CY	$ 23.32	$143,690,120.61
7	*Clearing and Grubbing*			31.87	Acres	$ 2,500.00	$ 79,675.00
8	*Stream Diversion*				LS		
9	*Dewatering*				LS		
10	*Foundation Excavation and Preparation*			1,594,217.66	CY	$ 18.00	$ 28,695,917.88
11	*Drilling Foundation Grout Holes*				FT		
12	*Cement for Foundation Grouting*				94lb/Bag		
13	*Drilling Foundation and Dam Drain Holes*				FT		
14	*Facing and Bedding Concrete*				CY		
15	*Outlet Works*				LS		$ 3,000,000.00
16	*Instrumentation*				LS		
17	**RCC Saddle Dams**			622,986.35	CY	$ 30.38	$ 18,926,325.31
18	*Clearing and Grubbing*			15.75	Acres	$ 2,500.00	$ 39,375.00
19	*Stream Diversion*				LS		
20	*Dewatering*				LS		
21	*Foundation Excavation and Preparation*			254,137.48	CY	$ 18.00	$ 4,574,474.64
22	*Drillling Foundation Grout Holes*				FT		
23	*Cement for Foundation Grouting*				94lb/Bag		
24	*Drilling Foundation and Dam Drain Holes*				FT		
25	*Facing and Bedding Concrete*				CY		
	Base Construction Subtotal (BCS)						**$208,780,888.45**
	Mobilization @3% of BCS						$ 6,263,426.65
	Subtotal BCS + Mobilization						**$215,044,315.10**
	Unscheduled Items @ 20% BCS+Mobilization						$ 43,008,863.02
	Direct Construction Subtotal (DCS)						**$258,053,178.12**
	Construction Contingencies @ 10% of DCS						$ 25,805,317.81
	Opinion of Probable Construction Costs (OPCC)						**$283,858,495.93**
	Project Administrative and Engineering Costs						
	Engineering: Design and Construction @ 15% of OPCC						$ 42,578,774.39
	Owner Engineering and Administrative @ 2% of OPCC						$ 5,677,169.92
	Legal Fees @ 2% of OPCC						$ 5,677,169.92
	Environmental Permitting, Mitigation @ 20% of OPCC						$ 56,771,699.19
	Opinion of Probable Project Costs						**$394,563,309.35**

3 GEOLOGICAL AND GEOTECHNICAL INVESTIGATION

3.1 General Remarks

Within the framework of a two-phase feasibility study, geological and geophysical fieldwork of the initial phase of this investigation was focused on the potential dam site areas at the upper reaches of Union Canyon. Surface geological mapping of accessible rock outcrops and geophysical (seismic refraction) surveys were conducted to obtain a preliminary estimate of the strength, deformation and other physical properties of the rock mass and the thickness of overburden and weathered rock. This information is needed to assess the suitability of the area for the foundation of a high dam and perform the preliminary design and stability evaluation of the proposed structure. It also forms the basis for determining the type and extent of further investigations to be conducted during the next phase of the feasibility study and during design.

3.2 Geological Setting and Geomorphologic Features

According to CTL/Thompson, Inc. (1983), Tweto, O. (1976 and 1979), and Scott, G. R. (1975) [10-13], complexly folded and faulted igneous and metamorphic rocks of Precambrian age including gneiss, granitic or granodioritic gneiss and shists are predominant in the project area of Union Park. These Precambrian rocks are unconformably overlain by Paleozoic sedimentary rocks such as conglomerates, quartzites, sandstones, dolomites or limestones which occur at the proposed powerhouse location and along some of the tunnel alignments. Later intrusions of granitic material mainly as dikes and the formation of quartz or pegmatite veins are common.

3.3 Dam Site

- General Aspects, Location: The main dam of the CCP will be located on Lottis Creek in the upper reaches of Union Canyon. The geological field investigation, which mainly consisted of fracture mapping (discontinuity surveys), covered the accessible rock outcrops of both valley flanks from the entrance of Lottis Creek into the canyon to about 2,200-ft. down the valley (Figures 3 and 4). Geophysical surveys were conducted along the axis of the proposed dam at the lowermost part of this area.
- Morphology and Surficial Deposits: The asymmetric valley in the project area has steeper slopes on the NE flanks with good direct exposure of the bedrock particularly at the entrance to the canyon. The NW flank of the valley is largely covered by talus material up to the elevation of approximately 10,000 ft. The valley is basically V-shaped. Valley-shape, missing striation of exposed rock, etc., indicate that the formation of the canyon is due to stream action rather than the outflow of ice from a Union Park glacier during the last ice age. Alluvial deposits are restricted to the actual valley floor. Their thickness is

Figure 3. Union Canyon with Lottis Creek looking northwest in downstream direction. (Dam site is located at the end of the road.)

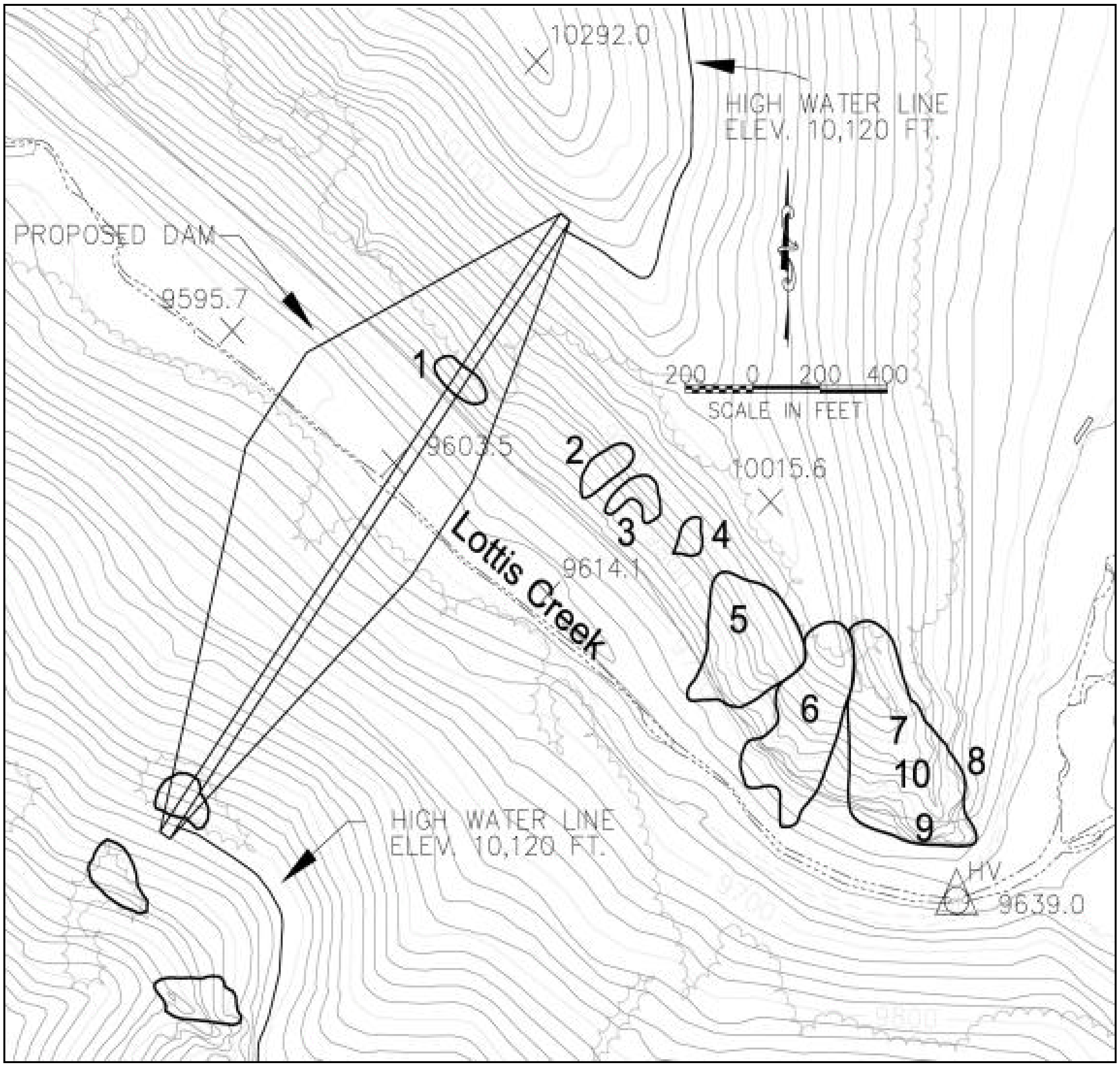


Figure 4. Accessible rock outcrop locations mapped during geologic field investigation.

estimated to be about 3 to 5 ft. (CTL /Thompson, Inc., 1983 [10]). Grain sizes range from sand to cobble. The alluvial deposits are partially covered by talus material and disturbed by former mining activities (Figure 7). Components of the talus material are angular and of cobble and block sizes in their majority. Large to very large blocks are more frequently observed in the upper part of Union Canyon at the right valley slope. Here recent rock fall has added to the talus material already in place (Figure 8). Depth of overburden was not measured directly. It appears to be shallow over most of the area. According to results from the seismic refraction survey (GEOPHYSICA, 2003 [14]), it is believed to be < 20 ft. at the lower NW section of the investigated area. Thicker accumulations are expected where fans of surficial material enter the canyon (avalanche chutes from the left, talus cones from the right), in particular in the upper reaches of Union Canyon.

3.4 Bedrock

- General Statements: A detailed description of the rock and rock mass encountered at the various outcrop areas is given in (UEBLACKER ASSOCIATES, 2004a [4]). The geotchnically significant information is also shown on the geological map (Figure 5) in summarized form. As far as applicable, the description follows the International Standard EN ISO/14689 (2001) [15].

- Rock Substance: According to the macroscopic field observation, the exposed bedrock consists mainly of gneissic granodiorite or granodiorite gneiss, derived from granodiorite by dynamo-thermal metamorphic processes. The foliation is clearly visible at most outcrop areas (Figure 6) and parallel in strike to minor fold axial planes. The main mineralogical components according to field observation are quartz, feldspar, mica, hornblende and chlorite (at some locations). The overall color is gray with thin dark layers or bands. In some areas, the rock has a slightly greenish appearance. The rock is mostly medium grained. At the outcrop the rock is slightly discolored. It shows no changes when placed in water for 24 hours and it possesses a high weathering resistance. The field examination has been checked by a petrographic study of thin sections (UEBLACKER ASSOCIATES, 2004a [4]). The gneiss or gneissic granodiorite rocks are very strong and possess a high modulus of elasticity, as shown by field and laboratory tests. It may be slightly anisotropic with regard to strength and deformation properties. Average values for the unconfined compressive strength and modulus of elasticity of intact rock are 200 MPa (29,000 psi) and 54,600 MPa (7,917,000 psi) respectively. Locally a darker colored, highly weathered rock has been observed for example at Outcrop Area 1. In the petrographic study it was identified as an altered monzodiorite. Quartz veins are frequently found as thin tabular bodies of a fraction of an inch to over one-foot in thickness. They are mostly oriented parallel to the foliation of the gneiss or gneissic host rock. Pegmatite veins or dikes have much less frequently been observed. Strength and deformation characteristics of these rock materials are equally good compared to the host rock. No weak or otherwise unfavorable rock material has been observed in the field survey.

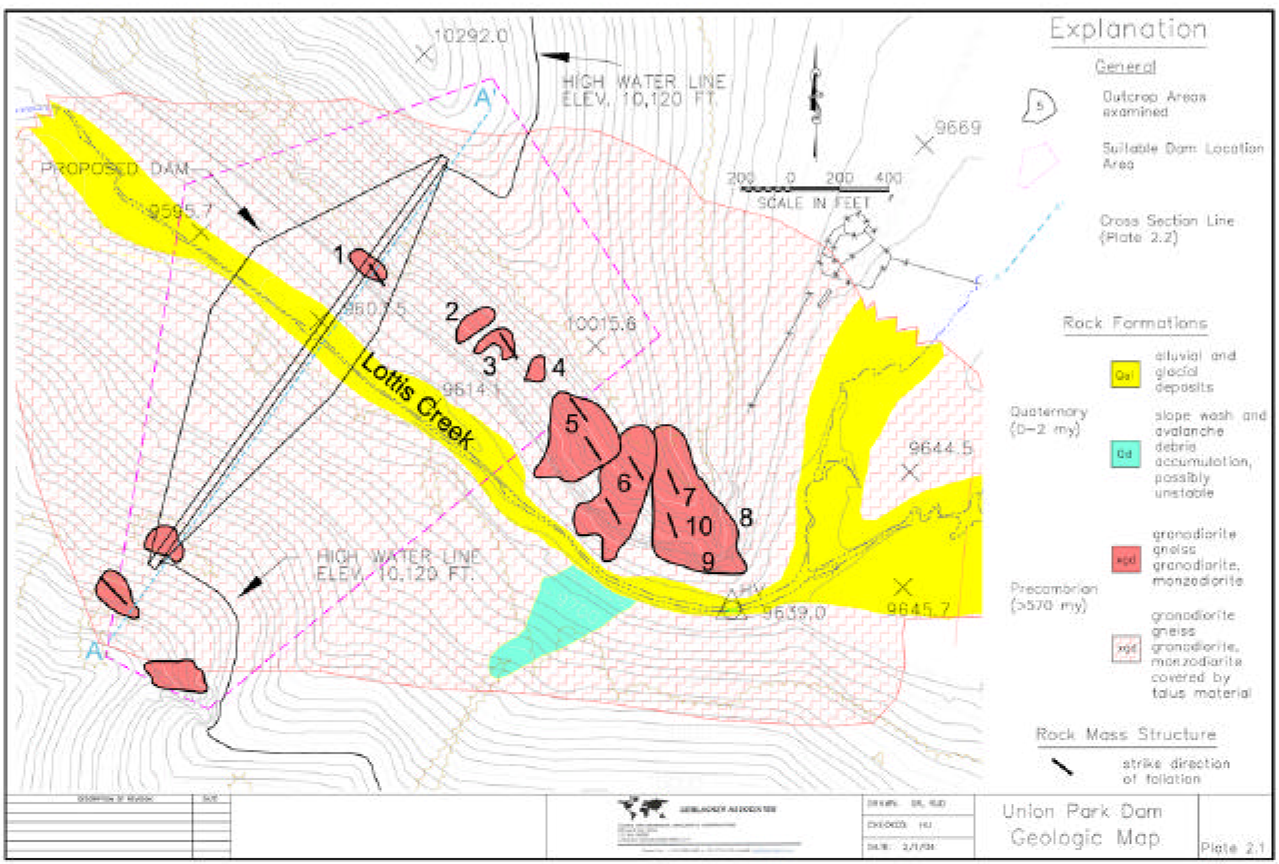


Figure 5. Geologic Map.

Figure 6. Foliation and minor fault structure (fold) in gneissic rock.

Figure 7. Upper reaches of Union Canyon downstream view;
talus material and alluvial deposits, historic mining activities.

Figure 8. Recent rock fall material, Outcrop Area 5, right valley slope.

- Rock Mass: Physical properties of the rock mass such as strength and deformation parameters or permeability may differ substantially from those of the rock substance. Such properties are strongly influenced by the type of discontinuities (joints), their orientation, spacing, persistence, aperture and filling, roughness, waviness, etc. Because of its importance in evaluating foundation and slope stability, a discontinuity survey covering the accessible exposures of the rock mass along the upper part of Union Canyon has been conducted. The results of this survey are reported in detail in UEBLACKER ASSOCIATES, 2004a [4]. Analysis of joint orientation measurements has shown that distinct joint sets can be identified at all outcrop areas. The mean orientations of the individual sets at the various outcrop locations or group of locations are presented in Table 2. Despite local variations at the various outcrop areas an overall pattern can be recognized from the summary diagram of Figure 10 combining the joint orientation measurements from all outcrop areas. According to the analysis, joint set J2 is the most prominent set. The joints of this set dip steeply in NE or SW directions and are oriented parallel in strike to the foliation of the gneissic rock and to the majority of larger quartz veins observed in the area. Joint set J1 is also prominent at all outcrop areas. At some locations it can be statistically separated into up to three subsets. The joints of set J1 dip upstream in S to SE directions at a moderate to steep angle. A third joint set, J3, about parallel in strike to joint set J2 can also be recognized. It dips in SW to WNW directions at a moderate to steep angle. Further minor joint sets are not prominent over larger areas. Rock mass characterization of outcrop areas is summarized in Table 3.

Figure 9. Joints of sets J1 and J2, Outcrop Area 9.

Table 2. Results of statistical evaluation of discontinuity survey; orientation of joint sets (dip angle / dip direction).

Area (No. of measurements)	J1 J11 J111	J2	J3 J33	J4	J5	J6
1 (325)	55/144	82/237	22/237	38/101	55/71	60/192
2 & 3 (110)	63/152 51/178 85/137	84/57	56/283 79/294			
4 (18)	58/156	87/248	58/293			
5 & 6 (123)	55/157 42/187	88/61	45/235	80/11	84/119	
7 & 8 & 9 & 10 (127)	57/170 81/156	71/67	60/252	29/325		
2 to 10 (378)	54/164	87/246	44/239			
1 to 10 (703)	54/157	82/242	23/236	36/100		

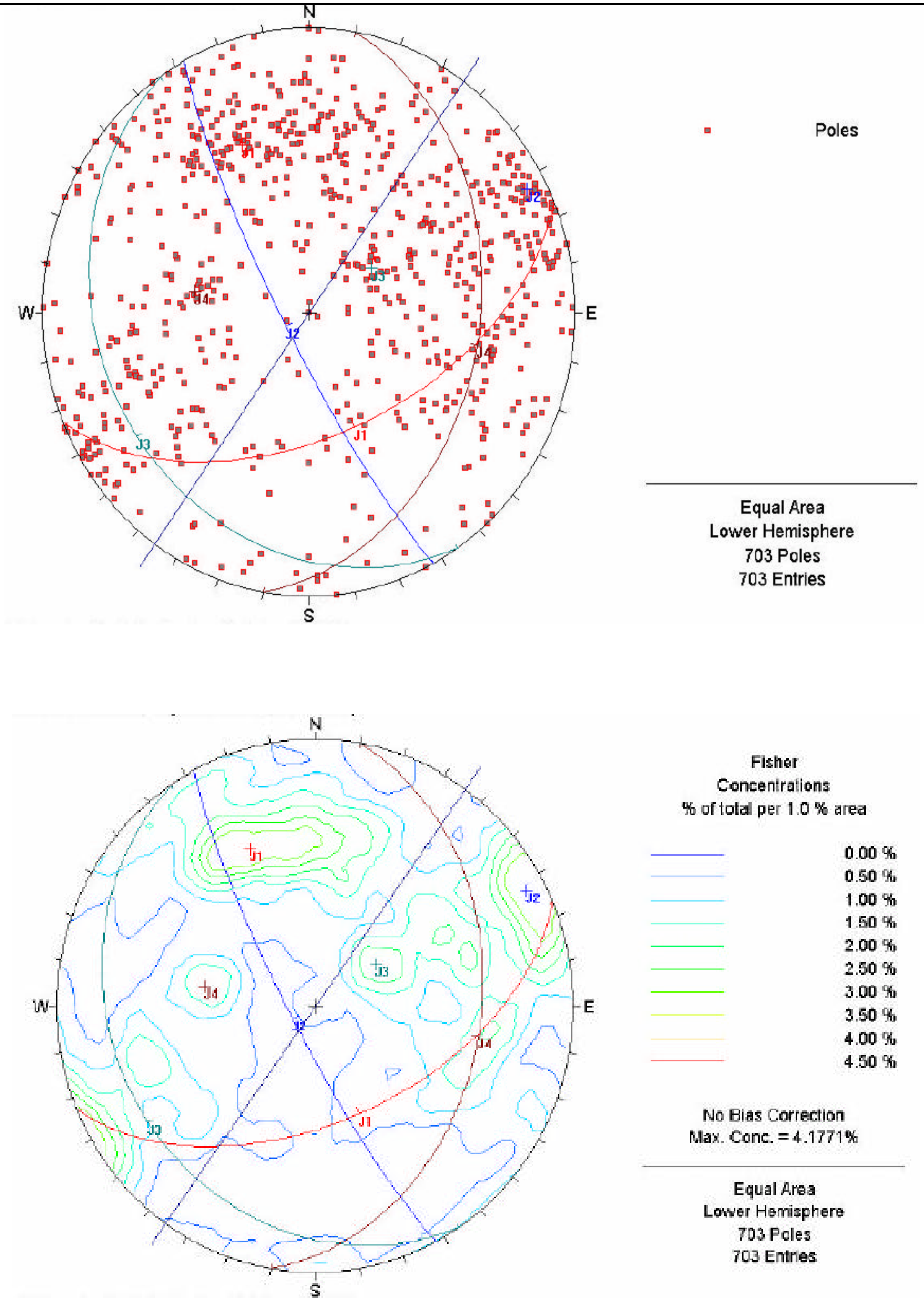


Figure 10. Union Park Dam, Outcrop Areas 1 to 10 combined. Distribution of poles to joint surfaces and pole concentration with identification of joint sets. Dam axis is oriented at N35.5E.

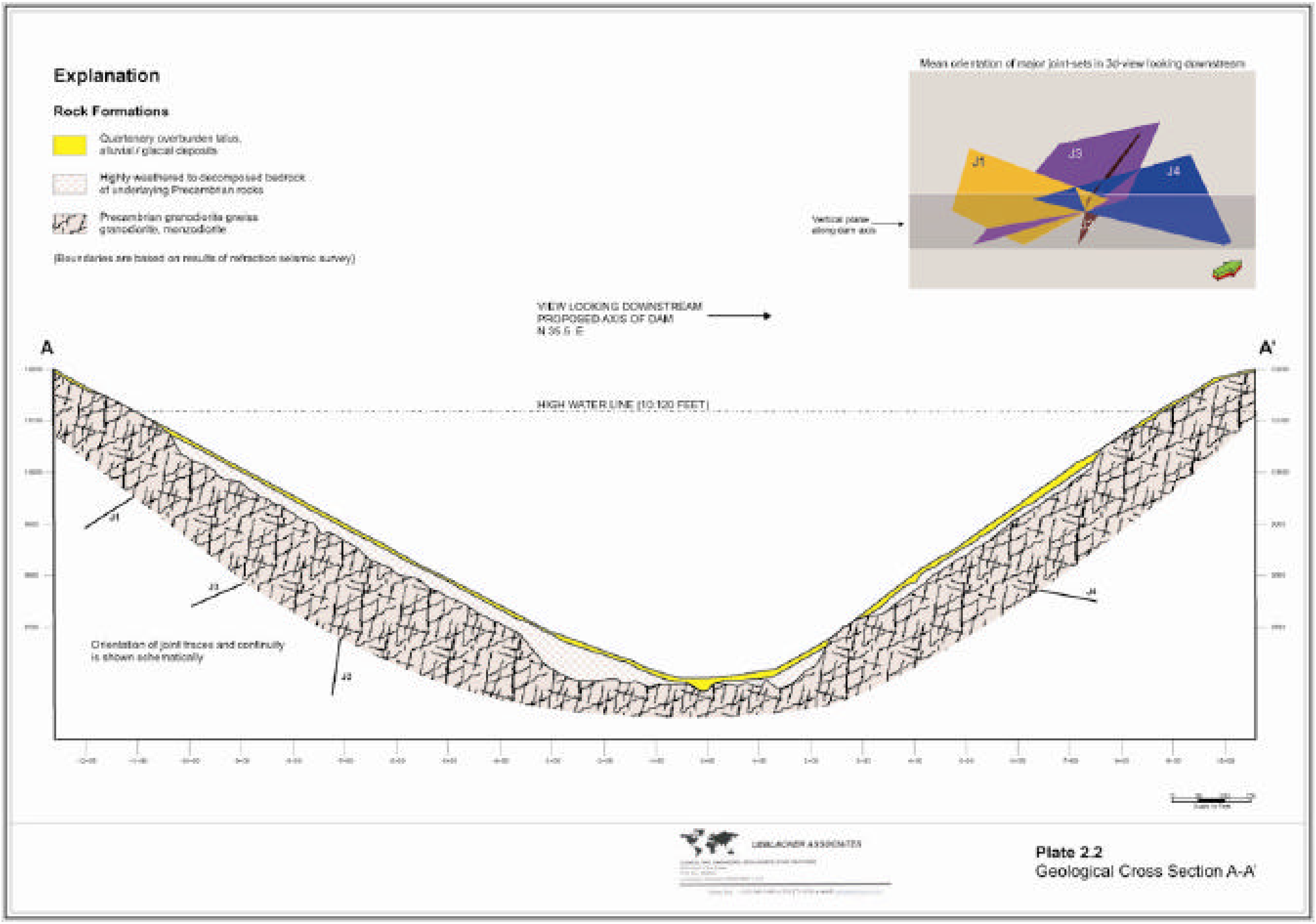


Figure 11. Geologic Cross-section along dam axis looking in downstream direction.

Obviously joint density expressed by the joint spacing varies considerably from one outcrop area to the next but also within a single outcrop area. Close to very close spacing was observed at Outcrop Area 1. In the upstream direction (Outcrop Areas 2 to 10), the average spacing as well as the extent of individual joints, increases. Large-scale joints at wide to very wide spacing are found at Outcrop Areas 5 to 10. (Figures 8 and 9). Discounting gravitational effects at steep cliffs, the joints are tightly closed to partly open. The overall blocky rock mass is generally characterized by interlocking of the joint blocks. Overall the surface conditions of the discontinuities are fair and devoid of weak fillings like clay or other soft materials. Many surfaces are relatively smooth, some are rough. Slickensided surfaces have less frequently been observed. Joint surfaces show slight to moderate weathering.

- Weathering and Surficial Loosening Effects: At most outcrop areas the rock mass is considered fresh to slightly weathered. That means that the rock substance shows little visible signs of weathering while the discontinuity surfaces are frequently discolored. Weathering has progressed somewhat deeper at parts of Outcrop Area 1 where the rock has been classified as slightly to moderately weathered according to the nomenclature given in Table 4.

Table 3. Rock mass characterization of outcrop areas.

Outcrop Area	Rock types	Weathering stage	Structural type	Surface condition of joints	Joint spacing	Joint aperture	Estimated GSI-rating (Hoek, 1994 [16])	Estimated rock mass class from RMR-rating for foundations (Bieniawski, et. al., 1976 [17])
1	Gneissic granodiorite	0 – 1	Blocky – very blocky	Fair Occasionally Poor	Medium to close	Tight, partly open	45 – 55	III
1	Altered monzo-diorites	1 – 2	Very blocky	Fair Occasionally Poor	Close to very close	Partly open to open	40 – 45	III locally IV
2 & 3	Gneissic granodiorite Granodiorite gneiss	0 – 1	Blocky, locally very blocky	Fair	Medium	Tight to partly open	48 – 62	III locally II
4	Granodiorite gneiss	0 – 1	Blocky, locally very blocky	Fair	Medium	Tight to partly open	52 – 62	III locally II
5 & 6	Granodiorite gneiss	0 – 1	Blocky, locally very blocky	Fair	Medium to wide	Tight to partly open	56 – 68	II locally III
7, 8, 9,10	Granodiorite gneiss	0 – 1	Blocky, locally very blocky	Fair	Medium to very wide, locally close to very close	Tight to partly open	56 – 66	II locally III

Table 4. Scale of weathering stages of rock mass.

Term	Description	Stage
Fresh	No visible sign of rock material weathering; perhaps slight discoloration on major discontinuity surfaces.	0
Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces.	1
Moderately weathered	Less than half of the rock material is decomposed or disintegrated. Fresh or discolored rock is present either as a continuous framework or as core stones	2
Highly weathered	More than half of the rock material is decomposed or disintegrated Fresh or discolored rock is present either as a discontinuous framework or as core stones.	3
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	4
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	5

Surficial loosening mainly caused by opening of existing joints or the development of new joints due to changes of stresses in the rock and gravitational effects can be observed. The depth of weathering and loosening effects can not directly be measured. According to field observations and the results from the seismic refraction survey (GEOPHYSICA, 2003 [14]), weathering and loosening should be relatively shallow in the right valley slope. A layer of low velocity rock has been identified in the left valley slope, the valley floor and lowest part of the right valley slope. The depth of overburden and weathered or loosened rock is also shown in the geological section (Figure 11). Based on the seismic refraction survey results, the thickness of the lower velocity rock mass reaches about 60 ft. at the toe of the left valley slope. It averages around 30 ft. in the same slope at higher elevations. Depth and nature of this low velocity rock mass have to be further explored by drilling.

3.5 Geological Hazards

In addition to foundation stability other factors like earthquake and flood hazards, slope stability, avalanche or debris flow, etc., are of importance with regard to site suitability for dam construction. According to earlier studies (CTL/Thompson, Inc., 1983 [10]), and local observations made during the recent field survey at the upper reaches of Union Canyon, no active or dormant faults have been observed at the proposed dam site and its vicinity. However, seismic risk can no longer be regarded as low. Recent studies completed by the U.S. Bureau of Reclamation for Taylor Park Dam (Hawkins F. F., & Vetter, U. R., 1998 [18]) indicate that Union Park Dam could be subjected to moderate to strong ground shaking as a result of earthquakes associated with known and suspected late-Quaternary faults in the region and random or background seismicity that can not be associated with known surface faults. Ground motion parameters for seismic loading of Union Park Dam (Figures 12 and 13) have been determined (UEBLACKER ASSOCIATES, 2004a [4]) using the attenuation relationships developed by Campbell, K. W. (1997[19]).

Avalanche and debris flow channels are clearly visible on the left valley slope. The uppermost reaches of Union Canyon are affected by this hazard. The morphology and results from earlier geophysical studies (CTL/Thompson, Inc., 1983 [10]) indicate an accumulation of potentially unstable material in the roadway embankment at the toe of the left valley slope in this part of Union Canyon. Rock fall occurs frequently in the canyon and poses a hazard to anyone working beneath the cliffs. Particularly affected are the uppermost reaches of the canyon. Deep-seated slides involving large volumes of rock material are not expected.

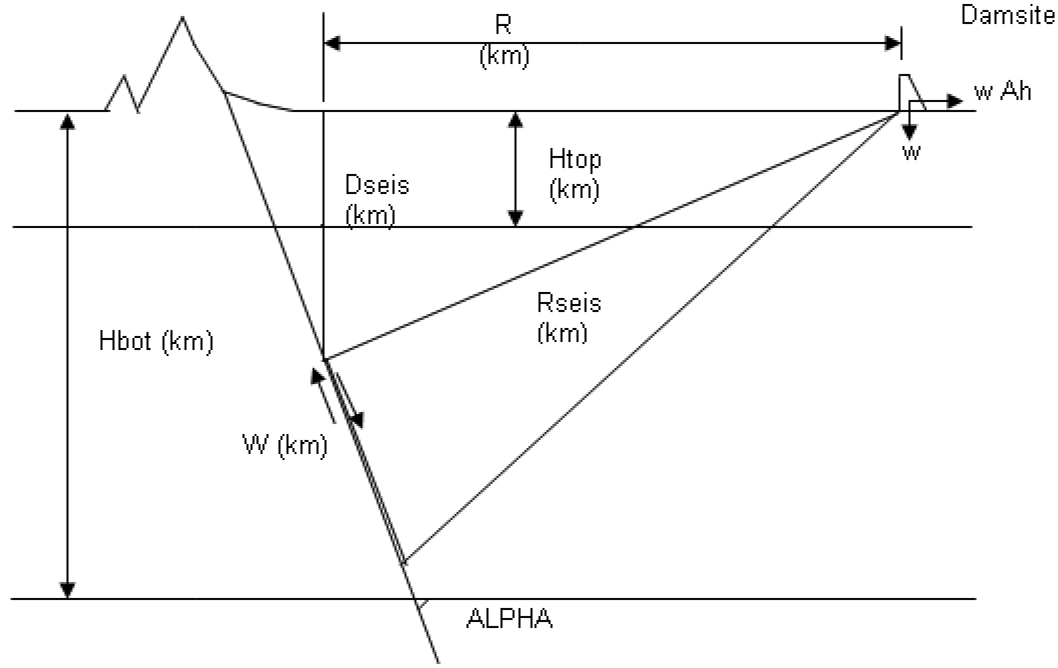
3.6 Geological Aspects of Site Suitability and Geotechnical Parameters

- Site Suitability: According to the presently available geological and geotechnical information, the area outlined in Figure 5 is well suited for the construction of a large dam allowing the storage of up to 1.2 million acre-ft. of water. Considering the morphology of Union Canyon and the availability of construction materials from nearby sources, a concrete gravity dam appears to be the most suitable type of structure. Areas considered for a dam farther upstream in Union Canyon have several disadvantages:

  (a) Lower topography (right valley slope)
  (b) Avalanche and debris flow hazard (left valley slope)
  (c) Large depth to sound bedrock (valley floor and left valley slope)

- Excavation Depth and Geotechnical Parameters: The approximate depth to the foundation level of a concrete gravity dam can be estimated from the results of the geological field mapping and geophysical (seismic refraction) surveys (Section A-A', Figure 11). For preliminary design purposes of the dam, lacking the more detailed geological and geotechnical information to be obtained from core drilling, an excavation depth to sound bedrock of 50 ft. is recommended.

  Based on current knowledge, foundation treatment at this level can most likely be restricted to curtain grouting. A grout curtain is usually required in order to limit water losses and to reduce water pressure at the base of the dam. At locations farther upstream, but within the limits outlined in Figure 5, the depth to sound bedrock for a suitable dam foundation could likely be somewhat shallower. Based on the geological conditions observed at Outcrop Area 1, the following geotechnical parameters may be assigned to the rock mass for preliminary slope stability studies (most likely and (low estimates)):



W	Down dip width of fault rupture in km
ALPHA	Fault angle in degrees
Hbot	Depth to bottom of seismogenic crust in km
Htop	Depth to top of seismogenic crust in km
Dseis	Depth to top of seismogenic fault rupture in km
R	Distance between site and vertical projection of seismogenic fault rupture in km
Rseis	Shortest distance between the recording site and the presumed zone of seismogenic rupture on the fault in km
w	Weight of dam in kN/m or Kip/ft
Ah	Horizontal component of peak ground acceleration (PGA) in g-units

Figure 12. Illustration of Simplified Fault Model used for evaluating Ground Motion Parameters.

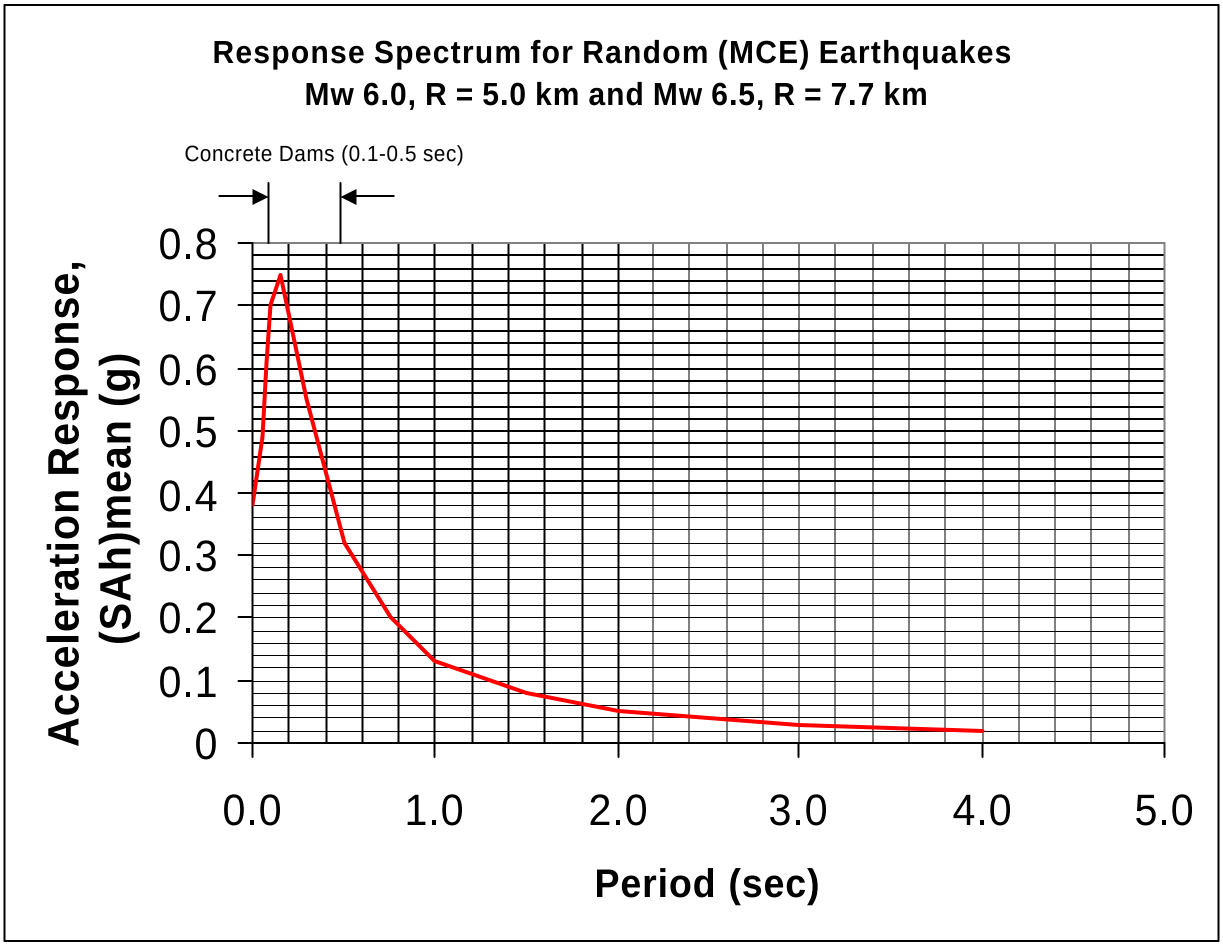


Figure 13. Acceleration Response Spectrum for Structures at the Union Park Dam Site.

Rock Mass Rating (Bieniawski, Z.T. et. al., 1976 [17]):

$RMR_{(76)}$ = 48 (40)

Geological Strength Index (Hoek, E., 1994 [16]; Cai, M., et. al., 2003 [20]):

for $RMR_{(76)}$ >18, GSI = $RMR_{(76)}$

GSI = 48 (40)

Rock Mass Shear Parameters (Mohr-Coulomb) estimated from RMR rating according to Bieniawski, Z. T. et. al. (1976) [17], also see Fecker, E. & Reik, G. (1996) [21]:

Friction Angle φ = 38° (35°)

Cohesion c = 0.2 MPa (0.16 MPa), (1 MPa = 145 psi)

Rock Mass Strength Parameters (Hoek/Brown criterion - Hoek, E., 1994 [16]):

m_b / m_i = 0.16 (0.12)
m_i = 29
s = 0.003 (0.001)
a = 0.5 (0.5)

Modulus of Deformation (Hoek, E., 1994 [16]):

E_m = 9,000 MPa (6,000 MPa)

Poisson`s Ratio (Hoek, E., 1994 [16])

υ = 0.25 (0.25)

Calculations based on P-wave velocity (Vp) measurements from the seismic refraction survey (GEOPHYSICA, 2003 [14]) indicate that the rock at foundation level will be of considerable better quality. Also, the shear strength parameters of the rock mass are highly stress dependent and must be determined considering the range of vertical or normal stresses acting on the base of the dam. Depending on the type of loading (static, hydrologic, seismic), the normal stresses acting on the base of the dam, as determined from finite element analyses, will have maximum values of between less than 500 and greater than 1,000 psi (<3.45 and >6.897 MPa).

For stability evaluation and preliminary design of the dam at the project feasibility level, the strength and deformation properties calculated for the rock mass shown together with the failure envelope (graph) in Figure 16 may be applied. However, to account for the much lower shear strength along the base of the dam a cohesion value of 70 psi (0.48 MPa) is recommended (Table 6).

All parameters will have to be re-evaluated during the second phase of the feasibility study, when the more detailed geological and geotechnical information from the core drilling and rock testing program is available.

4 STABILITY EVALUATION AND PRELIMINARY DESIGN OF MAIN RCC GRAVITY DAM

4.1 Structural Competence of Gravity Dams

The essential criteria governing the structural competence of a gravity dam (Novak, P., et. al., 1996 [22]) follow from the condition that the summation of all active and reactive, horizontal and vertical forces acting on the structure, as well as the summation of the moments of those forces, with respect to any point, must be equal to zero. Assessed in relation to all probable conditions of loading, including the reservoir empty condition, the profile must demonstrate an acceptable margin of safety with regard to:

(a) rotation and overturning
(b) translation and sliding, and
(c) over-stressing and material failure

Criteria (a) and (b) control overall structural stability. Both must be satisfied with respect to the profile above all horizontal planes within the dam and the foundation. The over-stress criterion, (c), must be satisfied for the dam concrete (tensile and compressive strength of RCC) and for the foundation (allowable bearing capacity of rock).

During feasibility level studies and for smaller structures, stability and stress analyses are usually conducted on the assumption that conditions of plane strain apply. Analysis is therefore carried out on a two-dimensional basis, considering a transverse section of the structure having unit width parallel to the longitudinal axis of the dam. Internal stresses are generally determined by the application of standard elastic theories (gravity method). More sophisticated techniques, including finite element analyses, are applied to stress determination for larger or more complex structures or to the investigation of specific problems.

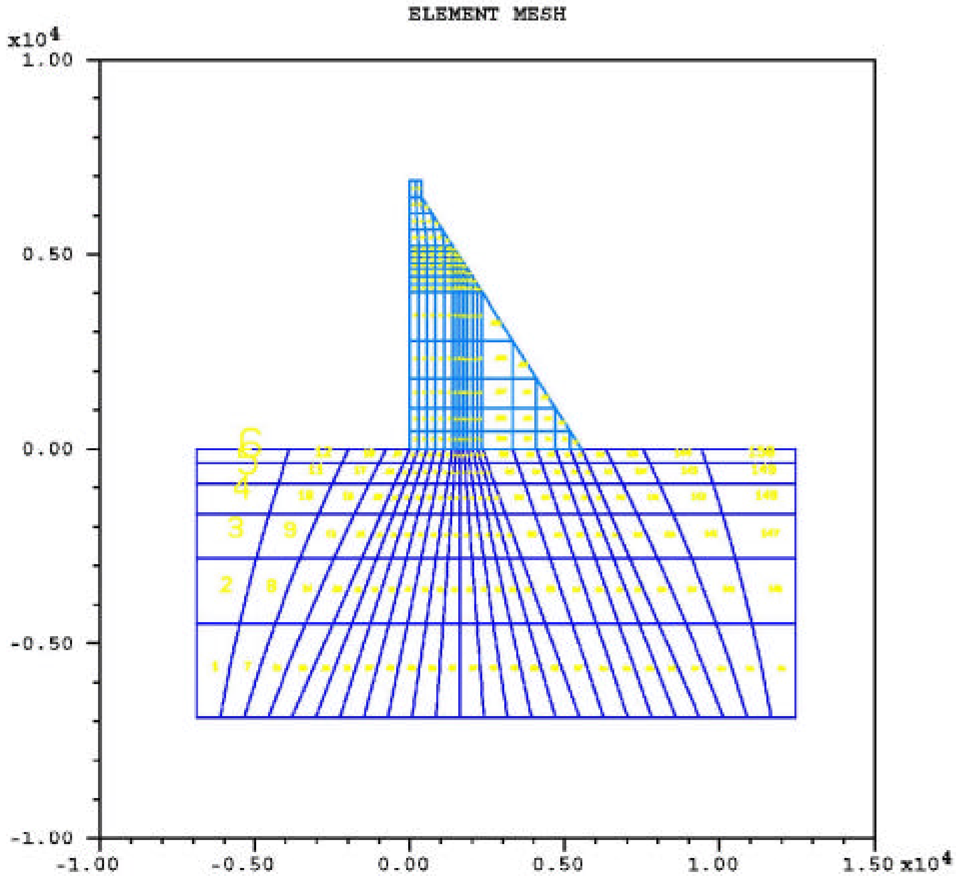


Figure 14. Finite Element Model of Original Non-overflow Section of Union Park Dam.

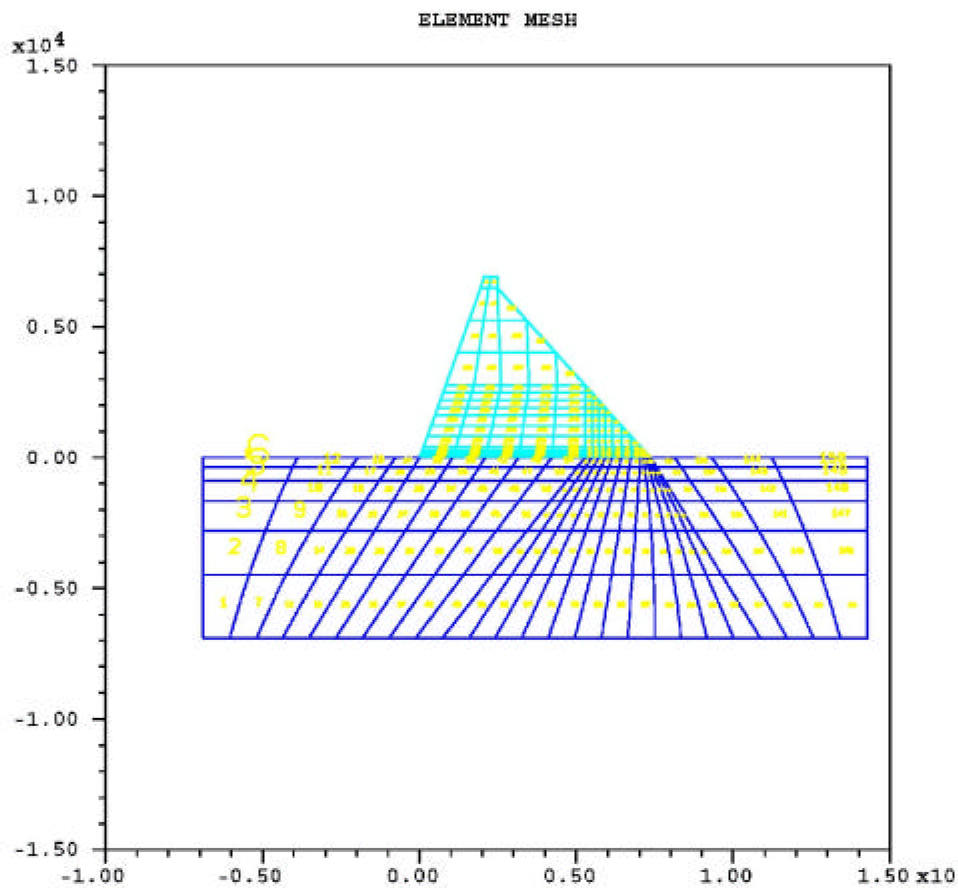


Figure 15. Finite Element Model of Modified Non-overflow Section of Union Park Dam.

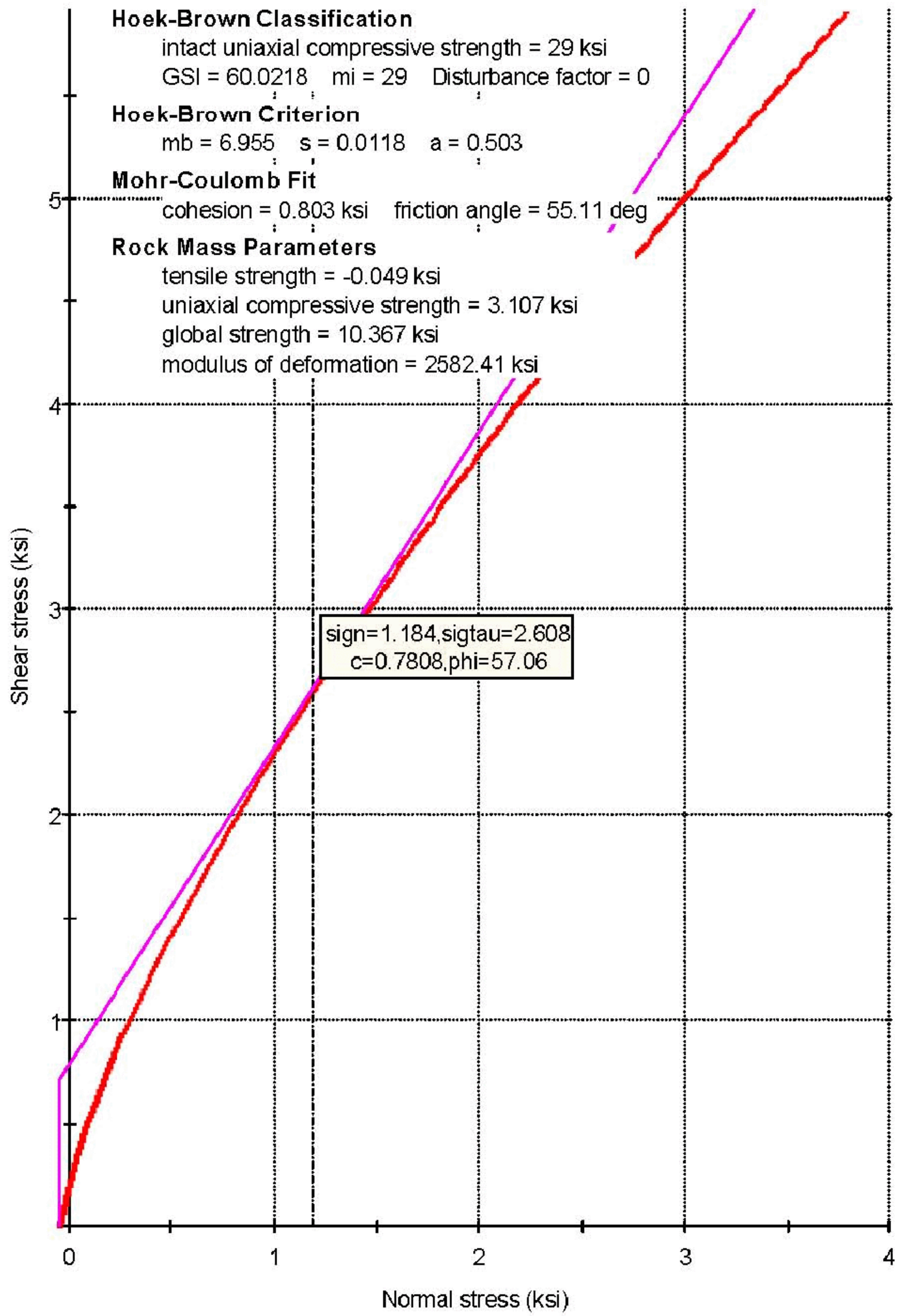


Figure 16. Analysis of Rock Mass Strength Union Park Dam Granodiorite
Left Abutment (sig3max=1.50 ksi).

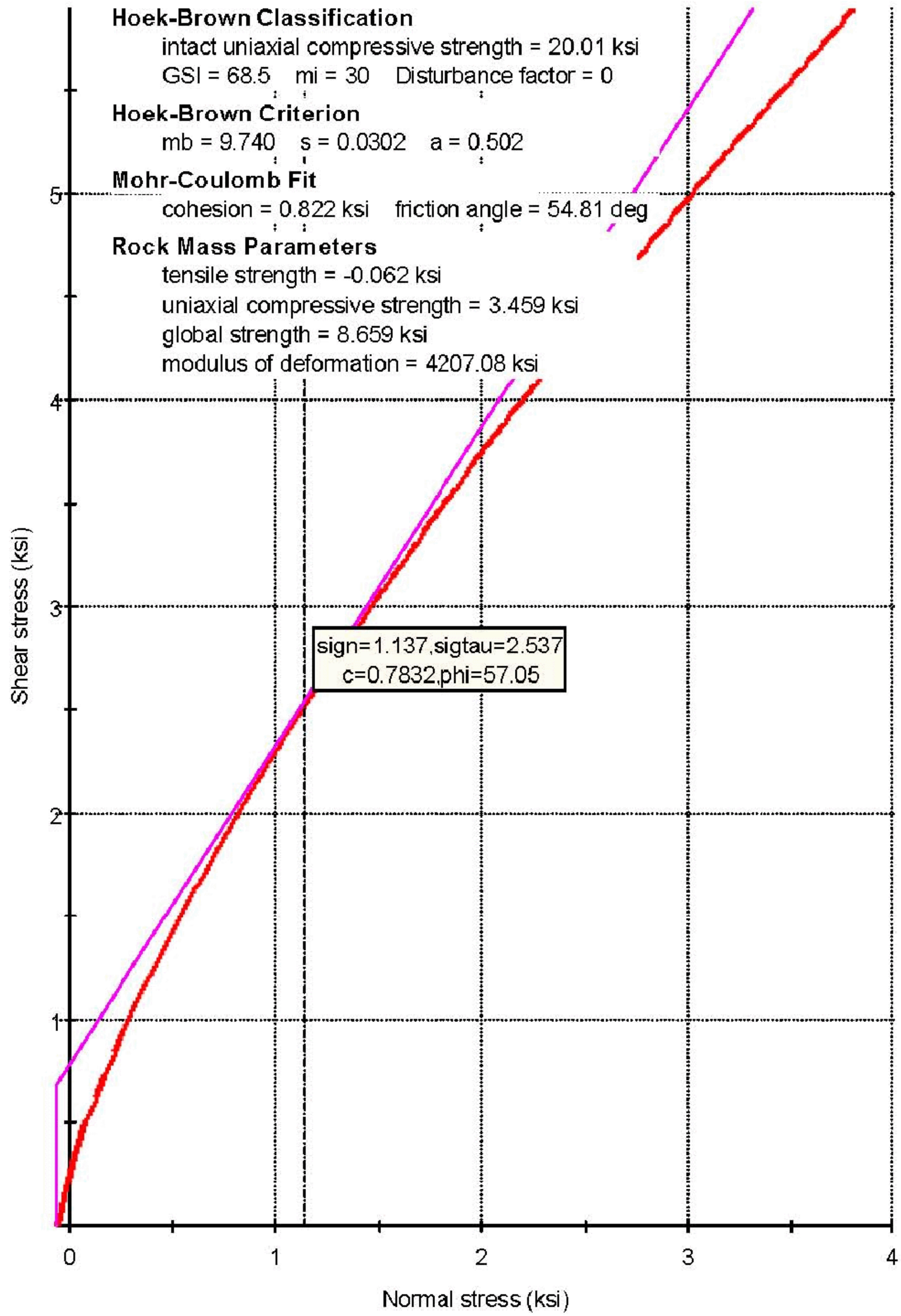


Figure 17. Analysis of Rock Mass Strength Olivenhain Dam Granodiorite
(sig3max=1.5 ksi).

4.2 Union Park Dam

To determine the internal stresses and evaluate the stability of the proposed RCC gravity dam, a number of finite element models of the dam's highest non-overflow section together with the foundation were developed. The model of the original design is illustrated in Figure 14. The strength and deformation properties calculated for the rock mass are shown in Figure 16 together with the non-linear Hoek-Brown failure envelope. The properties are based on the average P-wave velocity (Vp) measurements obtained from the shallow seismic refraction survey of the left abutment and are believed to be representative of the quality of the granodiorite bedrock at foundation level. Using the relationship between the rock mass quality Q and the shallow seismic P-wave velocity (Barton, N., 2000 [23]):

$$Vp = 3.5 + \log Q \qquad (1)$$

for Vp = 4,252 m/s and

$$Q = 10^{\{(Vp - 3500)/1000\}} \qquad (2)$$

$$Q = 5.649$$

Modulus of Deformation (Barton, N., 2000 [23]):

$$Em = 10\,(Q^{1/3}) \qquad (3)$$

$$Em = 17.81 \text{ GPa } (2{,}582.41 \text{ ksi})$$

As shown in Figure 16, the corresponding geological strength index for Union Park Dam granodiorite, calculated using the computer program RocLab (Rocscience, Inc., 2002 [24]):

$$GSI = 60.0215$$

Using the rock mass quality Q calculated with equation (2), the geological strength index GSI (Hoek, E., et. al., 1998 [25]) may also be determined with equation (4):

$$GSI = 9 \ln Q + 44 \qquad (4)$$

$$GSI = 59.583$$

For comparison purposes, the deformation and strength properties of a similar granodiorite rock mass from Olivenhain Dam, California (Keaton, J. R. et. al., 2003 [26]) are also provided (Figure 17). As can be seen both failure envelopes yield nearly identical values for the instantaneous cohesion (c) and friction (phi) of the rock mass at a normal stress level of about 1.0 ksi (6.897 MPa). This normal stress is within the range of magnitude of the compressive stresses acting on the base of the dam under seismic loading. Several computer runs were made by varying the structural configuration of the finite element model to evaluate the sliding stability and internal stresses of the dam under static (usual), hydrologic (unusual), and seismic (extreme) loading conditions. The required safety factors against sliding of the dam under these loading conditions are 3.0, 2.0, and >1.0 respectively. The types of two-dimensional finite element stress analyses performed with each model included a linear elastic static analysis, a crack static analysis, and a crack dynamic analysis (see Appendix). The finite element stress and stability analyses were performed with the computer code CG-Dams, pre-release of Version 2.2.0 (EPRI & ANATECH, 1995 [27]). The recommended minimum design strength values for the RCC used in the finite element analyses are shown in Table 5. Peak and residual shear strength values for the concrete/rock interface at the base of the dam are listed in Tables 6 and 7. The results of the calculations showed that the tensile stresses in the dam body generated by the seismic loading from a maximum credible earthquake (MCE) of magnitude Mw 6.0 at 5 km or Mw 6.5 at 7.7 km (maximum horizontal ground acceleration, Ah = 0.38g) caused cracking in the RCC of the original non-overflow design. The section required several modifications until the tensile stresses in the dam were low enough to eliminate cracking. Cracking of the contact along the base of the dam and rock mass in the foundation is permitted under seismic or extreme loading. The finite element model of the modified non-overflow structure of the dam is illustrated in Figure 15. The dimensions and loading of the modified design are illustrated in Figures A6.1 and A6.2 attached to the appended Material Properties and Analysis Summary (see Appendix).

Table 5. Minimum Design Strength of RCC (Rizzo, P. C., et. al., 2002 [28]).

Property	Minimum Design Strength At One Year
Static Compressive Strength	2,300 psi
Static Tensile Strength Parent RCC and bedded lift joints Unbedded lift joints	 239 psi 115 psi
Dynamic Direct Tensile Strength Parent RCC and bedded lift joints Unbedded lift joints	 359 psi 173 psi

Table 6. Summary of Peak Shear Strength Parameters at Concrete/Rock Contact (Dawson, R. V., et. al., 1998 [29]).

			Summary of Mohr-Coulomb Strength Parameters					
			Best Fit			Lower Bound		
Rock at Contact	No. Shear Tests	No. Tensile Tests	c (MPa)	phi (Degrees)	Tensile Strength (MPa)	c (MPa)	phi (Degrees)	Tensile Strength (MPa)
Granite- gneiss	4	6	1.30	57	0.83	0.48	57	0.31

Table 7. Summary of Mohr-Coloumb Residual Shear Srengths at Concrete/Rock Contact (Dawson, R. V., et. al., 1998 [29]).

Rock at Contact	No. of Tests	Best Fit		Lower Bound	
		Apparent c (MPa)	phi (Degrees)	Apparent c (MPa)	phi (Degrees)
Granite-gneiss	4	0.028	34	0	31

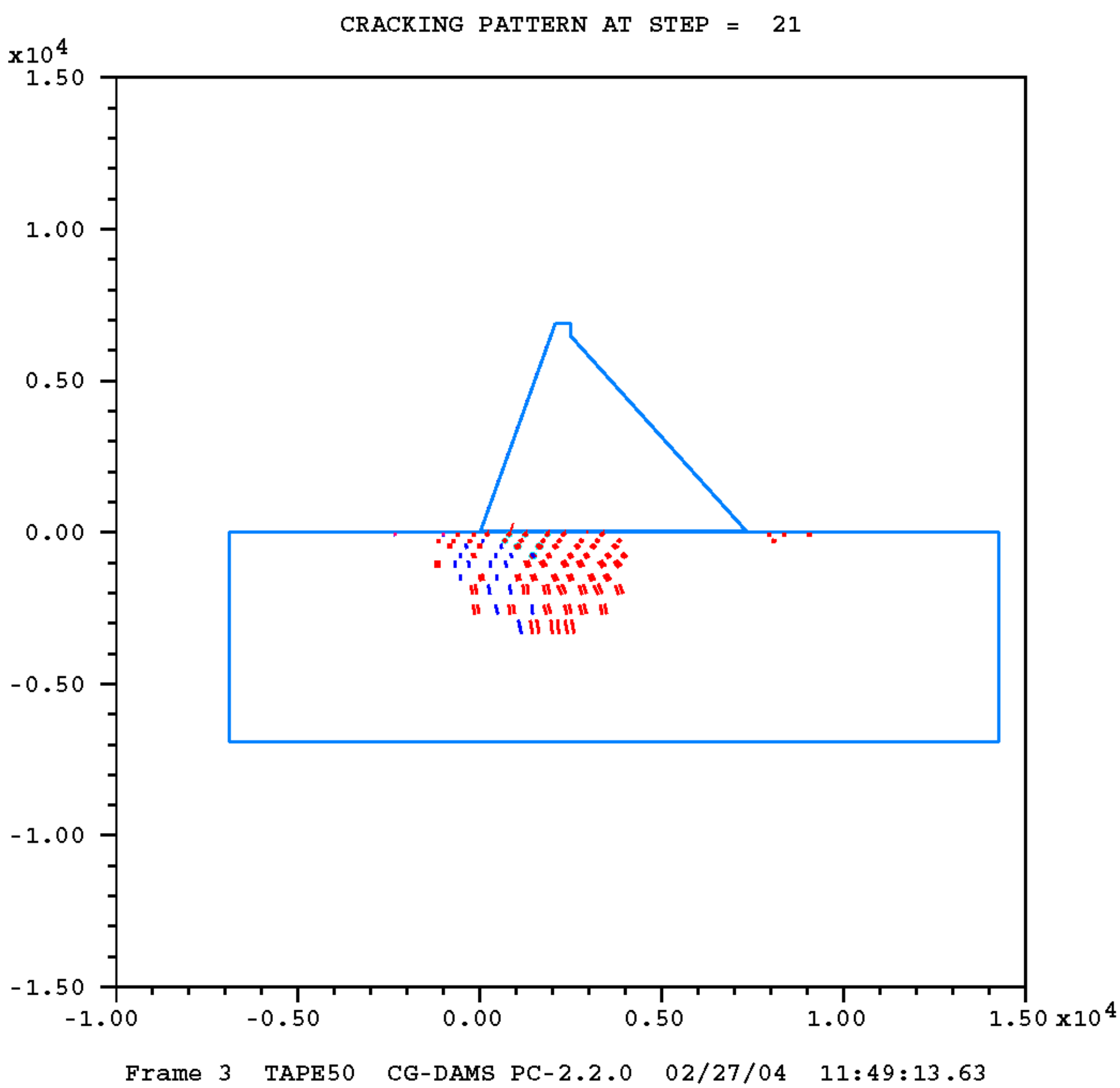


Figure 18. Results of crack dynamic finite element stress analysis (Appendix) showing the cracking pattern in the foundation and along the base of the dam (50.29% cracked base) due to high tensile stresses from seismic loading.

Figure 18 shows the results of a computer run using the geotechnical parameters listed in the appended Materials Properties and Analysis Summary (Appendix). Cracking is confined to the contact along the base of the dam and a region within the rock mass of the foundation. The results of other computer runs with this configuration may be examined in UEBLACKER ASSOCIATES, 2004a & 2004b [4,5].

Studies have been initiated during Phase 2 of the subject geological and geotechnical investigation to determine the inflow design flood for Union Park Reservoir. The stability of the 575-ft.-high RCC gravity dam was evaluated under hydrologic loading and it was determined that the dam will not need a spillway (UEBLACKER ASSOCIATES, 2004b and WRC Engineering, Inc., 2004 [6]).

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Main RCC Gravity Dam

It is hereby concluded that the geological conditions are favorable for the construction of a large dam in Union Canyon. The axis of the dam for the 1.2 million acre-ft. reservoir is more or less fixed due to topographical constraints. Additional more detailed geological and geotechnical field investigations supplemented by core drilling and rock testing are needed to verify and determine design parameters.

5.2 Reservoir Site, Saddle Dams and Access Road

The reservoir area and saddle dam sites will be examined in detail during the second phase of the feasibility study. The reservoir is regarded as an ideal storage site due to its favorable bowl-shaped morphology and the expected low permeability of the rock formation below the apparently shallow Quaternary deposits (CTL/Thompson, Inc., 1983 [10]). The saddle dams are located at the northern end of the reservoir in an area with little rock outcrop and will require seismic refraction surveys and core drilling for foundation exploration, stability evaluation and design. Preliminary dimensions and cost information on the saddle dams have been included in Table 1 of the construction/project cost estimate. Layout and design of the access road to the main dam and saddle dams will require accurate large scale topography and a terrain analysis based on detailed geological field mapping and seismic refraction surveys supplemented by test drilling.

5.3 Tunnels and Powerhouse

No new studies have been initiated with regard to the underground facilities of the proposed project during the subject feasibility investigation. Large underground openings, such as the powerhouse structure should be located in areas with favorable geological conditions. Detailed geological and geotechnical investigations, including in-situ stress measurements are therefore required, to determine the location and develop preliminary designs and construction cost estimates for the tunnels and powerhouse structure. Rock formations like the phyllites or some of the sedimentary rocks outcropping south of Taylor Park Reservoir should be avoided.

6 ACKNOWLEDGMENTS

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APPENDIX

MATERIAL PROPERTIES AND ANALYSIS SUMMARY

Union Park Dam Modified Non-Overflow Section Crack Pseudo-Dynamic Finite Element Stress Analysis UPD17.SMT
Horst Ueblacker, P.E., February 27, 2004

Elastic Modulus

Modulus of Dam (RCC) and Interface	Es	=	2,500,000 psi
Modulus of Rock Mass (Foundation)	Em	=	10.10^(Vp-3500/3000) (GPa)
Average P-Wave Velocity Left Abutment	Vp	=	4,252 m/s
	Em	=	17.81 GPa
	Em	=	2,582,410 psi

(see Figure 16: Analysis of Rock Mass Strength Union Park Dam Granodiorite, Uniaxial Compressive Strength of Intact Rock = 29 ksi (200 MPa), GSI = 60.0218);

Poisson's Ratio

Dam (RCC) and Interface	v(RCC)	=	0.20
Rock Mass (Foundation)	v(ROCK)	=	0.25

Unit Weight

Dam (RCC) and Interface	w(RCC)	=	150 lbs/ft^3
Rock Mass (Foundation)	w(ROCK)	=	168 lbs/ft^3

Tensile Strength

Parent RCC and Bedded Lift Joints	sigma(t)-RCC(CON1)dyn.	=	359.00 psi
Rock Mass (Foundation)	sigma(t)-rock mass(ROCK1)	=	49.00 psi
Concrete/Rock Interface	sigma(t)-interface(INT1)	=	44.95 psi

Tensile Fracture Strain

Assume Em/Es = 1.0, Es = Em = 2,500,000 psi (17.241 GPa),
e(t) = sigma(t)/2.50E+06

Parent RCC and Bedded Lift Joints	e(t)-RCC(CON1)	=	14.36E-05
Rock Mass (Foundation)	e(t)-rock mass(ROCK1)	=	1.96E-05
Concrete/Rock Interface	e(t)-interface(INT1)	=	1.79E-05

Assume sigma(t)-rock mass(ROCK1) = sigma(t)-interface(INT1) = 45 psi, then
e(t)-rock mass(ROCK1) = e(t)-interface(INT1) = 1.80E-05

Pseudo-Dynamic Parameters

Viscous Damping Ratio of Dam on Rigid Foundation w/empty Reservoir e1, (range e1 = 5%-10% or 0.05-0.1), e1 = 10%; Damping Factor of Foundation Rock n, n = {7/(2+e1%)}^1/2, n = 0.764; Pseudo-acceleration Sa, Sa = ah.S.n.B/q.(Tc/T)^k1 (g-units), Sa = 0.578 g, (for Ground A see Flesch, R. G., Felsbau **14**, 1996, Nr. 5, page 260-261 [29]: S = 1.0, k1 = 1.0, B = 2.5, q = 1.0, Tc<T<Td, Tc = 0.4 s, Td = 3.0 s), T = 0.754 s, T = Natural Period of Vibration of Dam with Impounded Water on Flexible Foundation (from Finite Element Analysis), Maximum Horizontal Ground Acceleration ah, ah = 0.38 g, for T > 0.5 s: Maximum Vertical Ground Acceleration av = ah/2, av = 0.19 g.

Table A6.1: Summary of Material Properties for Dynamic Analysis of Union Park Dam

	Unit Weight (lb/ft^3)	Elastic Modulus (psi)	Poisson's Ratio	Tensile Strength (psi)	Tensile Fracture Strain	Friction Angle (degrees)	Cohesion (psi)
Estimate							
Dam	150	2,500,000	0.20	359.00	14.36E-05	-	-
Foundation	168	2,582,410	0.25	49.00	1.96E-05	57	70
Interface	150	2,500,000	0.20	44.95	1.79E-05	57	70
Analysis							
Dam	150	2,500,000	0.20	359.00	14.40E-05	-	-
Foundation	168	2,500,000	0.25	45.00	1.80E-05	57	70
Interface	150	2,500,000	0.20	45.00	1.80E-05	57	70

Analysis Summary (Step 21)

Dam Geometry
- Crest elevation = 10140.00 ft
- Base elevation at heel = 9565.00 ft
- Base length = 612.50 ft

ROCK1 Material Properties
- Elastic modulus = 2.50E+06 psi
- Poisson's ratio = 0.25
- Tensile fracture strain = 1.80E-05

INT1 Material Properties
- Elastic modulus = 2.50E+06 psi
- Poisson's ratio = 0.20
- Tensile fracture strain = 1.80E-05

CON1 Material Properties
- Elastic modulus = 2.50E+06 psi
- Poisson's ratio = 0.20
- Tensile fracture strain = 1.44E-04

Water Elevations And Silt/Backfill Densities
- Reservoir surface elevation = 10120.00 ft
- Silt elevation = 9565.00 ft
- Silt horizontal density = 85.00 pcf
- Silt vertical density = 120.00 pcf
- Tailwater surface elevation = 9565.00 ft
- Backfill elevation = 9565.00 ft
- Backfill horizontal density = 85.00 pcf
- Backfill vertical density = 120.00 pcf

Uplift Data And Drain Location
- Upstream uplift pressure = 240.50 psi
- Downstream uplift pressure = 0.00 psi
- Drain elevation = 9590.00 ft
- Drain location = 50.00 ft
- Drain efficiency = 0.80

Pseudo-Dynamic Parameters (1st Mode Only)
- Wave reflection coefficient = 1.00
- Pseudo-acceleration = 0.58 g
- Max. horiz. ground acceleration = 0.38 g
- Max. vert. ground acceleration = 0.19 g

Interface Properties (Rough Crack Model Activated)

Unit cohesion	=	70.00 psi
Internal friction angle	=	57.00 deg

Crack Length

Cracked length	=	308.00 ft
Uncracked length	=	304.50 ft
% of base cracked	=	50.29

Uplift Force (First Appl. Method)

Initial uplift at start of analysis	=	-3203.85 kip/ft
Final uplift at end of analysis	=	-3062.77 kip/ft

Foundation Normal Forces

Reservoir vertical load on foundation	=	19913.40 kip/ft
Tailwater vertical load on foundation	=	0.00 kip/ft
Other vertical forces on foundation	=	0.00 kip/ft

Dam Normal Forces

Dam dead load	=	21756.76 kip/ft
Reservoir normal load (inc. silt)	=	2882.65 kip/ft
Tailwater normal load (inc. bkfl)	=	0.00 kip/ft
Other normal forces	=	0.00 kip/ft
Total normal forces	=	24639.41 kip/ft

Dam Lateral Forces

Reservoir (inc.silt) plus earthquake load	`=	19889.76 kip/ft
Tailwater lateral load (inc.bkfl)	=	0.00 kip/ft
Other lateral forces	=	0.00 kip/ft
Total lateral forces	=	19889.76 kip/ft

Shear Friction Factor of Safety

Q=(cl + (n+U)tan(phi))/v	=	1.82

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Fig. A6.1: Union Park Dam Modified Non-overflow Section Geometry

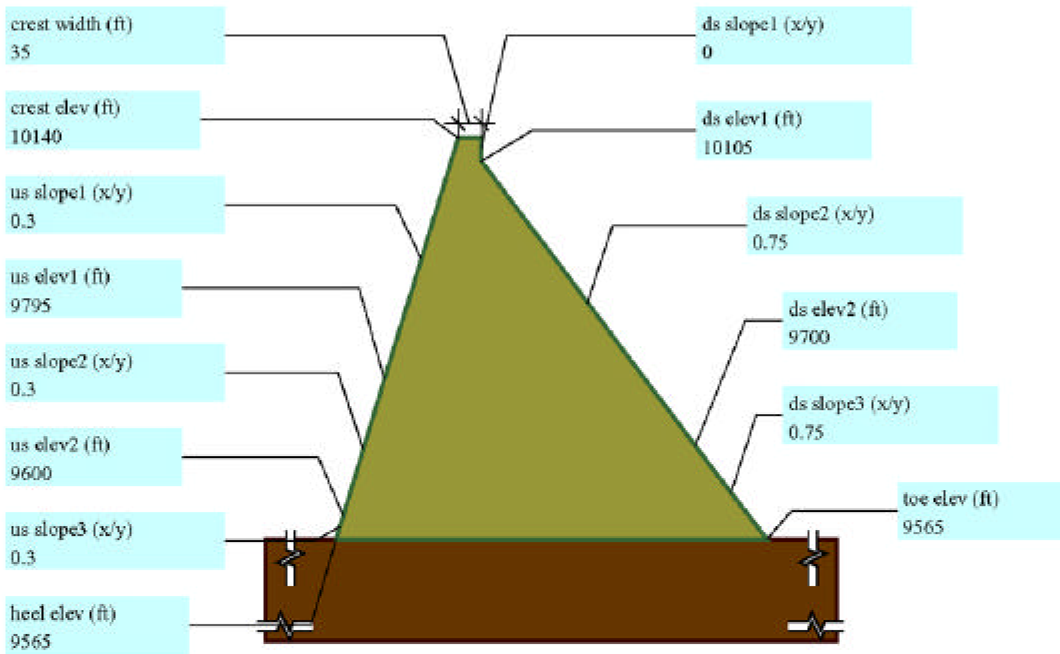

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Fig. A6.2: Union Park Dam Modified Non-overflow Section w/Hydrostatic Loading