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# Feasibility Level Geological and Geotechnical Investigation for Union Park Dam

## Preliminary Report Phase 1

By

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Prepared for

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**Overview:** Union Park Dam will be the key feature of a statewide water supply and peaking power capability called: Union Park Pumped-Storage Project. When completed, Union Park Dam and Reservoir will store up to 1,200,000 acre-ft. of Colorado's currently unused snow melt during heavy runoff years. These undeveloped Aspinall Pool and Colorado River entitlements can then be efficiently delivered by gravity tunnels and pipelines to west and east slope urban and rural areas, when and where needed, for growth, droughts, and other needs, including recovery of endangered species. Union Park will use the Bureau of Reclamation's existing Taylor Park and Blue Mesa Reservoirs as lower pools for both filling and reversible pumped-storage peaking power generation. It will be filled during heavy spring runoff months, when surplus water and hydropower are available for low cost pumping into high altitude storage. These conserved Colorado headwaters will then be available for multi-year droughts, when and where needed, throughout the state. An objective benefit-cost evaluation would quickly reveal that Union Park's peaking power revenues would cover most of its projected construction and operating costs. Union Park's benefits would also solve most of Colorado's escalating water quantity, quality, and drought protection problems throughout the Gunnison, Colorado, South Platte, Arkansas, and Rio Grande River Basins. With Union Park, the untapped Upper Gunnison Basin of origin would have assured optimal river flows during extreme drought and flood conditions.

**Background:** The Natural Energy Resources Company (NECO) obtained a conditional pumped-storage decree (82CW340) in 1984 to generate high value peaking power between the Bureau's existing Taylor Park Reservoir and a proposed 325,000 acre-ft. Union Park Reservoir. This decree also requires that Union Park's high altitude storage is used to maintain optimal minimum and maximum Taylor River flows for fish, recreation, and irrigation during extreme drought and flood conditions. During 1986, NECO recommended an enlarged 900,000 acre-ft. Union Park Reservoir to the Colorado Water Resources and Power Development Authority (CWRPDA), as a multi-purpose power and water project for Colorado's Gunnison, South Platte, and Arkansas River Basins. As a result, CWRPDA initiated a major study to evaluate Union Park and three other Gunnison trans-mountain alternatives. This half-completed CWRPDA and USBUREC study was abruptly cancelled in 1990 for unexplained reasons. During 1988, NECO sold its Union Park Decree and Multi-Purpose Project to Arapahoe County. After twelve years of litigation, Colorado's Supreme Court ruled in 98SA327 that Arapahoe's Union Park trans-mountain water right application duplicated the Bureau's 300,000 acre-ft. Aspinall Marketable Pool. Congress had authorized this overlooked Aspinall Pool water right in 1957 to help Colorado develop and beneficially use its substantial Colorado River Compact losses for statewide needs. In 2001, NECO's original pumped-storage decree and the entire Union Park multi-purpose project reverted back to NECO, per terms of its 1988 Sales Agreement with Arapahoe County. Since then, NECO has refined and enlarged its Union Park concept, to include: Colorado's unused Aspinall Pool water rights; the Blue Mesa—Union Park Pumped-Storage Phase; the Union Park—Fry-Ark Interconnect Phase; and the Union Park to Rio Grande Basin Phase.

**Overall Study Conclusion:** The following Union Park Dam Study concludes that geological conditions are favorable for construction of a large roller-compacted concrete (RCC) Dam in Union Canyon. This modern, strategically located dam and reservoir can safely store up to 1,200,000 acre-ft. of high quality, multi-year drought protection for Colorado's five major river basins. The dam's total estimated construction cost is $394,563,000. With its off-setting peaking power revenues, and dam costs of only $329 per acre-foot, Union Park Dam may become the world's most cost-effective water storage facility.

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# 1. Introduction

Union Park Dam, a modern roller compacted (RCC) gravity dam, with its maximum height of 575 ft. and crest length of 2,050 ft., will be the key feature of a major Colorado water storage and hydroelectric power project, the Union Park Project [1], [2]. The dam site (Plates 1.1 and 2.1) is located at an altitude of 10,000 ft. 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 be able store up to 1,200,000 acre-ft. of high quality water from spring snow melt which can be delivered by tunnels and pipelines to east and west slope farms and communities. The project will be operated as a pumped storage facility using the Bureau of Reclamation's existing Taylor Park and Blue Mesa Reservoir as lower pools. Union Park Dam and Reservoir will also serve as an emergency storage project for drought protection and to safeguard against the potential loss of existing water supplies in the region in the event of a possible dam failure caused by a maximum flood or earthquake.

## 1.1. General

During 2003 Ueblacker Associates entered into a contract with Natural Energy Resources Company (NECO) to perform the field work and prepare a report for the Phase 1 - Feasibility Level Geological and Geotechnical Investigation of Union Park Dam. The services to be provided under this agreement included a limited scope of work that specifically consisted of photogrammetric mapping, geology with main emphasis on fracture mapping, seismic refraction surveys, and geologic data analysis and interpretation. It did not include the stability evaluation and development of conceptual designs and construction cost estimates for the proposed RCC gravity dam prepared by Ueblacker Associates which are presented in Section 3 and 4, and in Appendix A6 of this report. Ueblacker Associates also added a detailed petrographic examination of bedrock samples (Appendix A1) and laboratory testing to determine the strength and deformation properties of intact rock (Appendix A2). Prof. Gerhard Reik, Ph.D., and M.Sc., Dipl.-Geol. Christian Weiler, Technical University Clausthal, Germany conducted the petrographic examination and laboratory tests. The fieldwork for this initial feasibility level investigation was conducted during the month of September 2003 under a special use permit issued by James R. Dawson, District Ranger, USDA, Forest Service, Gunnison, Colorado.

## 1.2. Objective

The main objective in conducting this study was to determine the location, size and type of dam required to safely and economically store up to 1,200,000 acre-ft. of water in Union Park Reservoir and develop recommendations for the more detailed geological and geotechnical investigations, and engineering analyses to follow.

## 1.3. Advantages of roller- compacted concrete

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 100 m high are presently either in operation or under construction. The highest RCC gravity dam, Miel I Dam, Colombia [21] with 1.75 million cubic meters (2.29 million cubic yards) of roller compacted concrete is 188 m (618.5 ft.) tall. It was completed in 2002 in only 25 months.

### 1.3.1. Costs

Construction cost histories of RCC and Conventional Mass Concrete (CMC) dams show that the unit cost per cubic meter 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 4.1 (Section 4) 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 2000 figures [22] 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 at the end of Phase 2.

### 1.3.2 Rapid Construction

Rapid construction techniques (compared to both concrete 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 [23]. These production rates make dam construction in one construction season readily achievable for even large structures. When compared to embankment or conventional concrete 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.

*1.3.2.1 Spillways and appurtenant structures*

Spillways for RCC dams can be directly incorporated into the structure. A typical layout allows discharging flows over the dam crest and down the downstream face. In contrast, the spillway for an embankment dam is normally constructed in an abutment at one end of the dam or in a nearby natural saddle. Generally the embankment dam spillway is more costly.

Incorporating a stepped spillway into the proposed Union Park RCC gravity dam will increase energy dissipation and help reduce the size and cost of the downstream stilling basin. With a roller compacted concrete dam, it is usual to shape the steps with a protective layer of medium to high-resistance concrete and to incorporate into the design a drainage system behind this layer.

Because of the potential risk of undermining the foundation of the dam, adequate protection against scour must be provided. The required apron below the downstream face of the dam can for this purpose be constructed with RCC. The RCC should be covered with a protective layer of high resistance concrete.

*1.3.2.2 Diversion and cofferdam*

Although not a concern for Union Park Dam, RCC gravity dams provide cost advantages in river diversion during construction and reduce damages and risks associated with cofferdam over-topping. The diversion conduit will be shorter compared to embankment dams. With a shorter construction period the probability of high water is lower, and thus the size of the diversion conduit and cofferdam height can be reduced from that required for both embankment and conventional concrete dams. Therefore, a dam may need to be designed only for a seasonal peak flow rather than annual peak flows. With the high erosion resistance of RCC, the potential for a major failure would be minimal and the resulting damage would be less, even if over-topping of the cofferdam did occur.

### 1.3.3 Other advantages

As compared to embankment dams, the smaller volume of an RCC dam makes the construction material source less of a driving factor in the site selection of a dam. Furthermore, the borrow source will be considerably smaller and more environmentally acceptable. The RCC dam is also inherently safer against internal erosion, overtopping, and seismic ground motions.

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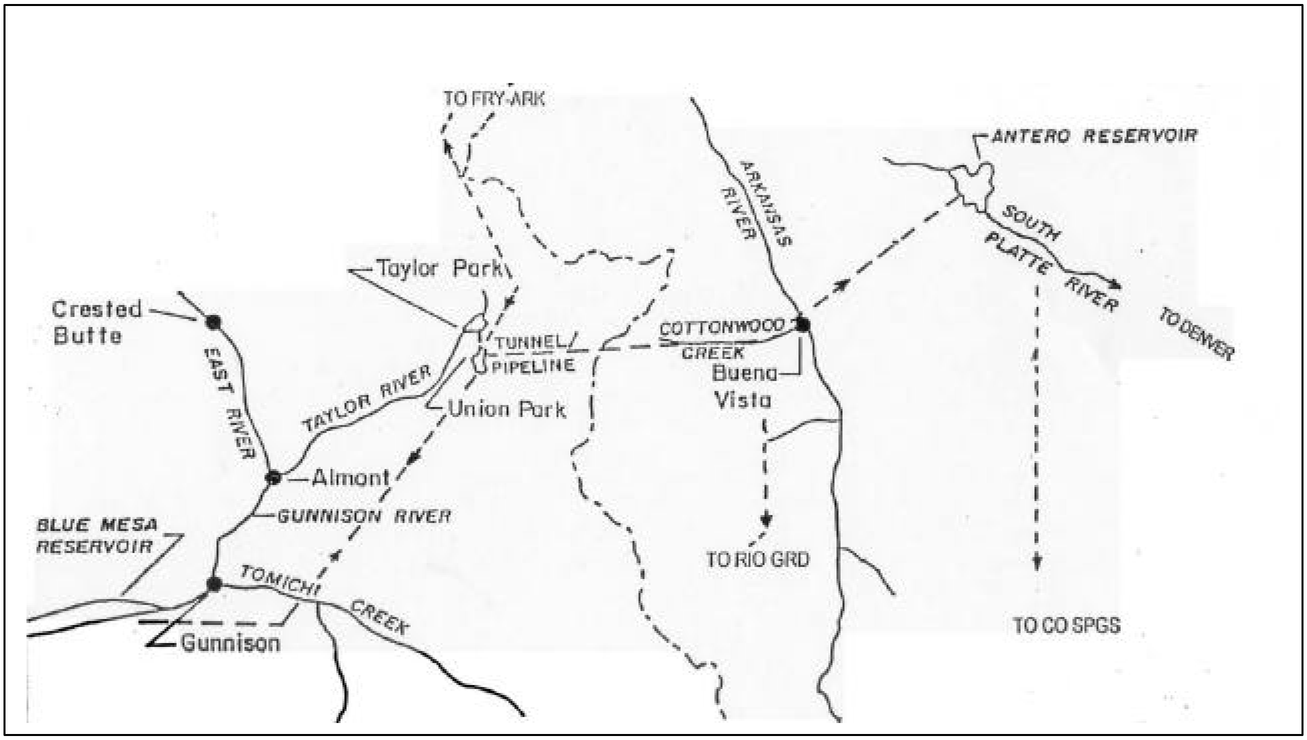



**Plate 1.1** Major Features and Phases of Union Park Pumped –Storage Project; Phase I: Union Park Dam; Union Park-Taylor Park Reversible Pumped-Storage Facility; Union –Antero Siphon/Conductor; Phase II: Blue Mesa-Union Park Reversible Pumped Storage Facility; Phase III: Union Park–Fry-Ark Reversible High Altitude Collection and Storage Interconnect Facility; Phase IV: Poncha Pass -Rio Grande Basin Branch of Union-Antero Siphon.

# 2. Geological and Geotechnical Investigation

## 2.1. General remarks

Within the framework of a two-phase feasibility study geological and geophysical fieldwork of Phase 1 was concentrating 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 to perform the necessary stability analysis and conceptual design.

It also forms the basis for decisions on the type and extent of further investigations to be conducted during Phase 2 of this investigation.

## 2.2. Geological setting and geomorphologic features

According to [2], [4], and [12] 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 intrusion of granitic material mainly as dikes and the formation of quartz or pegmatite veins are common.

## 2.3. Dam site

### 2.3.1 General aspects, location

The main dam of the Union Park Project will be located on Lottis Creek in the upper reaches of Union Canyon. The geological field investigation 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. Geophysical surveys were conducted along the axis of the proposed dam at the lowermost part of this area.

### 2.3.2 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 estimated to be about 3 to 5-ft. [2]. Grain sizes range from sand to cobble. The alluvial deposits are partially covered by talus material and disturbed by former mining activities (Photo 2.1, Appendix A4).

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 (Photos 2.2 & 2.3, Appendix A4).

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 (Appendix B), 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.

### 2.3.3 Bedrock

*2.3.3.1 General statements*

A detailed description of the rock and rock mass encountered at the various outcrop areas is given in Appendix A1, A2 and A3. The geotchnically significant information is also shown on the geological map (Plate 2.1) in summarized form. As far as applicable the description follows the International Standard ISO/DIS 14689,2 (draft; 2001) [7].

*2.3.3.2 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 (Photo 2.4, Appendix A4) 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 (Appendix A1).

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. Locally a darker colored, highly weathered rock has been observed for example at Outcrop Area 1. In the petrographic study (Appendix A1) 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.

### *2.3.3.3 Rock mass*

Physical properties of the rock mass like strength and deformation parameters or permeability may differ very much from those of the rock substance. Such properties are strongly influenced by the type of discontinuities (joints), their orientations, spacing, persistence, aperture and filling, roughness 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 Appendix A3. 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 2.1. Despite local variations at the various outcrop areas an overall pattern can be recognized from the summary diagram of Fig. 2.1 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.

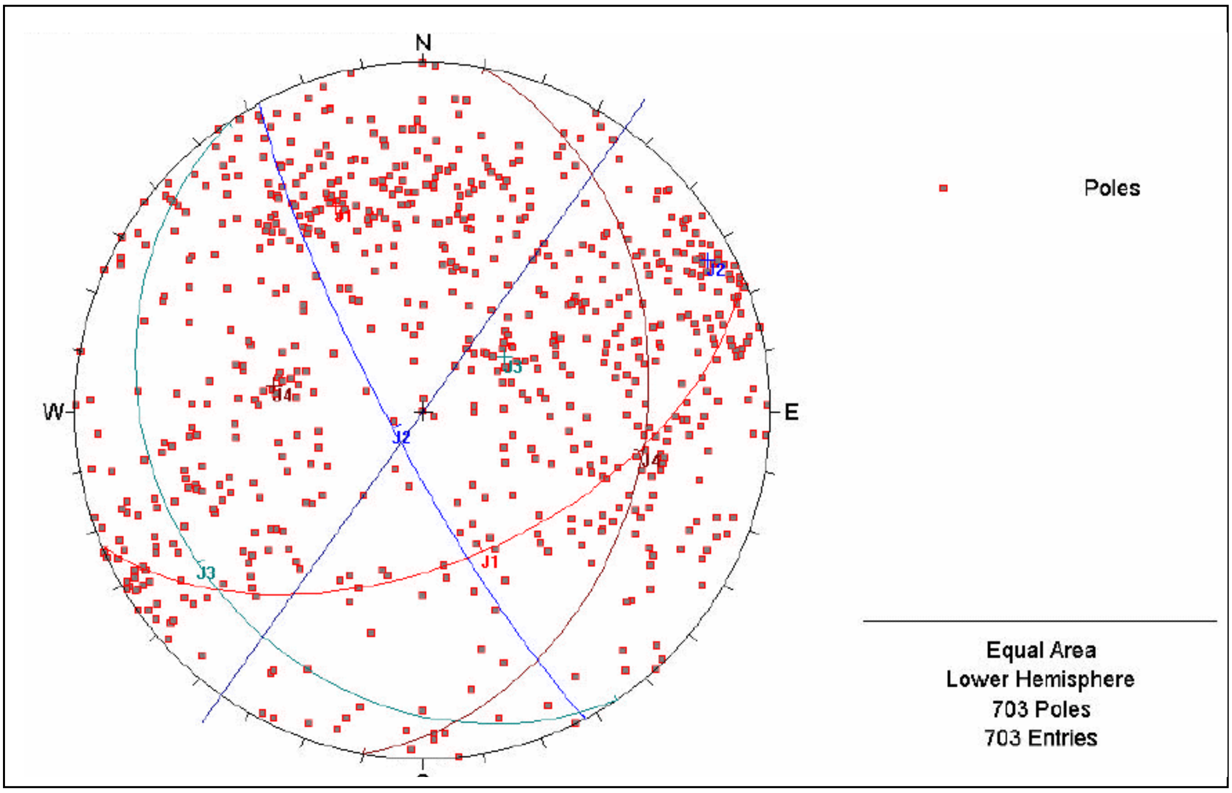
A third joint set, J3, about parallel in strike to joint set J2 can also be recognized. It dips in SW’ to WNW’ directions at a moderate to steep angle.

Further minor joint sets are not prominent over larger areas.

Rock mass characterization of outcrop areas is summarized in Table A3.7 in Appendix A3 and in Table 2.2.

Table 2.1: 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<br>51/178<br>85/137 | 84/57 | 56/283<br>79/294 | | | |
| 4 (18) | 58/156 | 87/248 | 58/293 | | | |
| 5 & 6 (123) | 55/157<br>42/187 | 88/61 | 45/235 | 80/11 | 84/119 | |
| 7 & 8 & 9 & 10 (127) | 57/170<br>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 | | |



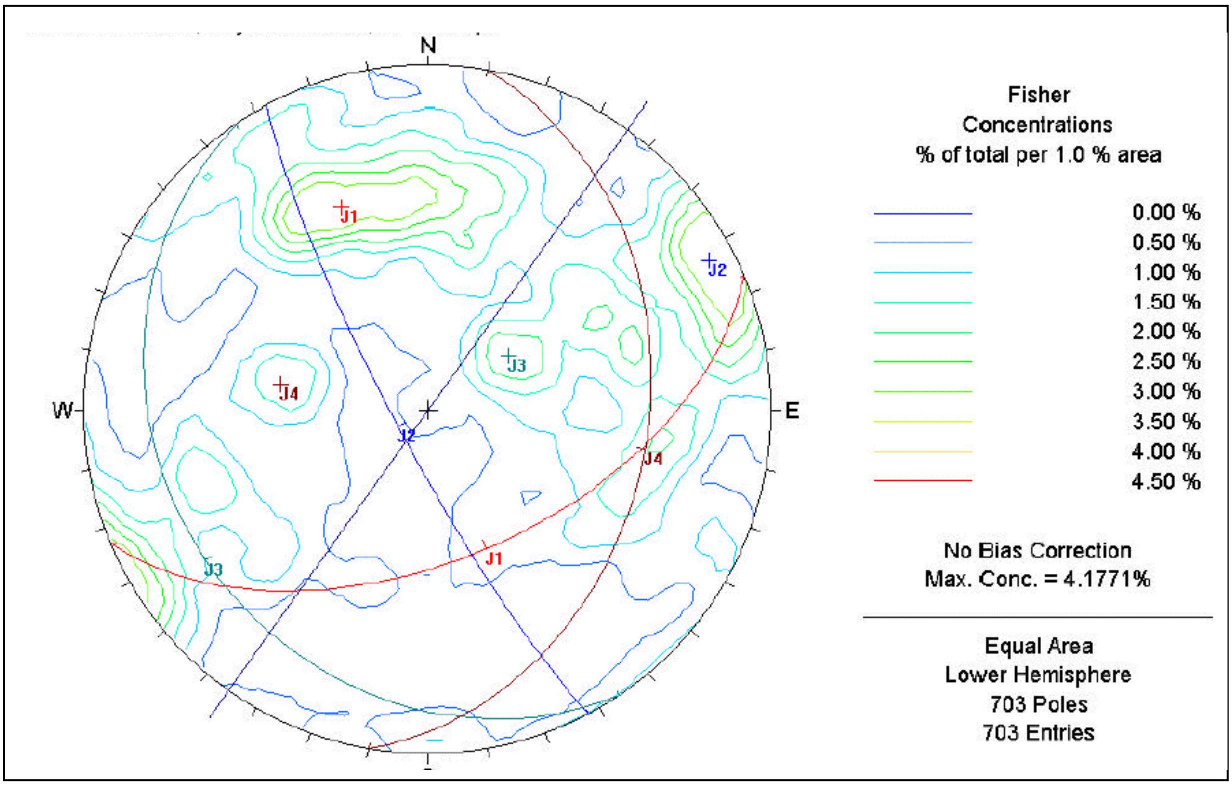




**Fig. 2.1:** Union Park Dam, Outcrop Areas 1 to 10 combined; Distribution of poles to joint surfaces and pole concentration with identification of joint sets. Dam axis is oriented at N35.5E.

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 (Photos 2.5 & 2.6, Appendix A4). 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. (Photos 2.7 & 2.8, Appendix A4).

Discounting gravitational effects at steep cliffs (Photo 2.9, Appendix A4), 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.

Table 2.2: Rock mass characterization of outcrop areas

| Outcrop Area | Rock types | Weather -ing stage | Structural type | Surface condition of joints | Joint spacing | Joint aperture | Estimated GSI-rating (Hoek, 1994) | Estimated RMR class foundations (Bieniawsky 1974) |
|---|---|---|---|---|---|---|---|---|
| 1 | gneissic granodiorite | 0 – 1 | blocky – very blocky | Fair<br>Occasion -ally. | Medium to close | tight, partly open | 45 – 55 | III |
| | altered monzo-diorites | 1 – 2 | very blocky | Poor | Close to very close | partly open to open | 40 – 45 | III. Locally IV |
| 2 & 3 | gneissic granodiorite<br>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 |

*2.3.3.4 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 further at parts of Outcrop Area1 where the rock has been classified as slightly to moderately weathered according to the nomenclature given in Table A3.2 of Appendix A3.

Surficial loosening mainly caused by opening of existing joints or the development of new joints due to changes of stresses in 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, 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 (Appendix B1). The depth of overburden and weathered or loosened rock is also shown in the geological section (Plate 2.2). 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.

*2.3.3.5 Geological hazards*

In addition to foundation stability other factors like seismic risk, slope stability, avalanche or debris flow etc., are of importance with regard to site suitability for dam construction.

According to earlier studies [2], [3] 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 [14] 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 have been derived (Appendix A5).

Avalanche and debris flow channels are clearly visible on the left valley slope (Photos 2.10 & 2.11, Appendix A4). The uppermost reaches of Union Canyon are effected by this hazard. The morphology and results from earlier geophysical studies 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.

### 2.3.4 Geological aspects of site suitability and geotechnical parameters

*2.3.4.1 Site suitability*

According to the presently available geological and geotechnical information, the area outlined on Plate 2.1 is well suited for the construction of a large dam allowing the storage of up to 1,200, 000 acre-ft. of water. Considering the morphology of Union Canyon and the availability of construction materials from nearby sources [2], [3], a concrete gravity dam appears to be the most suitable type of structure.

Areas considered for a dam further upstream in Union Canyon have several disadvantages:

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

*2.3.4.2 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', Plates 2.1 and 2.2). 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 further upstream, but within the limits outlined on Plate 2.1, 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 but not for stability evaluation of the dam (most likely and (low estimates)):

Geological Strength Index [8], [9]:

GSI = 48 (40)

Rock Mass Rating (1976):

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

Shear parameters of rock mass (Mohr-Coulomb) estimated from RMR rating according to [11], [15]:

friction angle $\varphi = 38°$ (35°)

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

Strength parameters of rock mass (Hoek/Brown criterion) [8]:

$m_b$ / $m_i$ = 0.16 (0.12)

$m_i$ = 29

s = 0.003 (0.001)

a = 0.5 (0.5)

Deformation modulus [8]:

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

Poisson`s ratio [8]:

$\upsilon$ = 0.25 (0.25)

Calculations based on P-wave velocity measurements from the seismic refraction survey (Appendix B1) indicate that the rock below 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 the vertical stresses acting on the base of the dam. Depending on the type of loading (static, dynamic or both), the vertical or normal stresses acting on the base of the dam will have maximum values of between less than 0.5 and greater than 1.0 ksi (<3.45 and >6.89 MPa).

For evaluating the stability of the dam under static and dynamic (earthquake) loading conditions, 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 Fig. 3.3, are recommended. For comparison purposes, the strength and deformation properties and failure envelope developed for a similar granodiorite rock mass (Olivenhain Dam, California [10]) are also provided and illustrated in Fig. 3.4.

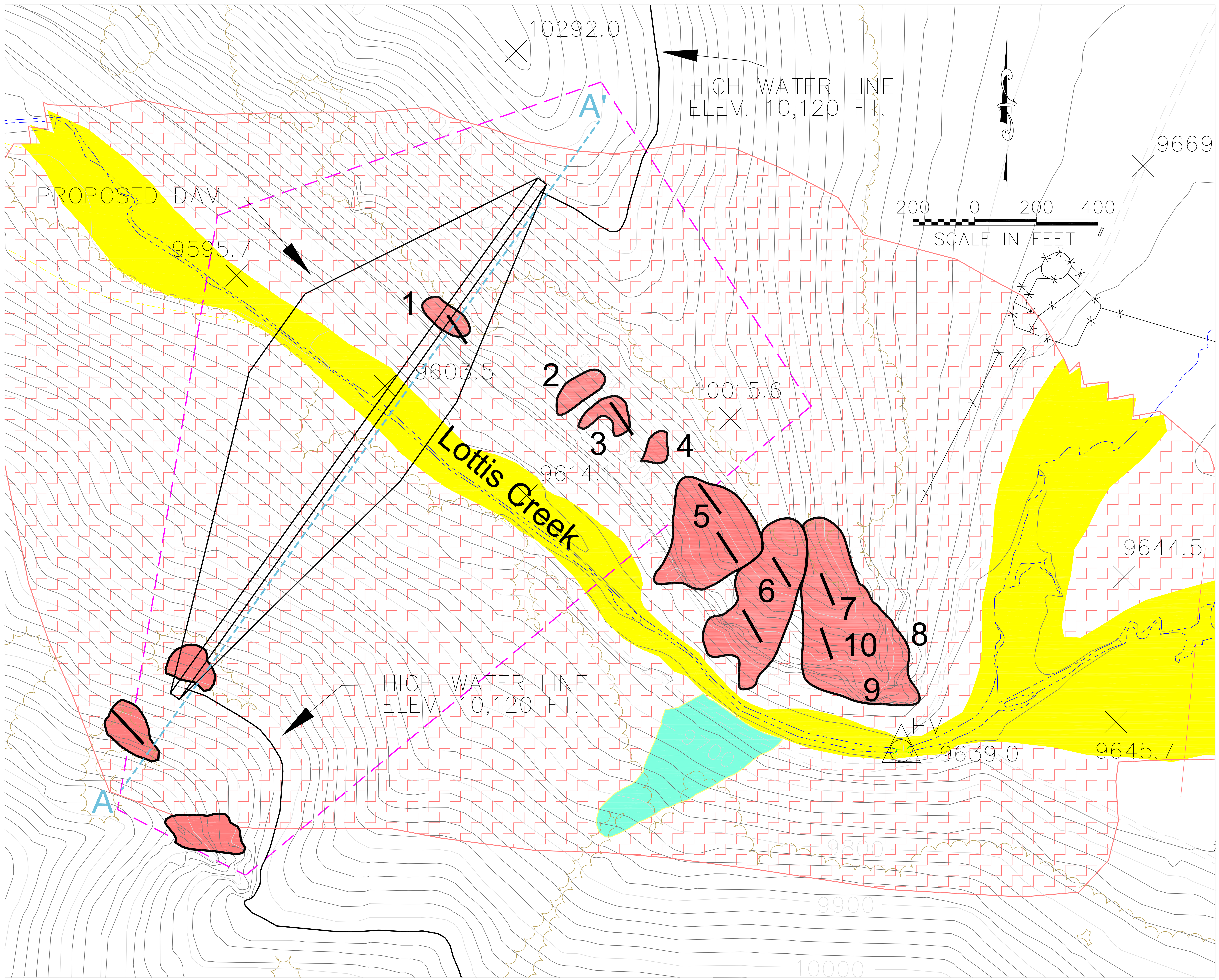
All parameters will have to be re-assessed and finalized during the second phase of the feasibility study, when the more detailed geological and geotechnical information from the core drilling and rock testing program is available.

## 2.4. Reservoir site, saddle dams, and access road

The reservoir area and saddle dam sites will be examined in detail in the next phase of this feasibility study (Section 5). The reservoir is regarded as an ideal storage site due to its favorable bowl-shaped morphology (Photos 2.12 & 2.13, Appendix A4) and the expected low permeability of the rock formation below the apparently shallow Quaternary deposits. The saddle dams are located in an area with little rock outcrop and will require seismic refraction surveys and core drilling (Plate 5.3) for foundation exploration and development of conceptual designs. Preliminary dimensions and cost information on the saddle dams have been included in Table 4.1 of the construction/project cost estimate (Section 4). The saddle dams may not be required if the reservoir capacity is reduced to 900,000 acre-ft. 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.

## 2.5. Tunnels and powerhouse

No studies have been initiated with regard to the underground facilities of the proposed project during Phase 1 of the feasibility study. Large underground openings, like the powerhouse structure, should be located in areas with favorable geological conditions. Detailed geological and geotechnical investigations are therefore required to determine the location and develop conceptual designs and construction cost estimates for the tunnels and powerhouse structure (Section 5). Rock formations like the phyllites (Sample A3, Appendix. A1) or some of the sedimentary rocks outcropping south of Taylor Park Reservoir should be avoided.




# Explanation

## General

5 Outcrop Areas examined

Suitable Dam Location Area

Cross Section Line (Plate 2.2)

## Rock Formations

Quaternary (0–2 my)

Qal — alluvial and glacial deposits

Qd — slope wash and avalanche debris accumulation, possibly unstable

Precambrian (>570 my)

xgd — granodiorite gneiss granodiorite, monzodiorite

xgd — granodiorite gneiss granodiorite, monzodiorite covered by talus material

## Rock Mass Structure

strike direction of foliation

| DESCRIPTION OF REVISION | DATE |
|---|---|
| | |
| | |
| | |
| | |
| | |

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DRAWN: GR, RJD
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DATE: 2/1/04

Union Park Dam
Geologic Map

Plate 2.1

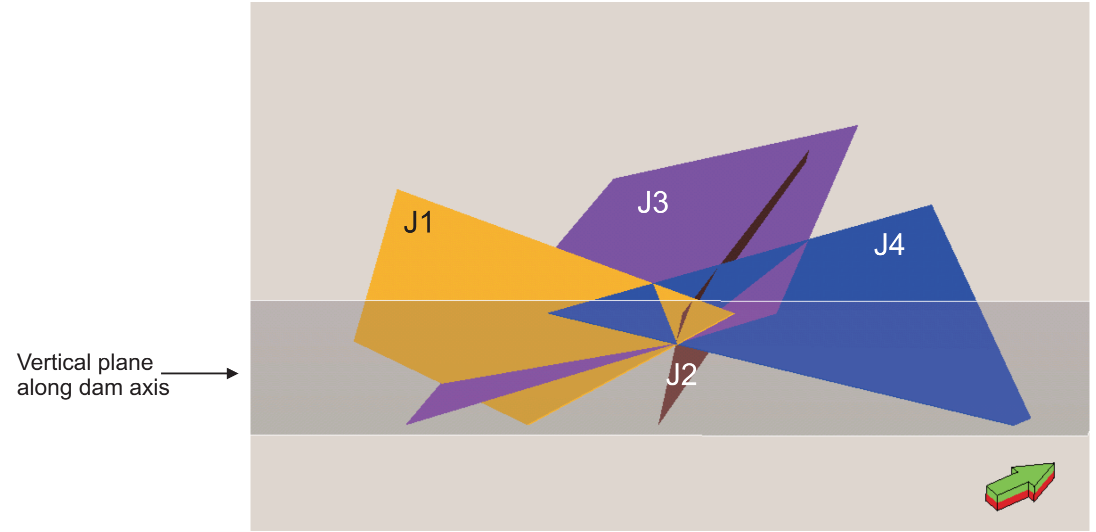
## Explanation

### Rock Formations

- Quartenary overburden talus, alluvial / glacial deposits
- Highly weathered to decomposed bedrock of underlaying Precambrian rocks
- Precambrian granodiorite gneiss granodiorite, monzodiorite

(Boundaries are based on results of refraction seismic survey)

Mean orientation of major joint-sets in 3d-view looking downstream




VIEW LOOKING DOWNSTREAM
PROPOSED AXIS OF DAM
N 35.5 E

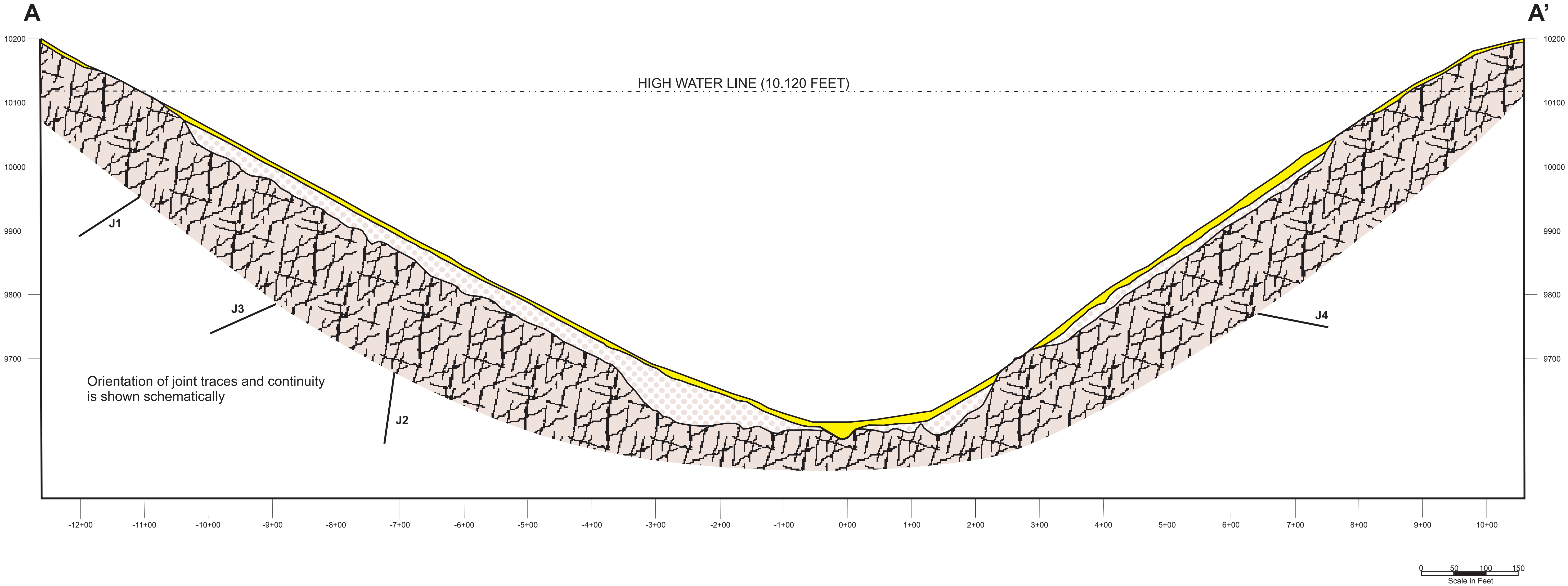




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**Plate 2.2**
Geological Cross Section A-A’

# 3. Stability Evaluation and Conceptual Design of Main RCC Gravity Dam

## 3.1. Structural competence of RCC gravity dam

A completed RCC gravity dam should function as a monolithic elastic structure, integrally bonded to its rock foundation, that is, its structural performance should be equivalent to that of a Conventional Mass Concrete (CMC) gravity dam with a similar configuration. In other words, to assure structural equivalence or for the two types of gravity dams to be equal in quality, safety and durability, they should have equivalent margins of safety against cracking, rupture, over-stressing, shearing - sliding and leakage through the concrete and construction or layer joints.

The essential criteria governing the structural competence of a gravity dam 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 and for the rock foundation (allowable bearing capacity of the rock mass).

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.

## 3.2. Stability evaluation

While stability of a gravity dam may depend more on the quality and behavior of the weaker parts of the foundation, the conjunctive stability of the structure and foundation can also be degraded by poor quality and performance of the dam. Structural cracking, high internal pore pressures, uncontrolled leakage, leaching and alkali-aggregate reaction in the dam concrete can alter the stress patterns in the foundation and abutments to such a degree that the margins of safety against failure in the foundation are reduced below acceptable limits. Therefore the conjunctive stability of the dam-foundation complex must be studied as thoroughly for RCC gravity dams, as for comparable CMC dams.

In addition, stability of the abutments of an RCC gravity dam also requires special attention, if there is axial transfer of loads from the dam or if the abutments present a hazard of potentially unstable slopes. Two and three-dimensional stability analyses of the abutments would be necessary to determine the type of treatment required; it may also influence the structural design of the dam.

### 3.2.1 Union Park Dam

To evaluate the stability and strength of the proposed RCC gravity dam, a number of finite element conceptual design models of the dam's non-overflow structure and foundation were developed. The model of the original design is illustrated in Figure 3.1. The strength and deformation properties of the rock mass used in the stability analysis of the dam are shown in shown in Fig. 3.3 together with the non-linear Hoek-Brown failure envelope. The properties are based on the average p-wave velocity measurements obtained from the seismic refraction survey of the left abutment and are believed to be representative of the quality of the granodiorite bedrock below foundation level. For the average p-wave velocity Vp = 4,252 m/s, a rock mass deformation modulus Em = 2,582,410 psi (17.81 GPa) was calculated [20]. As shown in Fig.3.3, the corresponding geologic strength index for Union Park Dam granodiorite, calculated with [17], is GSI = 60. For comparison purposes, the deformation and strength properties of a similar granodiorite rock mass from Olivenhain Dam, CA [10] are also provided (Fig. 3.4). 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 vertical compressive stresses acting on the base of the dam under extreme (combined hydrostatic and seismic) loading conditions.

Several computer runs were made by varying the structural configuration of the finite element model to evaluate the sliding stability and strength of the dam under normal operating (usual), extreme, and post-seismic loading conditions. The required safety factors against sliding of dam under these loading conditions are 3.0, >1.0, 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. The recommended minimum design strength values for the RCC used in the finite element analyses are shown in Table 3.1. The peak and residual shear strength values used for the concrete/rock interface at the base of the dam are listed in Table 3.2 and 3.3.

The results of the calculations showed that the tensile stresses in the RCC 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 (Appendix A5) caused cracking in the dam body of the original non-overflow design. The section required several modifications until the tensile stresses in the dam were low enough to eliminate cracking of the concrete (RCC). Cracking of the contact along the base of the dam and rock mass in the foundation is permitted under these extreme loading conditions. The finite element model of the final conceptual design of the non-overflow structure is illustrated in Fig. 3.2. The results of computer runs with this configuration may be examined in Appendix A6.

No studies have been initiated during Phase 1 to determine the inflow design flood for sizing the spillway and developing conceptual designs of the overflow structure for Union Park Dam. The stability of the dam will need to be further evaluated for the flood or unusual loading condition which requires a minimum safety factor of 2.0 against sliding.

Table 3.1: Minimum Design Strength of RCC [18]

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

Table 3.2: Summary of Peak Shear Strength Parameters at Concrete/Rock Contact [19]

| | | | **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 3.3: Summary of Mohr-Coloumb Residual Shear Srengths at Concrete/Rock Contact [19]

| 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 |

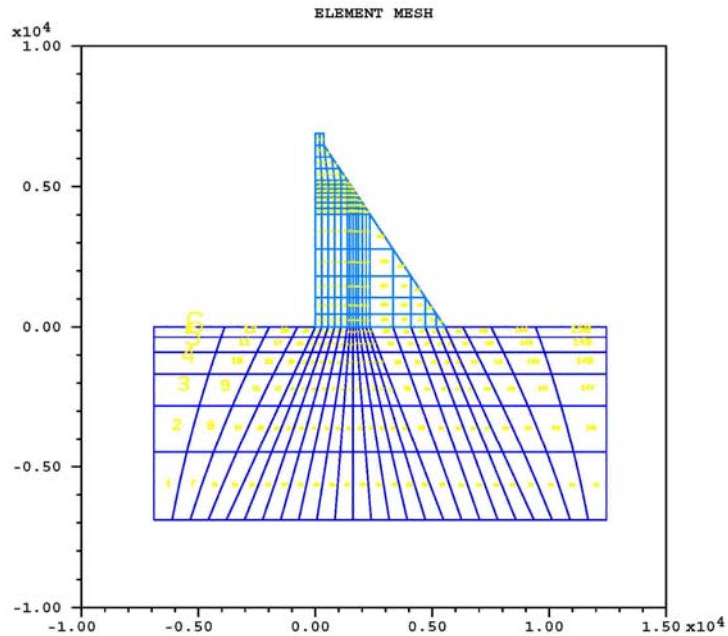


**Fig. 3.1:** Finite Element Model of Original Non-overflow Section of Union Park Dam

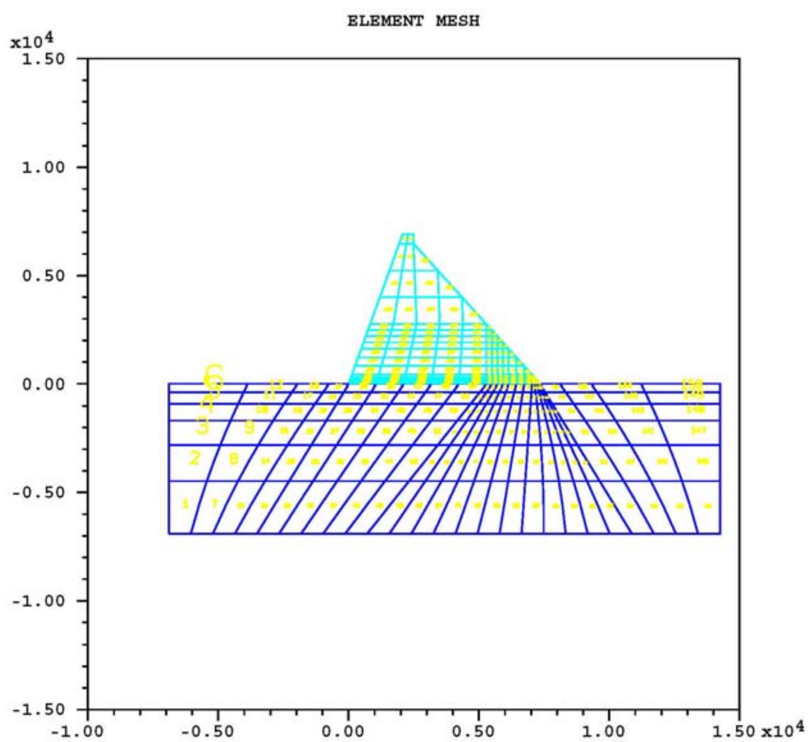


**Fig. 3.2:** Finite Element Model of Modified Non-overflow Section of Union Park Dam

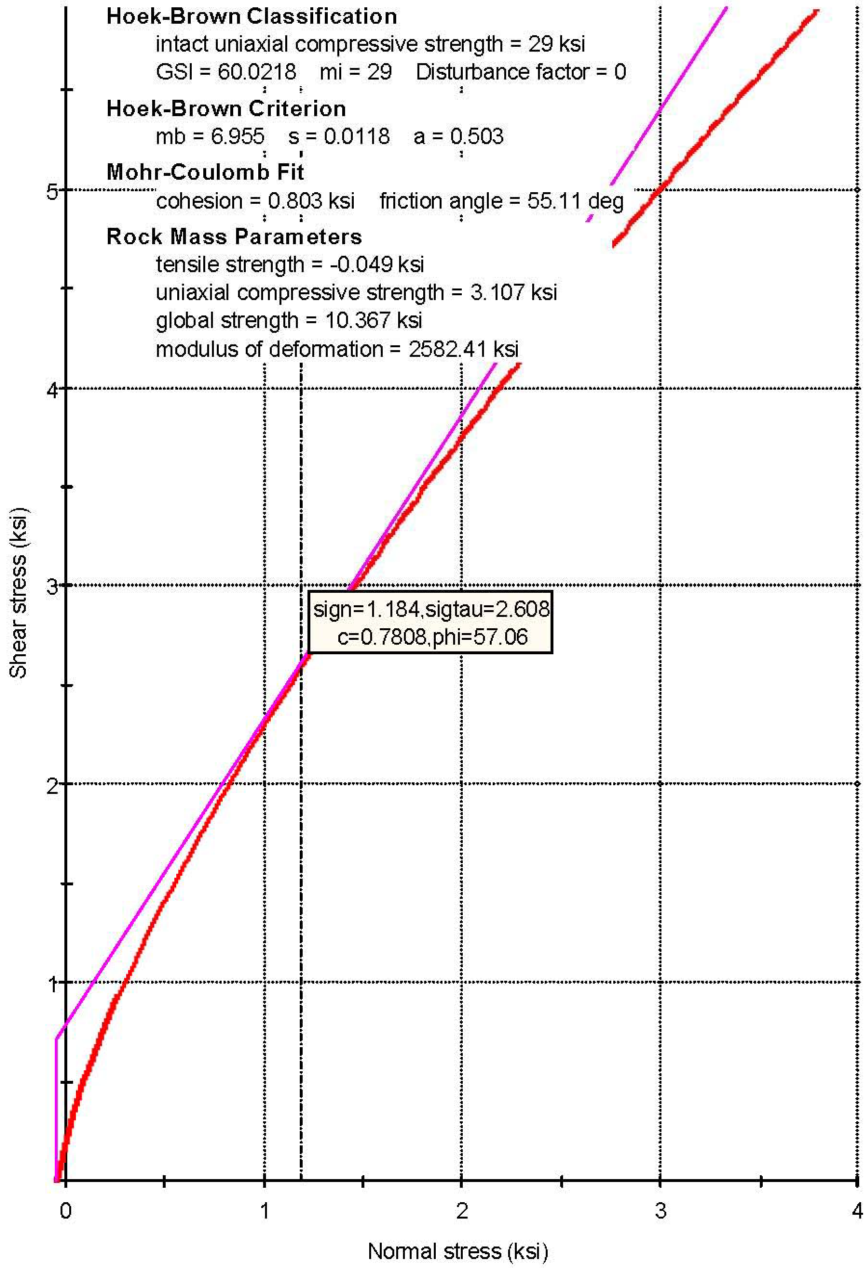


**Fig. 3.3:** Analysis of Rock Mass Strength Union Park Dam Granodiorite Left Abutment (sig3max=1.50 ksi)

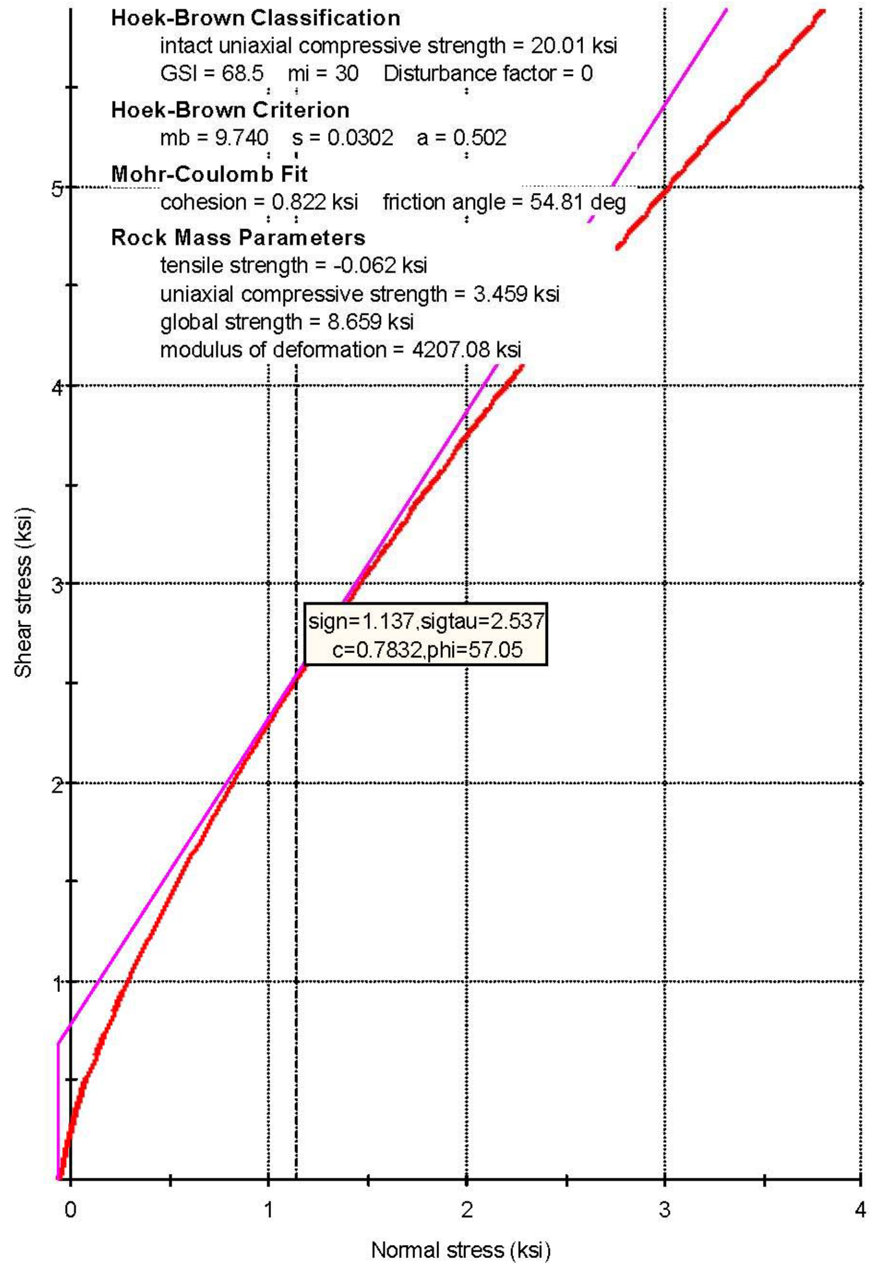


**Fig. 3.4:** Analysis of Rock Mass Strength Olivenhain Dam Granodiorite (sig3max=1.5 ksi)

# 4. Opinion of Probable Construction/Project Costs- Main Dam, Saddle Dams, and Reservoir

The cost estimate presented in Table 4.1 is very preliminary and will need to be revised after the additional geological and geotechnical information from the core drilling program is available. For example, to estimate the quantities and cost for drilling and grouting of the dam foundation, the hydraulic properties of the rock mass must be known. This information will be obtained from packer testing during core drilling. Packer testing is performed by injecting water under pressure into the rock mass surrounding the bore hole and measuring the permeability. After the hydraulic properties of the rock mass are known, a seepage analysis of the dam can be conducted, from which the dimensions and cost of the grout curtain can be determined. Additional subsurface information from core drilling is also needed to revise the quantity and cost estimate for foundation excavation and preparation.

Appendix B2 includes a cost estimate for the proposed core drilling program. A total of nine vertical borings, ea. 200 ft. deep, are proposed to be drilled along the axis of the main dam (Plate 5.1). Drilling, logging and packer testing is estimated to cost $ 274,118.

Foundation exploration along the alignment of the proposed saddle dams requires seismic refraction surveys and drilling of eight vertical core borings (Plate 5.3) at an estimated cost of $121,705. Each boring will need to be drilled to a depth of 150 ft.

One vertical 700-ft. boring is proposed for the initial subsurface exploration of the powerhouse structure (Plate 5.2). The estimate for drilling, logging, packer, and hydro-fracture testing of this bore hole is $108,594. Hydro-fracture testing is required to determine the initial stress conditions in the rock at the level of the powerhouse structure.

As pointed out earlier, Table 4.1 also 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 2000 figures [22] 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 at the end of Phase 2.

The saddle dams may not be needed if the reservoir capacity is reduced to 900,000 acre-ft. 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.

**Table 4.1:**

**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 and Geologists, Lakewood, CO* *1/25/04*

| 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** |

## 5. Conclusions and Recommendations

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 1,200,000 acre-ft. reservoir is fixed due to topographical constrains. The axis of the dam for a smaller reservoir can be located within the boundaries shown on Plate 2.1. Additional more detailed geological and geotechnical investigations supplemented by core drilling and rock testing are needed to verify and determine design parameters.

In the event that a smaller reservoir is selected, we recommend not to proceed with core drilling (Plates 5.1, 5.2, 5.3 and Appendix B2) until the surface geological mapping, seismic refraction surveys, stability evaluations, conceptual designs, and construction cost estimates for a smaller dam are completed. As pointed out above, the saddle dams may not be needed if the reservoir capacity is reduced to 900,000 acre-ft. The field work to be completed in investigating the site for a smaller dam requires a special use permit from the USDA which can be obtained by Ueblacker Associates.

If it is decided to proceed with the larger dam and reservoir, a more elaborate special use permit is required from the USDA to enter the area and conduct the core drilling. It is our understanding, that the owner of the project (NECO), will in this case be required to engage a private firm or consultant familiar with USDA permit requirements to conduct the necessary environmental studies.

Regardless of the size of the dam and reservoir selected, the following tasks should be completed during Phase 2 of this investigation:

- Photogrammetric mapping to produce the large-scale topography of the project area needed to more accurately layout facilities such as the dam(s), access road, powerhouse structure, and tunnels.
- Geologic field mapping of the reservoir area, dam sites, access road alignment, powerhouse location, and tunnel alignments to produce accurate geologic maps and cross sections.
- Seismic refraction surveys to evaluate dam foundation and reservoir slope stability, and potential borrow source areas, including sampling and testing of overburden soils and construction materials.
- Studies to determine the reservoir inflow design flood for evaluating the stability of the dam under hydrologic loading, and developing conceptual designs of the spillway and overflow structure.
- Additional studies to complete the earthquake hazard evaluation and verify ground-motion parameters (Appendix A5).


HWL 10,120 FEET
1,200,000 AF

100 0 100 200
SCALE IN FEET

34,000 Area of high velocity. Box height and orientation does not imply depth, extent or dip.

| DESCRIPTION OF REVISION | DATE |
|---|---|
| | |

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DRAWN: JP
CHECKED: HU
DATE: 2/12/04

UNION PARK DAM
RESULTS OF SEISMIC REFRACTION SURVEYS
AND PROPOSED CORE BORINGS
CROSS SECTION SHEET

Plate 5.1


| DESCRIPTION OF REVISION | DATE |
|---|---|
| | |
| | |
| | |
| | |
| | |

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UNION PARK DAM

POWER HOUSE DRILL HOLE LOCATION

PLATE 5.2


| DESCRIPTION OF REVISION | DATE |
|---|---|
| | |
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UNION PARK DAM

SADDLE DAM DRILL HOLE LOCATIONS

PLATE 5.3

# 6. Acknowledgments

The following professionals provided assistance in conducting the Phase 1 – Feasibility Level Geological and Geotechnical Investigation for Union Park Dam:

- Photogrammetric Mapping and Drafting: John Wedding and Jerry Panek
- Engineering Geology and Rock Mass Characterization: Gerhard Reik, Horst Ueblacker, Christian Weiler, Randy Dorian, and Sharon Stinnette
- Laboratory Strength Testing of Intact Rock and Petrographic Analysis: Gerhard Reik and Christian Weiler
- Photo Documentation of Geologic Field Mapping, Management of Electronic Data Files, and Drafting: Randy Dorian
- Geophysics (Seismic Refraction Surveys): Linda Hadley, Dan Dougherty, and William Darken
- Earthquake Hazard Evaluation and Determination of Ground-Motion Parameters: Horst Ueblacker
- Stability Evaluation and Conceptual Design of Main RCC Gravity Dam: Horst Ueblacker
- Opinion of Probable Construction/Project Costs: Horst Ueblacker
- Report: Horst Ueblacker, Randy Dorian, Gerhard Reik, and Hanna Brouver

Professional judgments presented in this preliminary report are based partly on evaluation of technical information gathered and partly on our understanding of the prevailing geological conditions in the area and characteristics of the facility being planned. We do not guarantee the performance of the project in any respect, only that our engineering and judgments rendered meet the standard of care of our profession.

If we may be of further service in discussing the contents of this report, please call the undersigned at (303) 988-9489.

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# Union Park Dam

# Appendix A 1

# Petrographic Examination of Bedrock Samples

# A 1 Petrographic Examination of Bedrock Samples

## A 1. 1 Introductory Remarks

Two rock samples from outcrops at of he right valley flank of Union Canyon and one sample collected from rock cuts along the highway East of Taylor Park Dam have been examined in thin section microscopically. Samples A1 and A2 are characteristic for the bedrock at the upper reaches of Union Canyon.

Sample A3 is an example of weak rock which may be encountered along the waterway structures. The results of microscopic studies on thin sections are reported in the following .

## A 1. 2 Results of Microscopic Examination

### A 1. 2.1 Sample A 1

Sample A1 obtained from outcrop area 1 (see Fig. A1.9) is characteristic of the only slightly weathered granodioritic rock exposed at Union Canyon outcrops.

The rock is medium to fine grained, light grey to grey in colour and equigranular in texture.
The fine to medium grained granodioritic rocks are composed essentially of quartz, plagioclase feldspar, and K-feldspar, biotite and chloritized hornblend. Sphene, iron oxides and zircon occur as accessory minerals whereas sericite and kaolinite occur as secondary minerals.
Quartz is found in appreciable amount as euhedral to subhedral irregular crystals and invaded most of the essential constituents; plagioclase feldspar, K-feldspar and mafic minerals.
Plagioclase is represented by euhedral to subhedral crystals. The plagioclase crystals are colourless and sometimes are clouded due to their alteration to sericite, epidote and clay minerals especially at their core rather than their rims. They show lamellar twining (Fig. A1.1). Some megacrystals from plagioclase crystals show zoning. Sometimes these megacrystals show poikilitic texture where they contain small flaky crystals from biotite, chlroite and quartz (Fig. A1.1).
K-feldspar is represented by orthoclase, microcline and perthite. They occur as euhedral to subhedral elongated crystals. Exsolution lamellae of albite in K-feldspar lead to perthitic texture. Microcline shows crosshatched twining whereas orthoclase shows Carlsbad simple twining (Fig. A1.2). Most of K-feldspar crystals are highly altered to kaolinite and sometimes intergrowth with quartz leads to graphic texture.
Biotite occurs as greenish brown to brownish green crystals. It is pleochroic and has strong birefringence. It forms euhedral to subhedral flaky to tabular crystals and shows a distinct trend of orientation (Fig. A1.3). Biotite is commonly altered to chlorite and the alteration increases along the cleavage plane.
Two types of chlorite occur, the first type is flaky in shape and the other is euhedral to subhedral suggesting that the chlorites originated from biotite (flaky to tabular crystals) and hornblende (euhedral to subhedral prismatic crystals).
Iron oxides occur as euhedral to subhedral crystals associated with mafic minerals and sphene. Sometimes it is found as by-product due to the

alteration of mafic minerals to chlorite. Sphene forms brown euhedral to subhedral rhombic crystals and shows non-to weakly pleachroism. It exhibits high birefringence and high relief. It is commonly associated with biotite, chlorite and opaque minerals. Zircon occurs as minute euhedral six sided prismatic crystals. It is associated with plagioclase and sometimes enclosed in biotite or chlorite (Fig. A1.3). According to its mineral composition and location in the Streckeisen diagram the rock is identified as granodiorite. Considering its structural arrangement of minerals (Fig. A1.4) it is a granodiorite gneiss.

### A 1. 2.2 Sample A 2

Sample A 2 has been obtained from outcrop area 1 (see Fig. A1.9). This type of rock is representative of part of this outcrop area.

The rock specimen examined is medium to fine grained and of grey colour. It has a porphyritic texture with whitish plagioclase phenocrysts. The mineral constituents of the rock are strongly altered and largely composed of very highly sericitized to kaolinitized feldspars (Fig. A1.5). They are very turbid in colour due to the alteration. The feldspar phenocrysts are less altered than feldspar occurring in the groundmass. A considerable amount of quartz occurs in the feldspar groundmass giving rise to graphic texture (Fig. A1.5). Iron oxides occurs in two forms rounded to subrounded grains and in some cases angular to subangular crystals. Sometimes it is associated with chlorite (Fig.A1.6). Muscovite occurs as euhedral to subhedral crystals enclosed in feldspar groundmass. Sometimes it is associated with iron oxide minerals and sphene. Locally muscovite occurs within the sericitized feldspar (Figs A1.5 and A1.7). According to its modal composition and location in the Streckeisen diagram the rock is identified as an altered monzodiorite.

### A 1. 2.3 Sample A 3

Sample A 3 is a fine-grained metamorphic rock with well-developed foliation due to the arrangement of phenocrystals of feldspar, quartz and mafic minerals (Fig. A1.8). The studied phyllite typically consists of highly altered feldspar, quartz, biotite and chlorite. The feldspar is frequently altered to sericite and kaolinite. It commonly forms the groundmass and sometimes occurs as phenocrysts. Quartz is found in two generations rounded to subrounded grains or as part of the kaolinitic to sericitized groundmass. Iron oxides occur as fine to minute crystals associated with green flaky chloritized biotite.

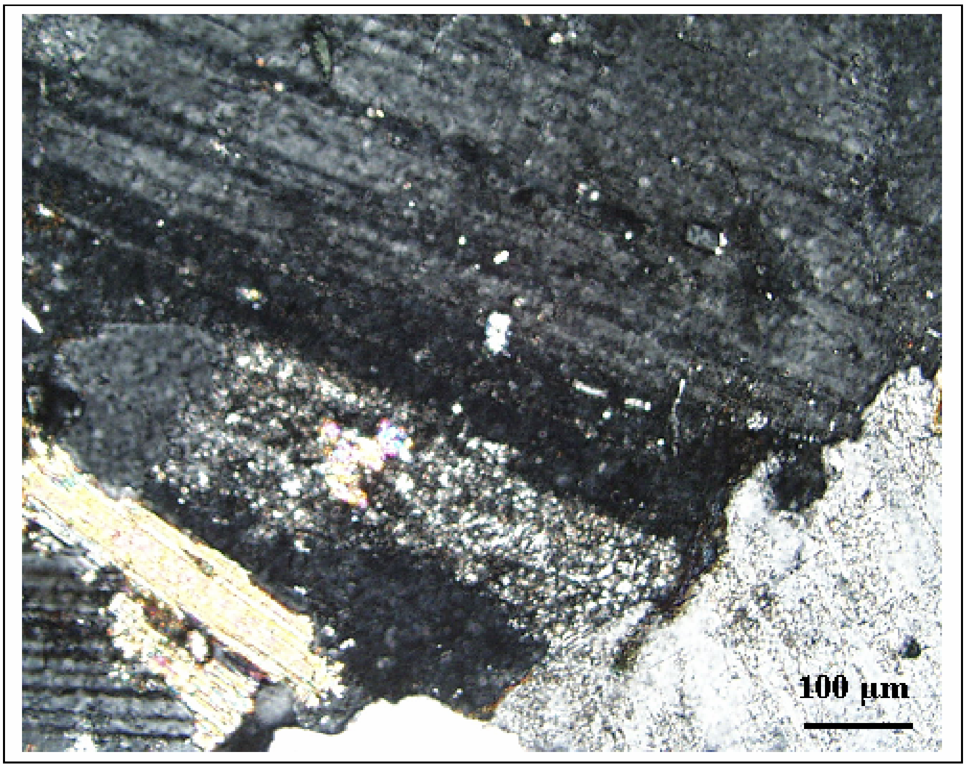



**Fig. A1.1:** Plagioclase shows lamellar twining and signs of alteration to sericite. Flaky biotite crystals are also occur and poiklitic texture. (granodioritic rocks).

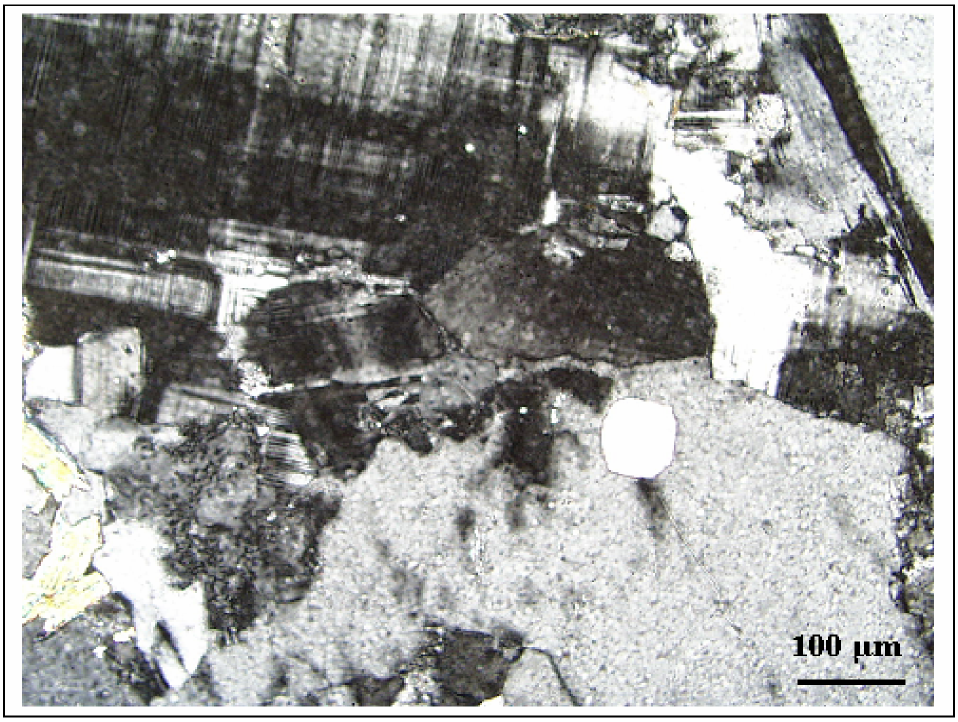



**Fig. A1.2:** Microcline crystal shows cross-hatched twining. Perthitic texture due to the exsolution of albite within K-feldspar is also recognized (granodioritic rocks).

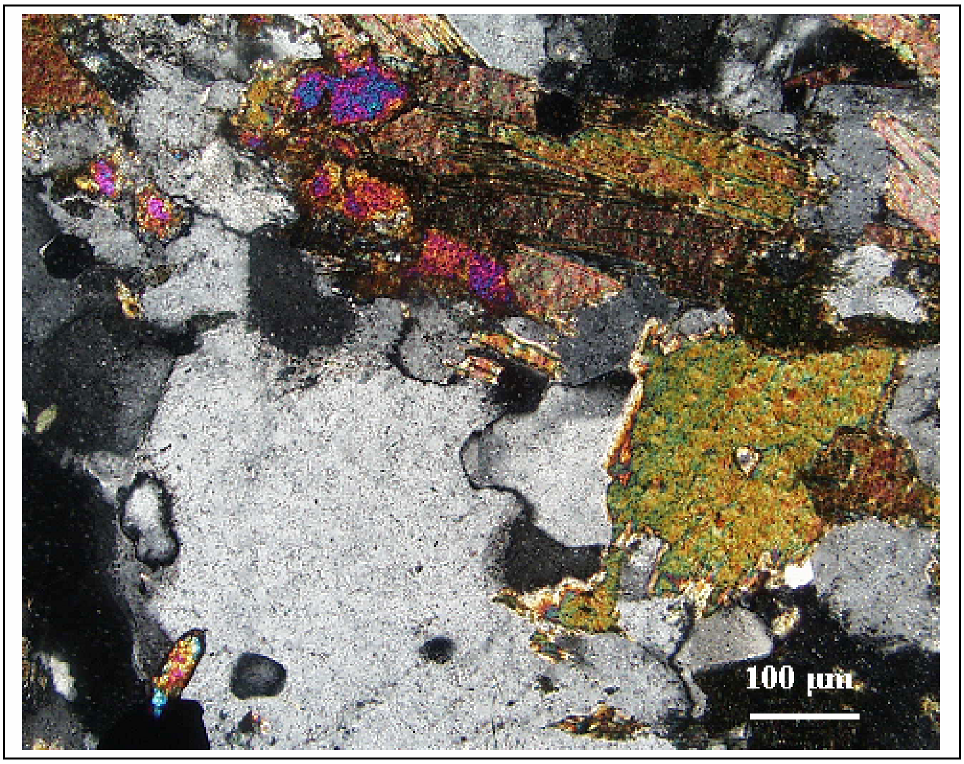



**Fig. A1.3:** Flaky euhedral to subhedral crystals of biotite in association with iron oxide and zircon (granodioritic rocks).

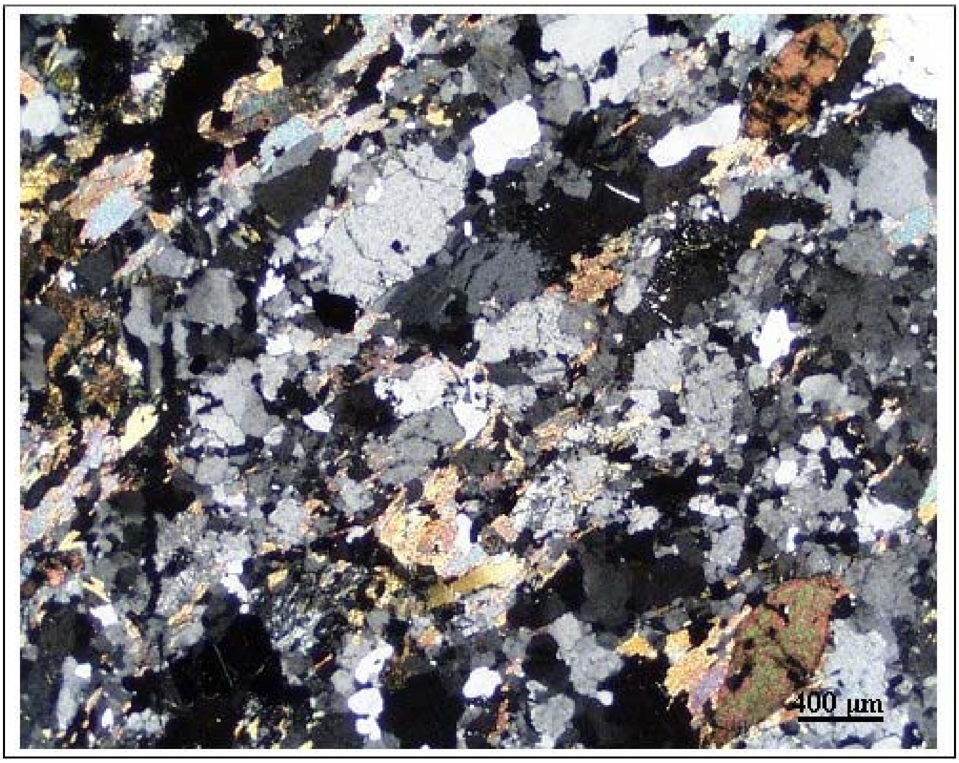



**Fig. A1.4:** Gneissic structure of granodioritic rock due to concentration of white and dark minerals in parallel layers or bands.

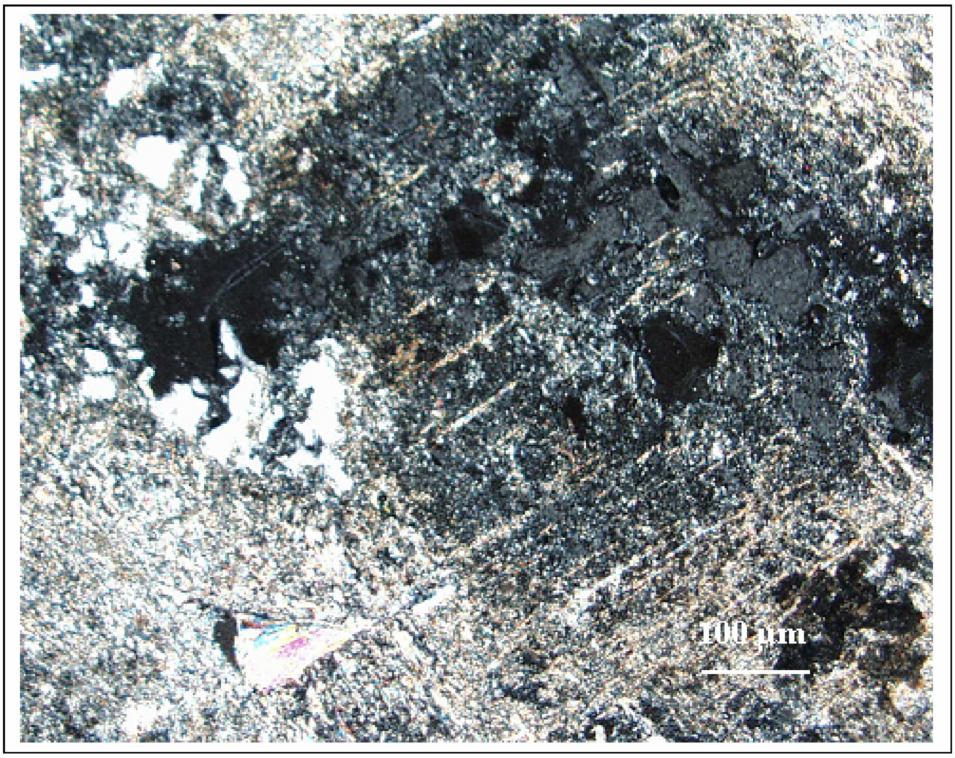



**Fig. A1.5:** Highly sericitized plagioclase crystal with intergrowth of quartz grains giving rise to graphic texture. Muscovite crystals are also seen in association with plagioclase feldspar (monzodioritic rocks)

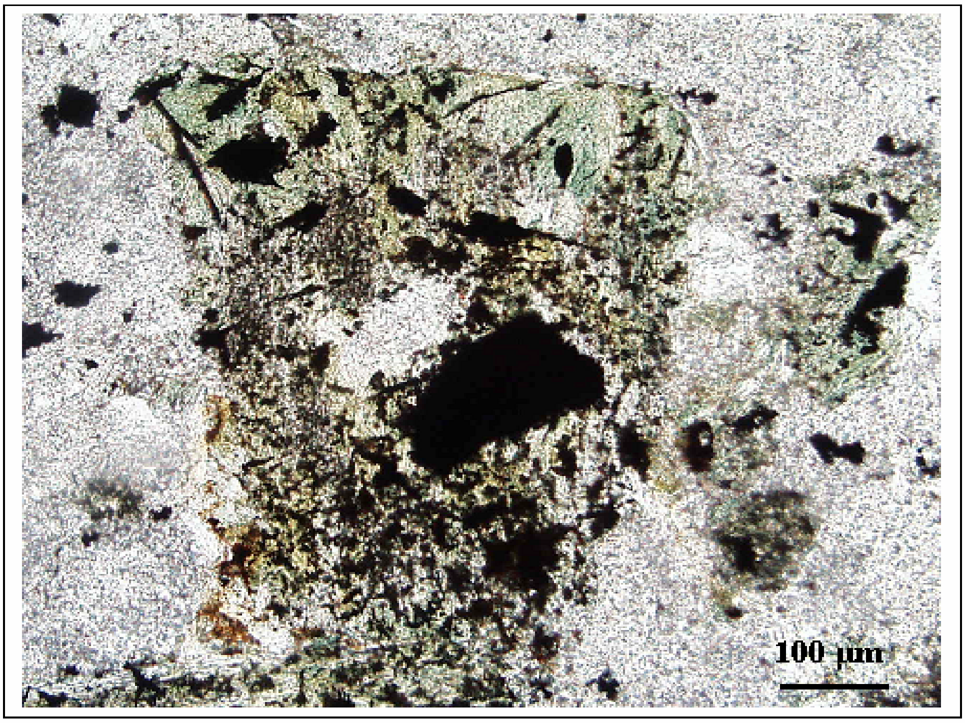



**Fig. A1.6:** Chlorite crystals associated with iron oxide in highly altered plagioclase groundmass (monzodioritic rocks).

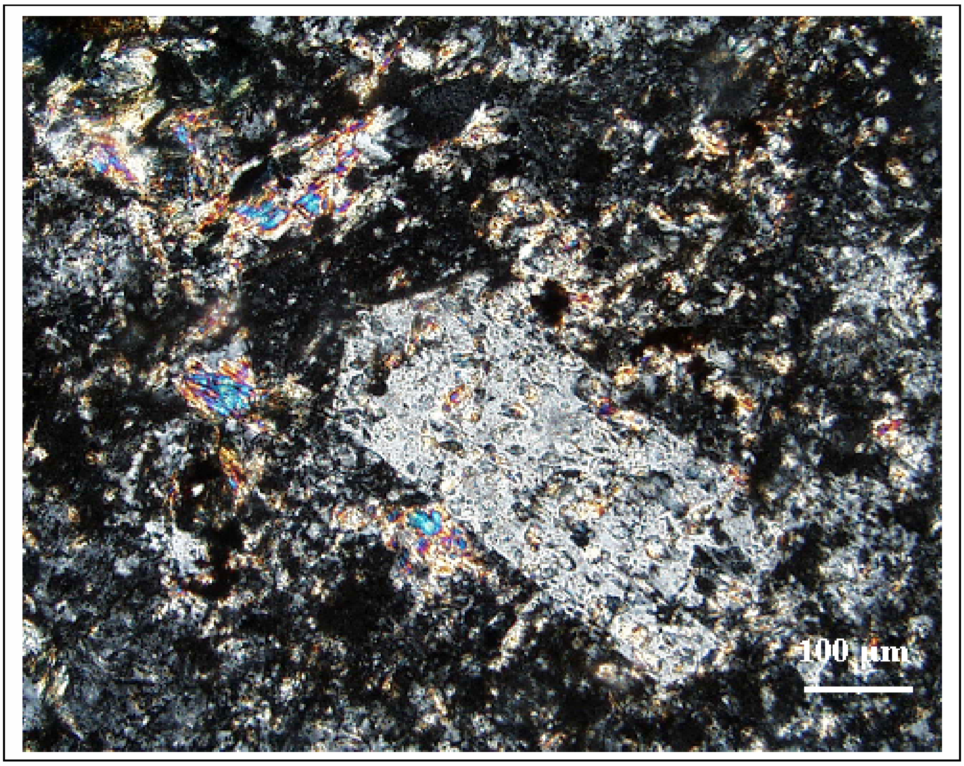



**Fig. A1.7:** Plagioclase crystals occur as phenocrysts and groundmass. Some muscovite crystals are scattered in the groundmass (monzodioritic rocks).

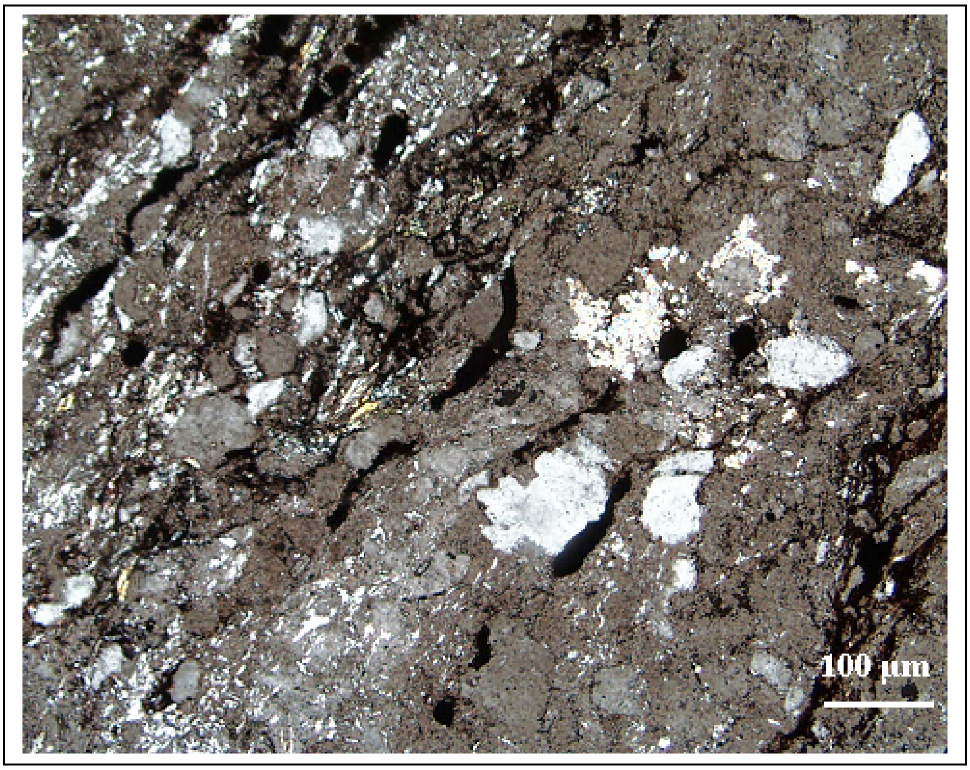



**Fig. A1.8:** Phyllite showing orientation and foliation texture of quartz grains, flaky chloritized biotite and iron oxides in a highly weathered feldspar groundmass.

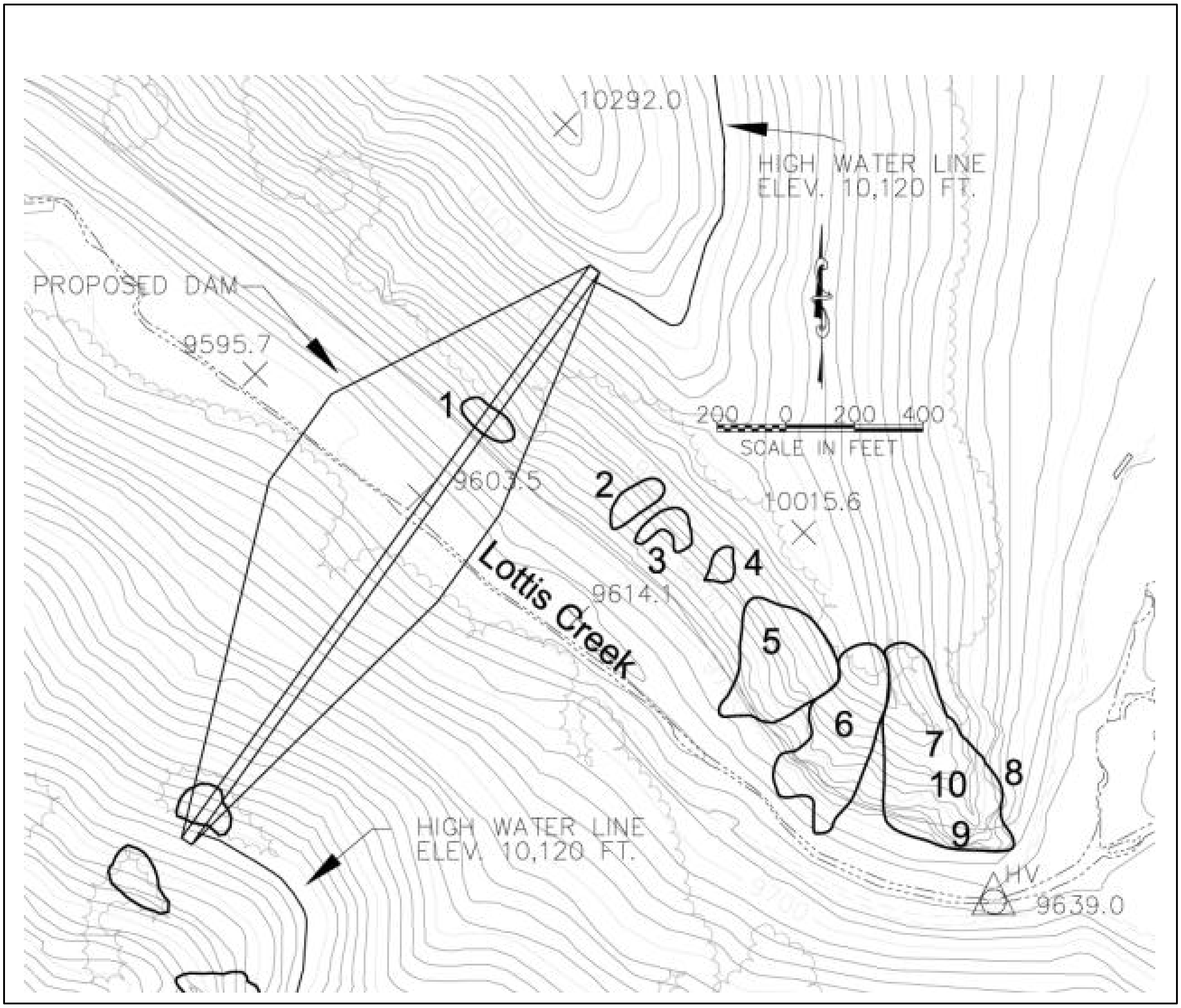



**Fig. A1.9:** Location of Outcrop Areas

# Union Park Dam

# Appendix A 2

# Strength of Intact Rock

# A 2 Strength of Intact Rock

## A 2. 1 Field tests

The granodiorite gneiss and gneissic granodiorite rocks have been classified as “very strong rock” by geological field tests according to ISRM recommendations and ISO 14689.

The uniaxial strength of “very strong rock” ranges from 100 MPa to 250 MPa.

Field tests with a Schmidt hammer on a large massive block of granodiorite gneiss from Outcrop Area 9 (location see Fig A1.9) gave an average value of 56 after correction. According to the graph provided with the instrument this corresponds to a maximal strength of:

$\sigma_c$ = 35,000 psi or

= 241 MPa

and a deformation modulus of:

$E_t$ = 10,500,000 psi or

= 72,414 MPa

for the fresh intact rock.

## A 2. 2 Laboratory tests

Two uniaxial tests on rock cores obtained from granodiorite gneiss samples derived from Outcrop Area 1 have been conducted. The two rock samples were fresh to slightly discolored.

Uniaxial strength values of:

$\sigma_c$ = 175 MPa for core piece a, and

$\sigma_c$ = 190 MPa for core piece b

were obtained ( Fig. A2/2 and A2/3).

As shown on the photographs of the Fig. A2/2 and A2/3 the conjugate shear fractures leading to failure of the samples are parallel to the strike of the foliation. This may indicate a slight anisotropy of the intact rock strength.

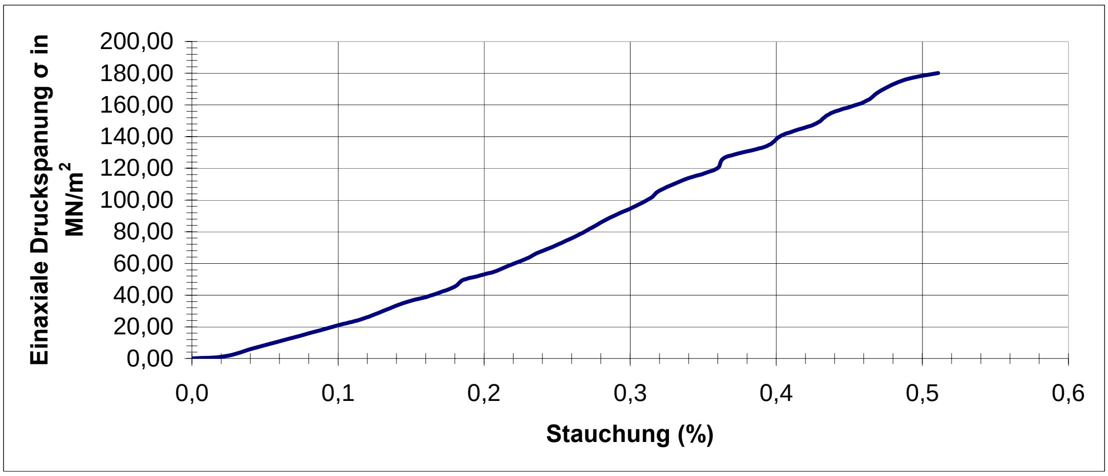
The deformation moduli determined are:

$E_t$ = 48,210 MPa for sample A2/a and

$E_t$ = 43,163 MPa for sample A2/b (see Fig. A2/2 and A2/3).

| | | | |
|---|---|---|---|
| **Einaxiale Druckversuche an Gesteinsproben** | Aktenzeichen: | Anlage: A2 | Blatt: 1 |
| **(nach TP BF - StB - Teil C1)** | **Projekt:** | **Union Park Dam** | |
| | **Projekt Nr.:** | | |
| | **Probe Nr.:** | **a** | |

| | | | |
|---|---|---|---|
| ***Bearbeiter: Ali Ismail, M.Sc., Dipl.-Geol. Christian Weiler*** | | **Entnahmestelle:** | |
| **Versuchsdatum:** | 27.10.2003 | **Probeart:** | |
| **Prüfkörperlänge (l):** | 48,10 mm | **Entnahmetiefe:** | |
| **Prüfkörperdurchmesser (d):** | 30,00 mm | **Boden- / Gesteinsansprache:** | |
| **Höhen/Durchmesserverhältnis (l/d):** | 1,60 | | |
| **Korrekturfaktor (bei l/d<2): f = 8/(7+2d/l):** | 0,970 | **Stratigraphie:** | |
| | | **Entnahmedatum:** | |







| | | | |
|---|---|---|---|
| **Bruchspannung (σ):** | 180,16 MN/m² | **Feuchtdichte:** | 2,70 g/cm³ |
| **einaxiale Druckfestigkeit (bei l/d<2) $\sigma_u$=f*σ:** | 174,75 MN/m² | **Wassergehalt:** | |
| **Stauchung beim Bruch:** | 0,51 % | **Trockendichte:** | |
| **Verformungsmodul ($V_{40-60}$:)** | 48210 MN/m² | | |

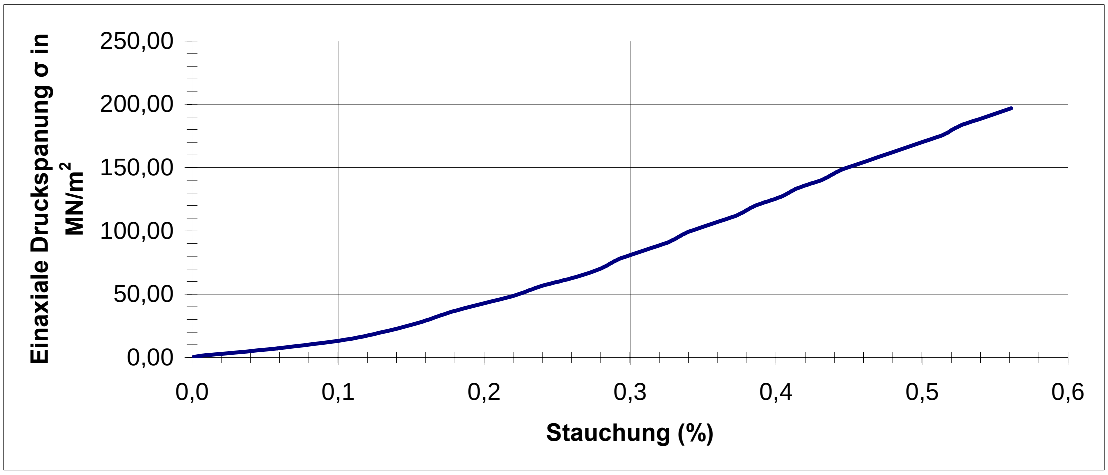
**Bemerkungen:**

**Technische Universität Clausthal**
Institut für Geologie und Paläontologie
Abteilung für Ingenieurgeologie
**Prof. G. Reik, PhD**
Leibnizstraße 10, 38678 Clausthal-Zellerfeld

Fig. A2.2: Results of Uniaxial Strength Test , Sample A2/a

| | | | |
|---|---|---|---|
| **Einaxiale Druckversuche an Gesteinsproben** | Aktenzeichen: | Anlage: A2 | Blatt: 2 |
| **(nach TP BF - StB - Teil C1)** | **Projekt:** | **Union Park Dam** | |
| | **Projekt Nr.:** | | |
| | **Probe Nr.:** | **b** | |

| | | |
|---|---|---|
| ***Bearbeiter: Ali Ismail, M.Sc., Dipl.-Geol. Christian Weiler*** | | **Entnahmestelle:** |
| **Versuchsdatum:** | 27.10.2003 | **Probeart:** |
| **Prüfkörperlänge (l):** | 46,20 mm | **Entnahmetiefe:** |
| **Prüfkörperdurchmesser (d):** | 30,00 mm | **Boden- / Gesteinsansprache:** |
| **Höhen/Durchmesserverhältnis (l/d):** | 1,54 | |
| **Korrekturfaktor (bei l/d<2): f = 8/(7+2d/l):** | 0,964 | **Stratigraphie:** |
| | | **Entnahmedatum:** |




| | | | |
|---|---|---|---|
| **Bruchspannung (σ):** | 196,95 MN/m² | **Feuchtdichte:** | 2,67 g/cm³ |
| **einaxiale Druckfestigkeit (bei l/d<2) $\sigma_u$=f*σ:** | 189,86 MN/m² | **Wassergehalt:** | |
| **Stauchung beim Bruch:** | 0,56 % | **Trockendichte:** | |
| **Verformungsmodul ($V_{40-60}$:)** | 43163 MN/m² | | |

**Bemerkungen:**

**Technische Universität Clausthal**
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**Prof. G. Reik, PhD**
Leibnizstraße 10, 38678 Clausthal-Zellerfeld

Fig. A2.3: Results of Uniaxial Strength Test, Sample A2/b

# Union Park Dam

# Appendix A 3

# Discontinuity Survey

# A 3 Discontinuity Survey

## A 3. 1 Introductory remarks

Because of the importance of discontinuities – generally described as “surfaces within the rock mass that are open or may become open by the action of stresses resulting from an applied load ” [ 7 ] – a discontinuity survey was conducted. The survey covered the accessible exposures of the rock mass along the upper part of Union Canyon. At the various outcrop areas (see location map Fig. A1.9) the orientation of joint surfaces were measured and main characteristics of joints and joint-sets have been observed.

## A 3. 2 Joint orientation

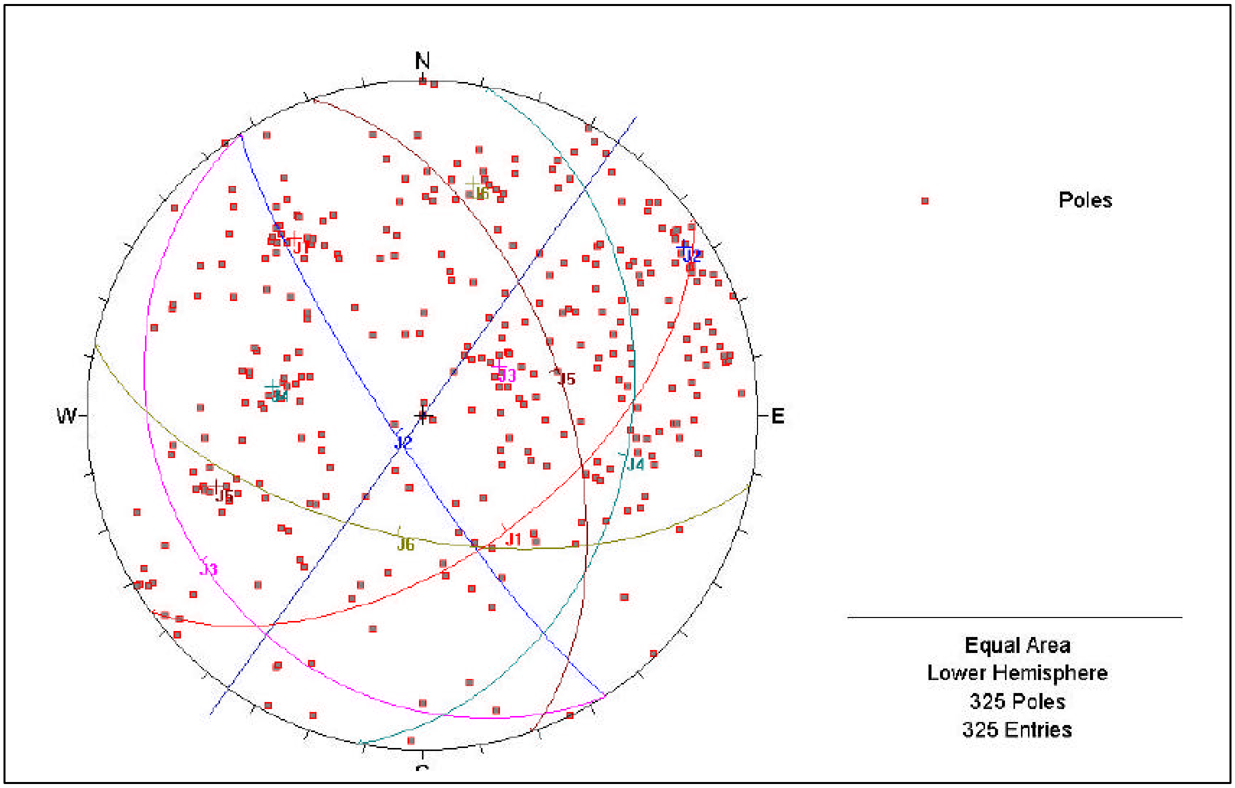
The data from geological compass measurements have been processed and analyzed with the “dips” program [13]. The results are shown in the form of stereographic projection plots for the various outcrop areas or group of areas (Fig. A3.2.1 to A3.2.7). The data have been analyzed with regard to a preferred orientation and grouping into joint sets. The mean orientation of the sets so obtained are given in Fig. A3.2.1 to A3.2.7.

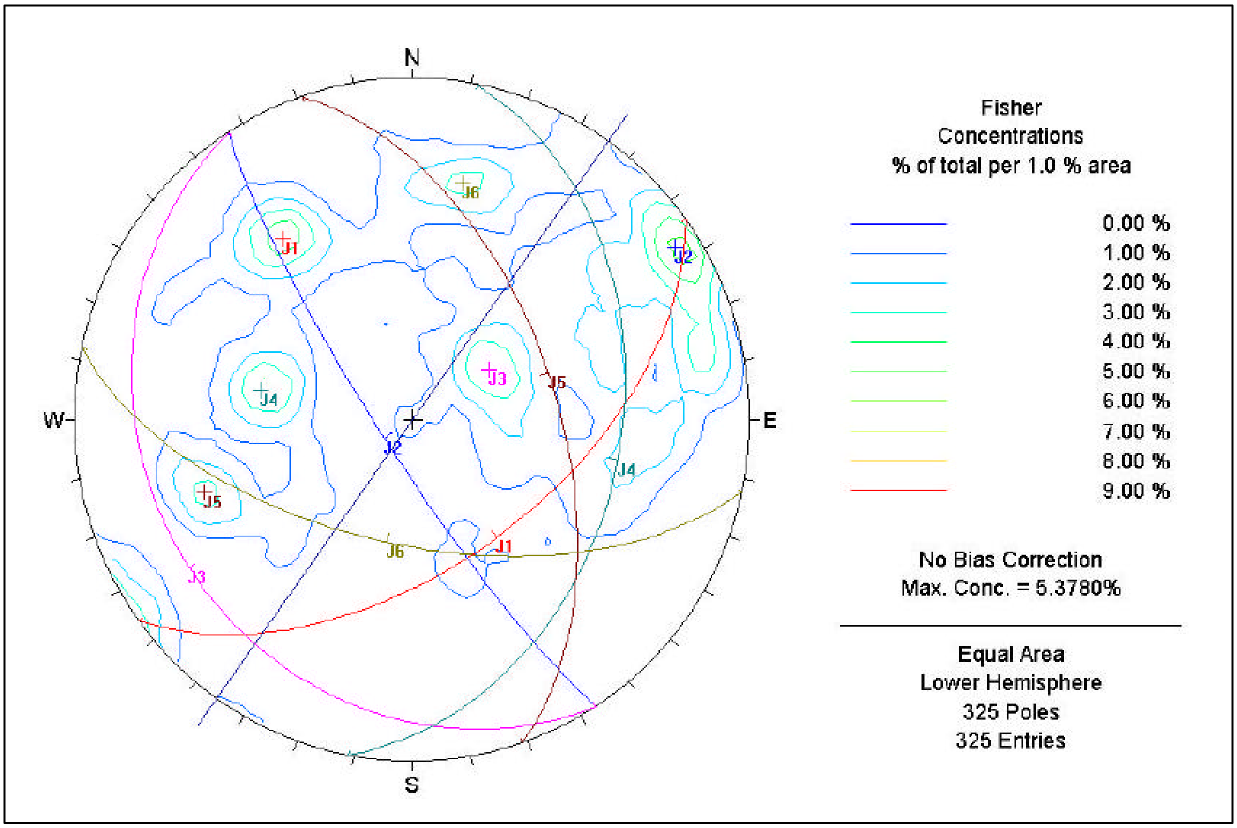
Despite the scatter in the orientation data distinct joint sets can be identified. The most consistent set J2 dips steeply in NE’ or SW’ directions. These joints are oriented approximately parallel to the foliation of the gneissic rock and to most of the larger quartz veins observed in the area. The strike of joint set J2 is also about parallel to the trend of the prominent rock ridge on the right valley side.

Joint set J1 can also clearly be recognized at all locations. It can be statistically separated into two or three subsets at some outcrop areas. The joints of this set dip upstream in S’ to SE’ directions at intermediate to steep angle. The mean orientation of J1 is approx. normal to the orientation of the mean orientation of joint set J2 (Photos A3.1, A3.2 and A3.3, App. A4).

A third set of joints -J3 - about parallel in strike to joint set J2 can be recognized at most locations. It dips towards Lottis Creek in SW’ direction at low to intermediate dip angle (Photos A3.3 & A3.4, App. A4). At Outcrop Areas 2 to 4 the orientation changes to WNW’ dip directions.

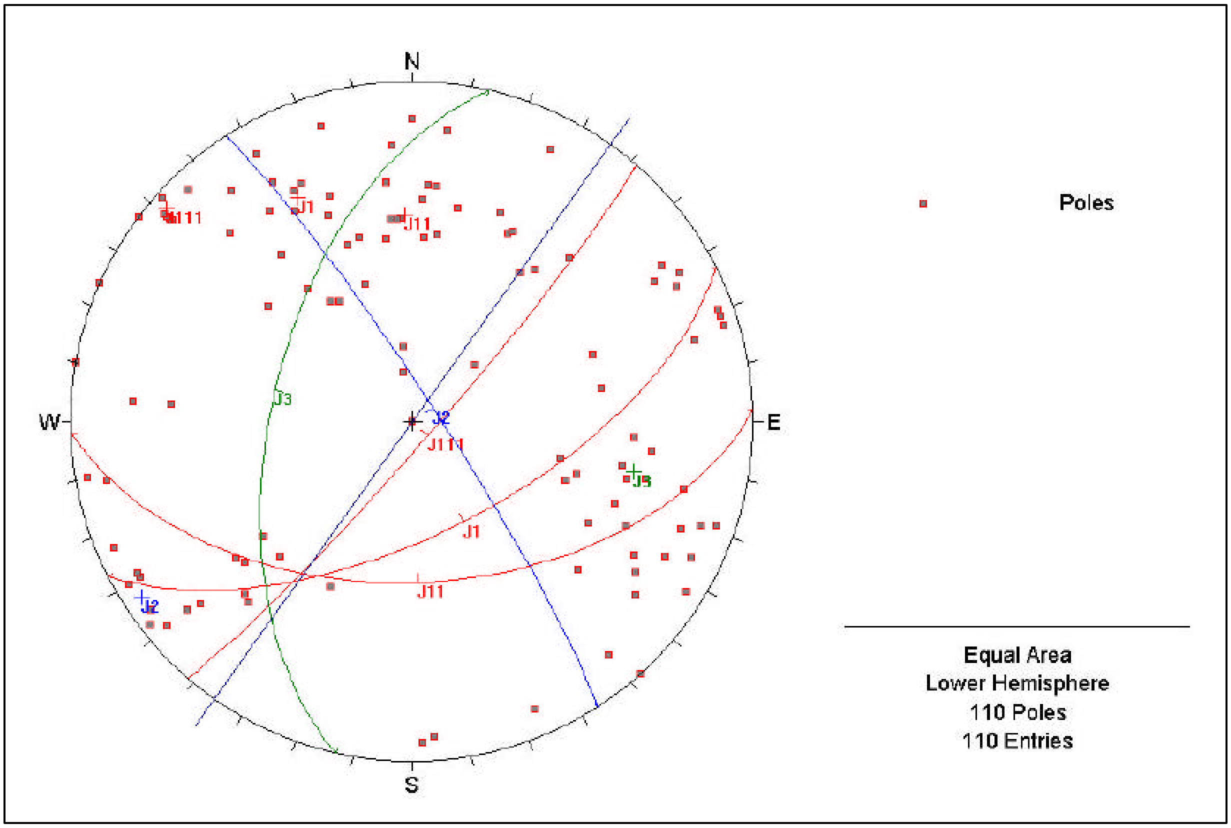
Further minor joint sets can be recognized by the concentration of poles to the joint planes for example at Outcrop Areas 1 and 7 to 10. These sets are not consistent over larger areas.

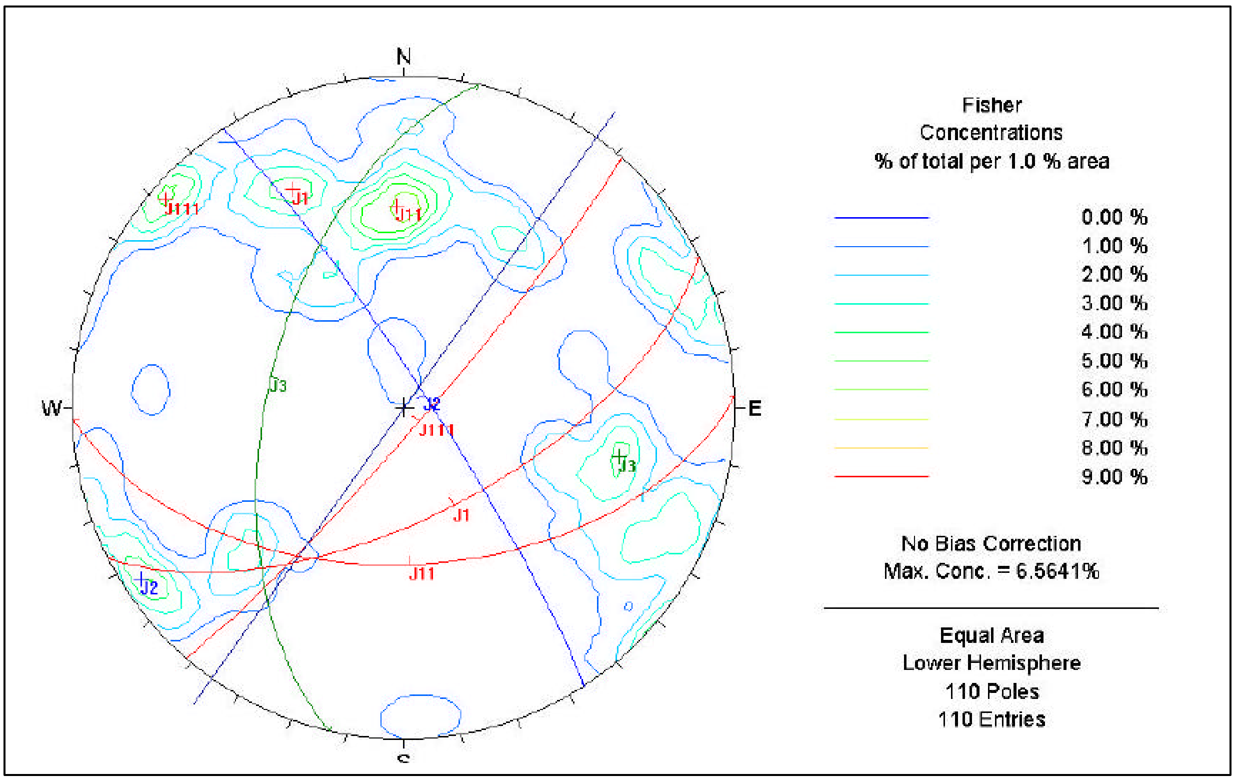



| Mean orientation of joint sets at outcrop area 1 (dip/dip direction in degrees): | | | | | |
|---|---|---|---|---|---|
| J1 | J2 | J3 | J4 | J5 | J6 |
| 55/144 | 82/237 | 22/237 | 38/101 | 55/71 | 60/192 |

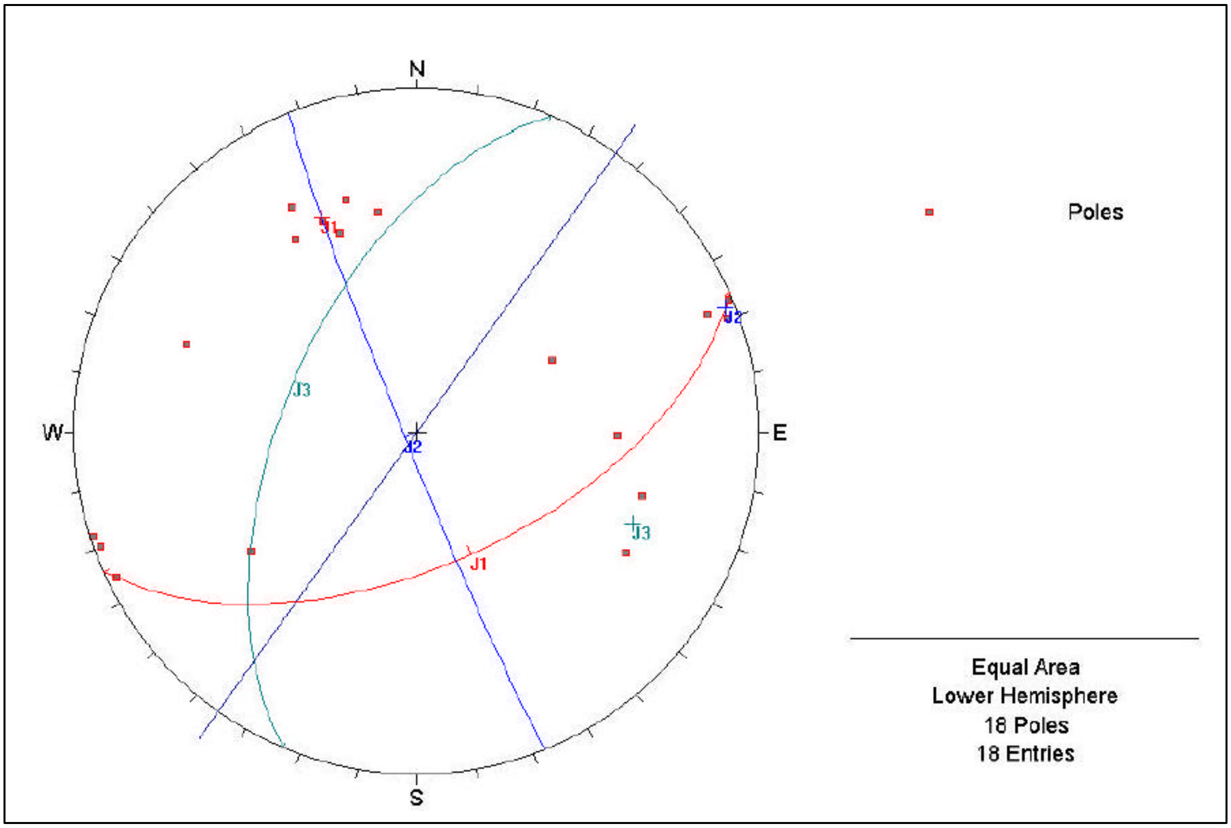
**Fig. A3.2.1:** Union Park Dam, Outcrop Area 1, Distribution of poles to joint surfaces and pole concentration with identification of joint sets.

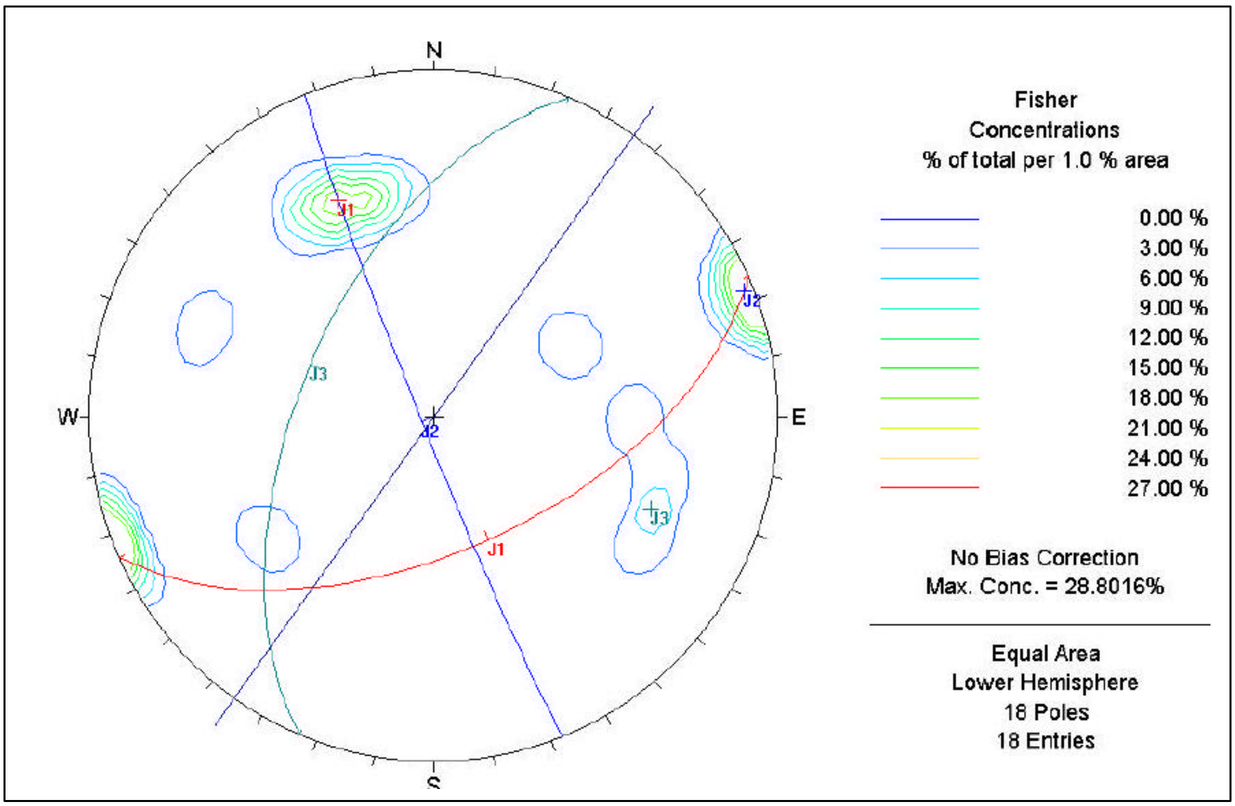



| Mean orientation of joint sets at outcrop areas 2 and 3 (dip/dip direction in degrees): | | | | |
|---|---|---|---|---|
| J1<br>63/152 | J11<br>51/178 | J111<br>85/131 | J2<br>84/57 | J3<br>56/283 |

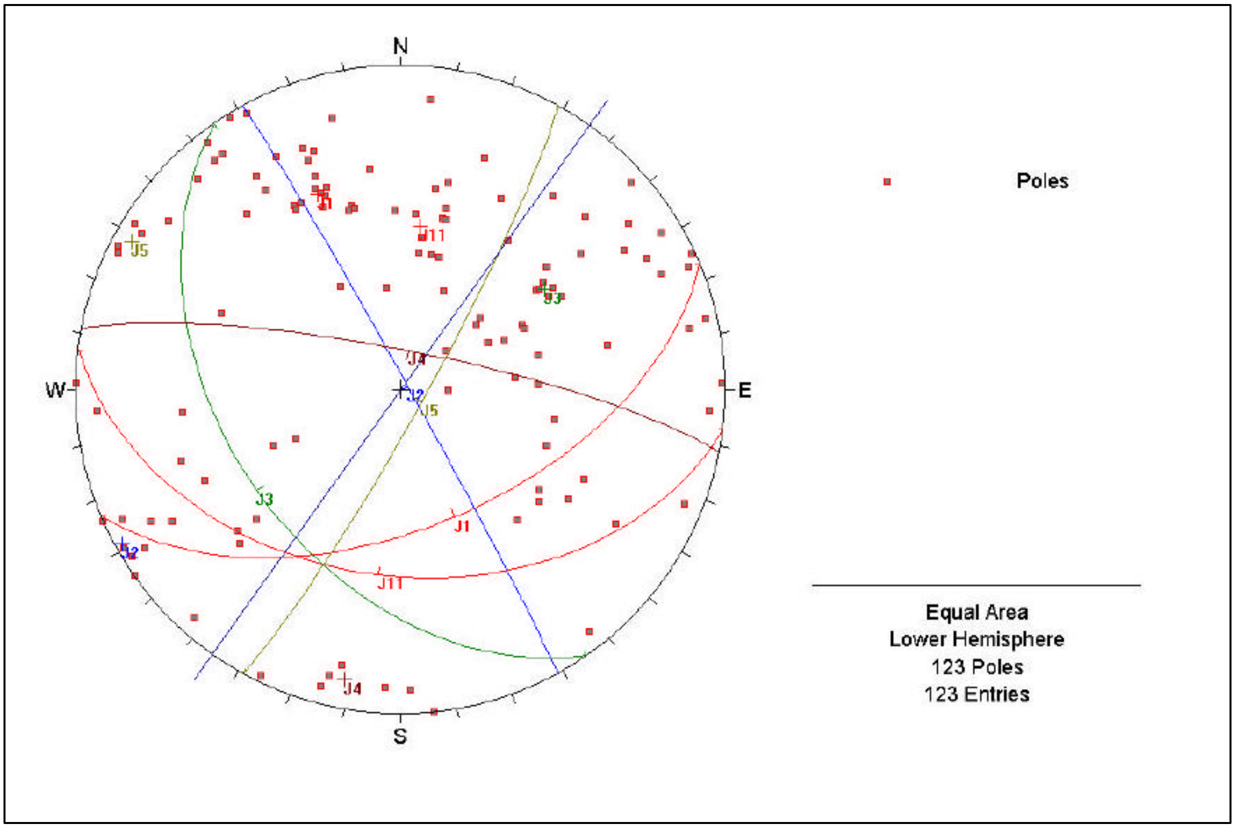
**Fig. A3.2.2:** Union Park Dam, Outcrop Areas 2 & 3, Distribution of poles to joint surfaces and pole concentration with identification of joint sets.

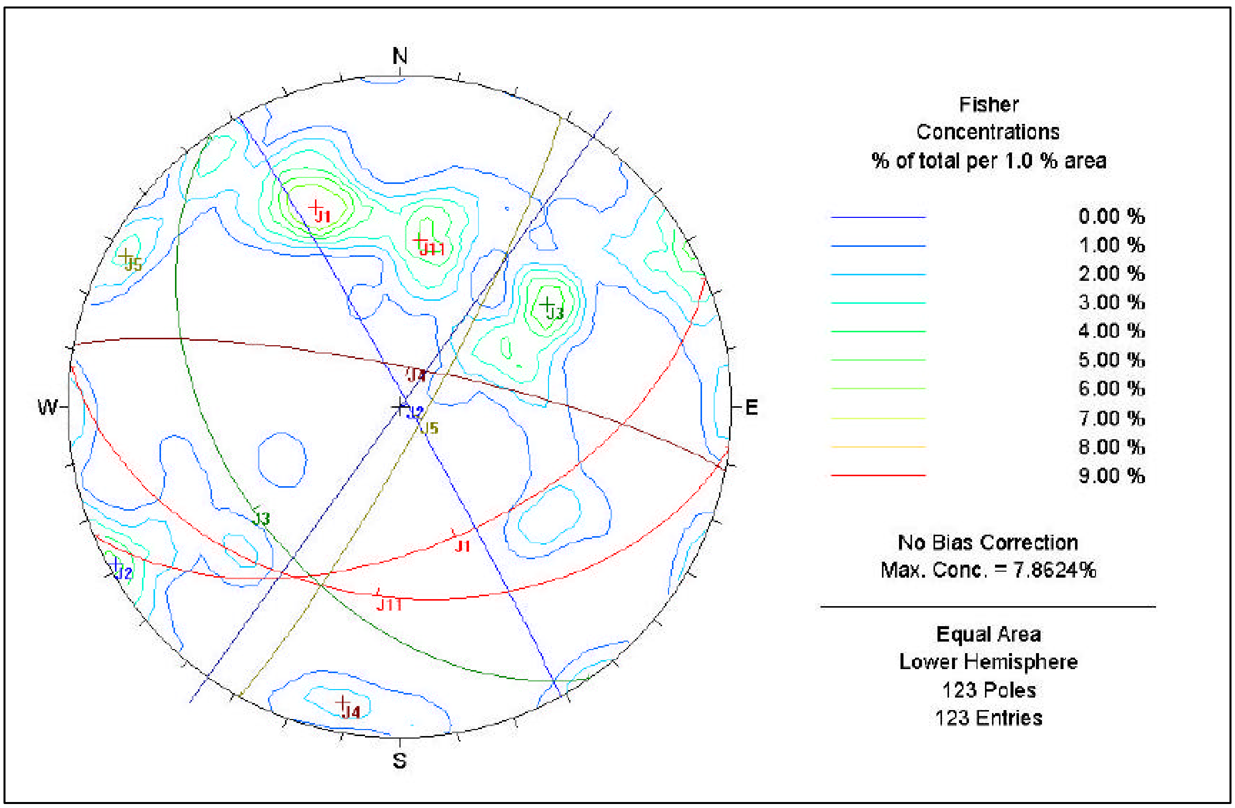



| Mean orientation of joint sets at outcrop area 4 (dip/dip direction in degrees): | | |
|---|---|---|
| J1<br>58/156 | J2<br>87/248 | J3<br>58/293 |

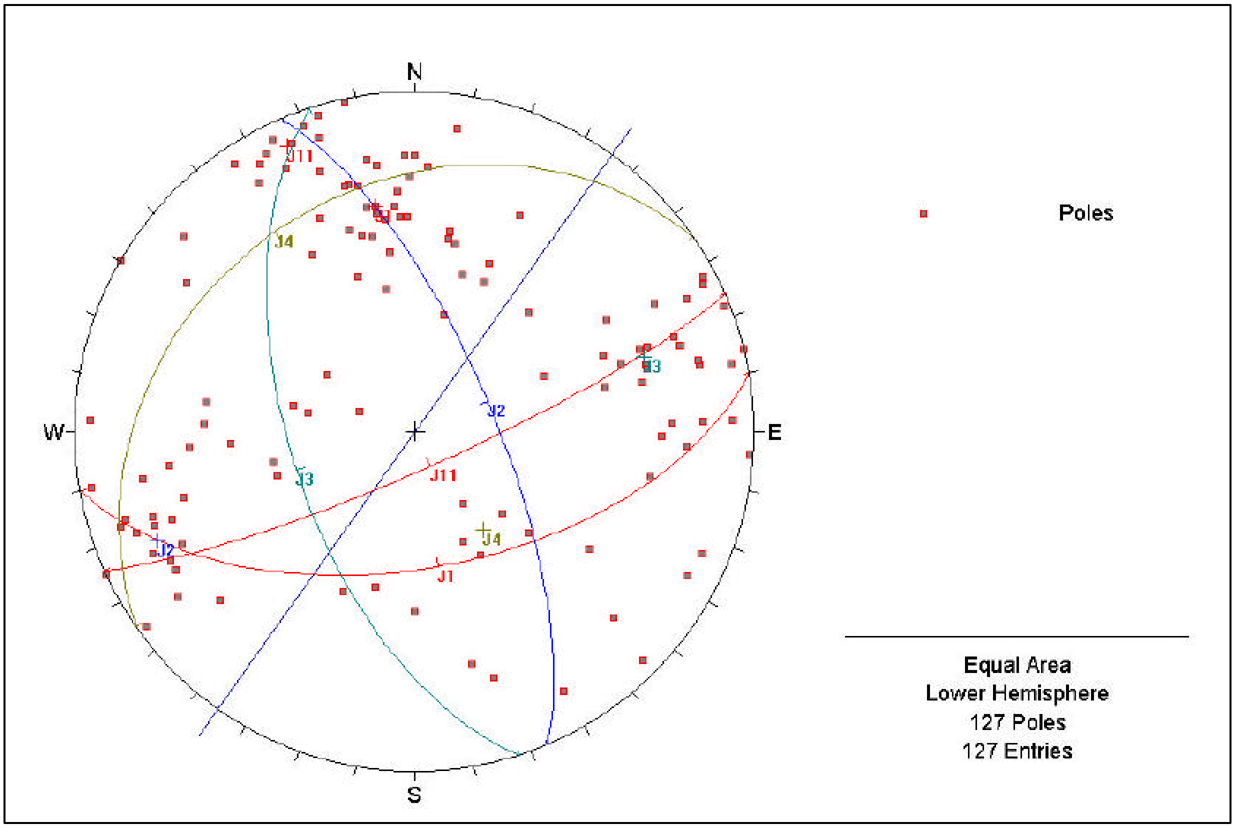
**Fig. A3.2.3:** Union Park Dam, Outcrop Area 4, Distribution of poles to joint surfaces and pole concentration with identification of joint sets.

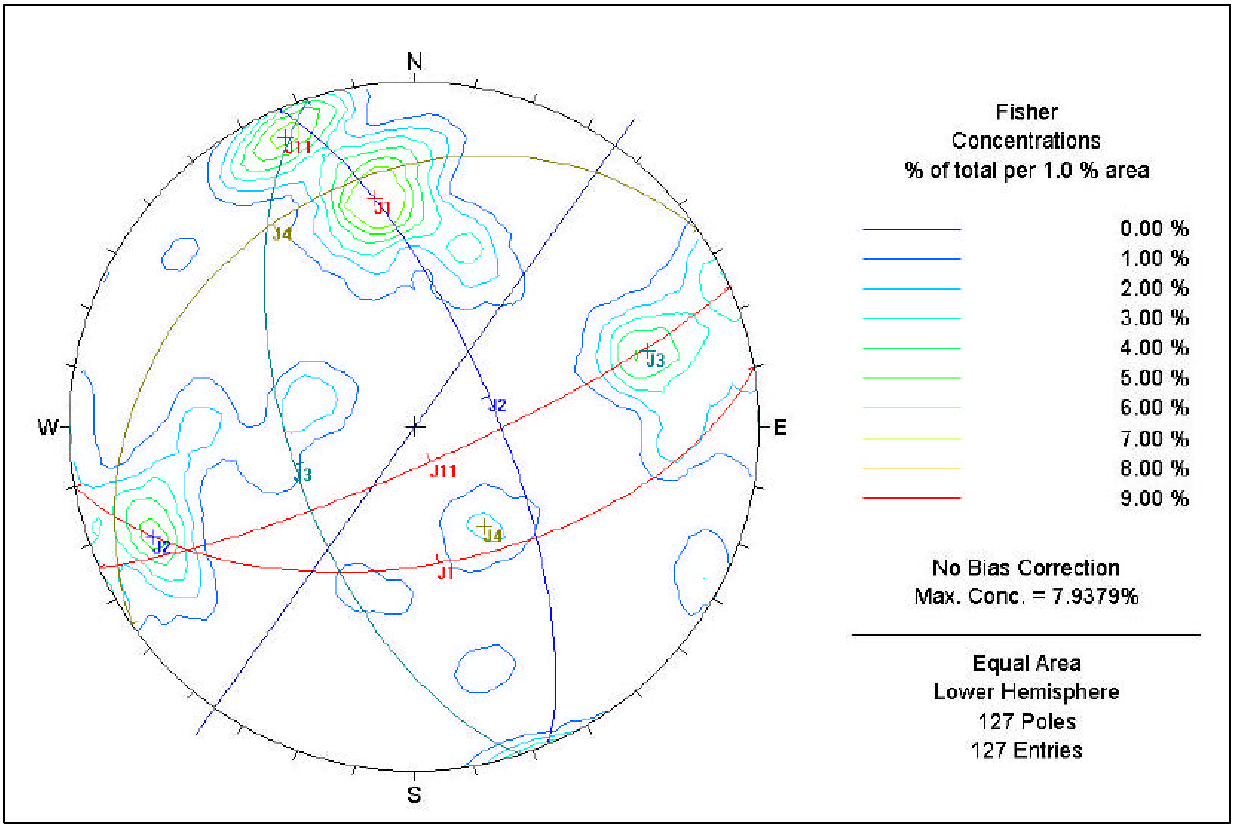



| Mean orientation of joint sets at outcrop areas 5 & 6 (dip/dip direction in degrees): | | | | | |
|---|---|---|---|---|---|
| J1<br>55/157 | J11<br>42/187 | J2<br>88/61 | J3<br>45/235 | J4<br>80/11 | J5<br>84/119 |

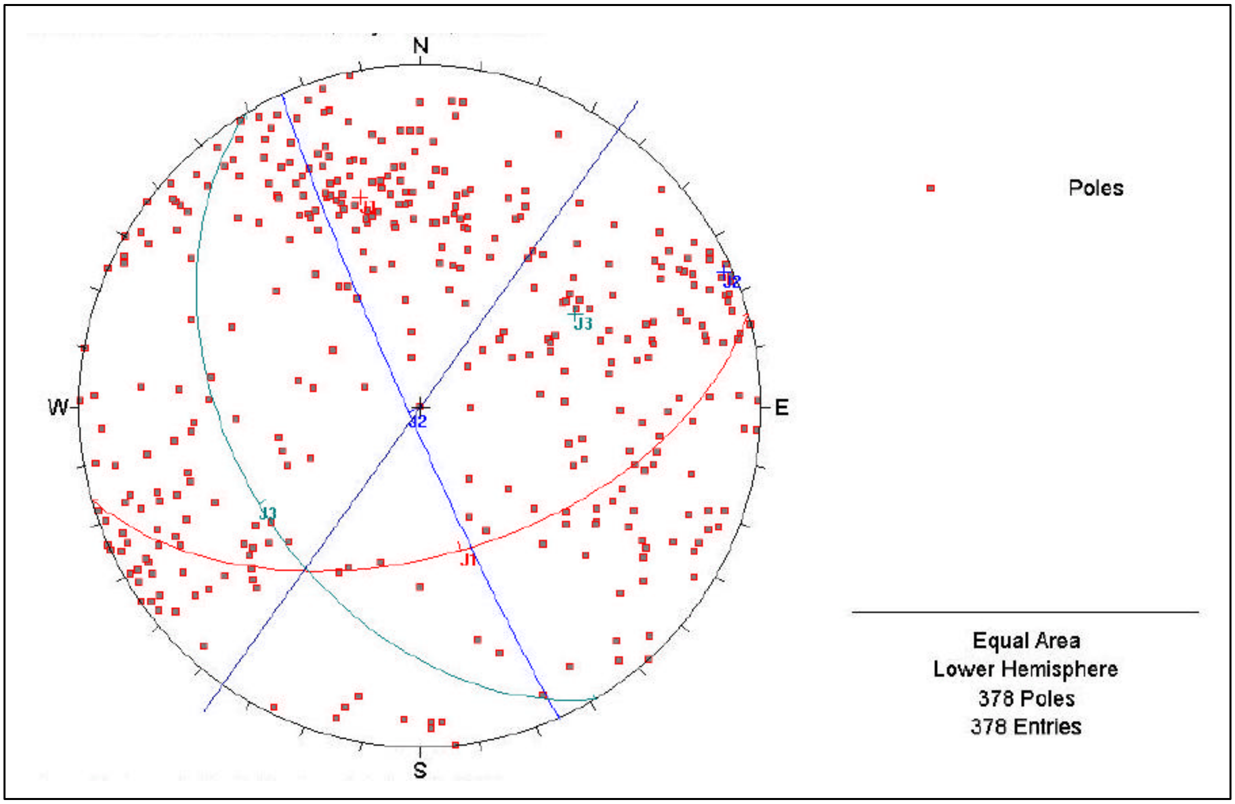
**Fig. A3.2.4:** Union Park Dam, Outcrop Areas 5 & 6, Distribution of poles to joint surfaces and pole concentration with identification of joint sets.

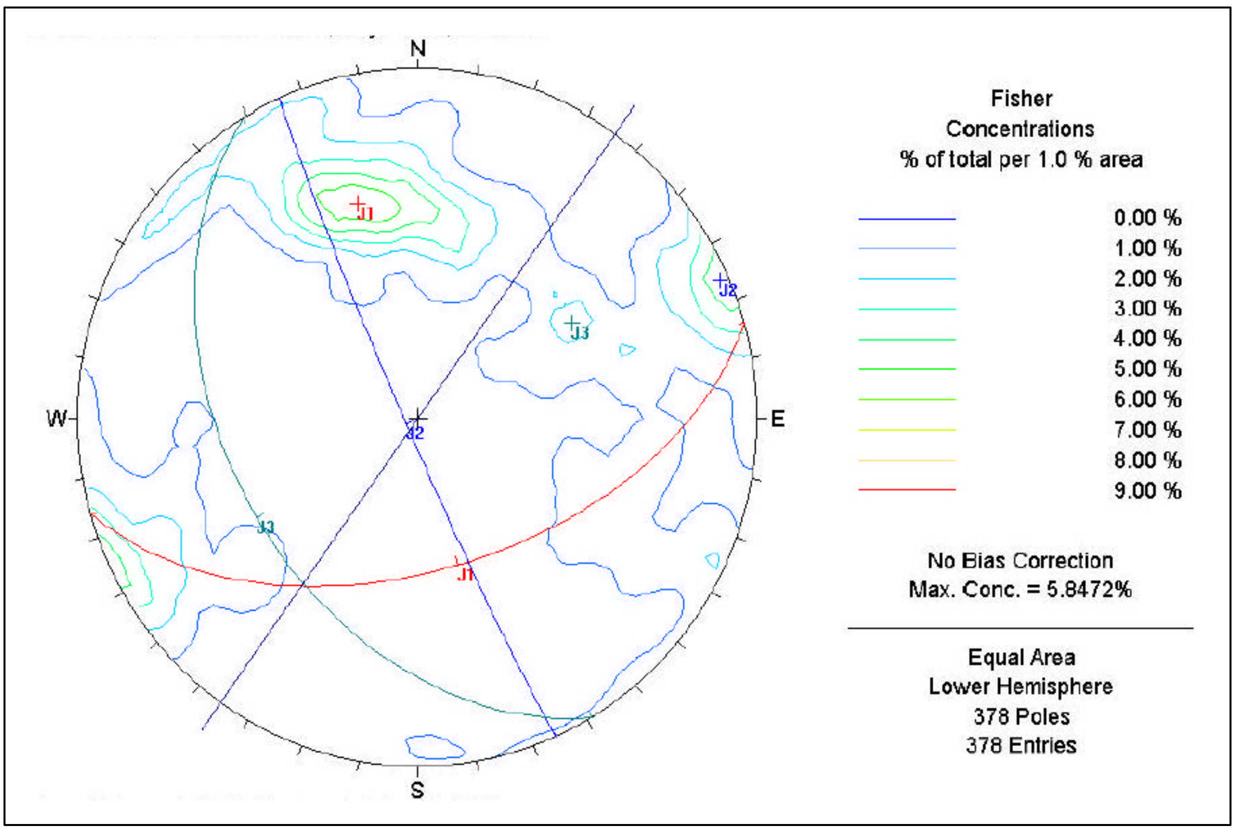



| Mean orientation of joint sets at outcrop areas 7to 10 (dip/dip direction in degrees): | | | | |
|---|---|---|---|---|
| J1 | J11 | J2 | J3 | J4 |
| 57/170 | 81/156 | 71/67 | 60/252 | 29/325 |

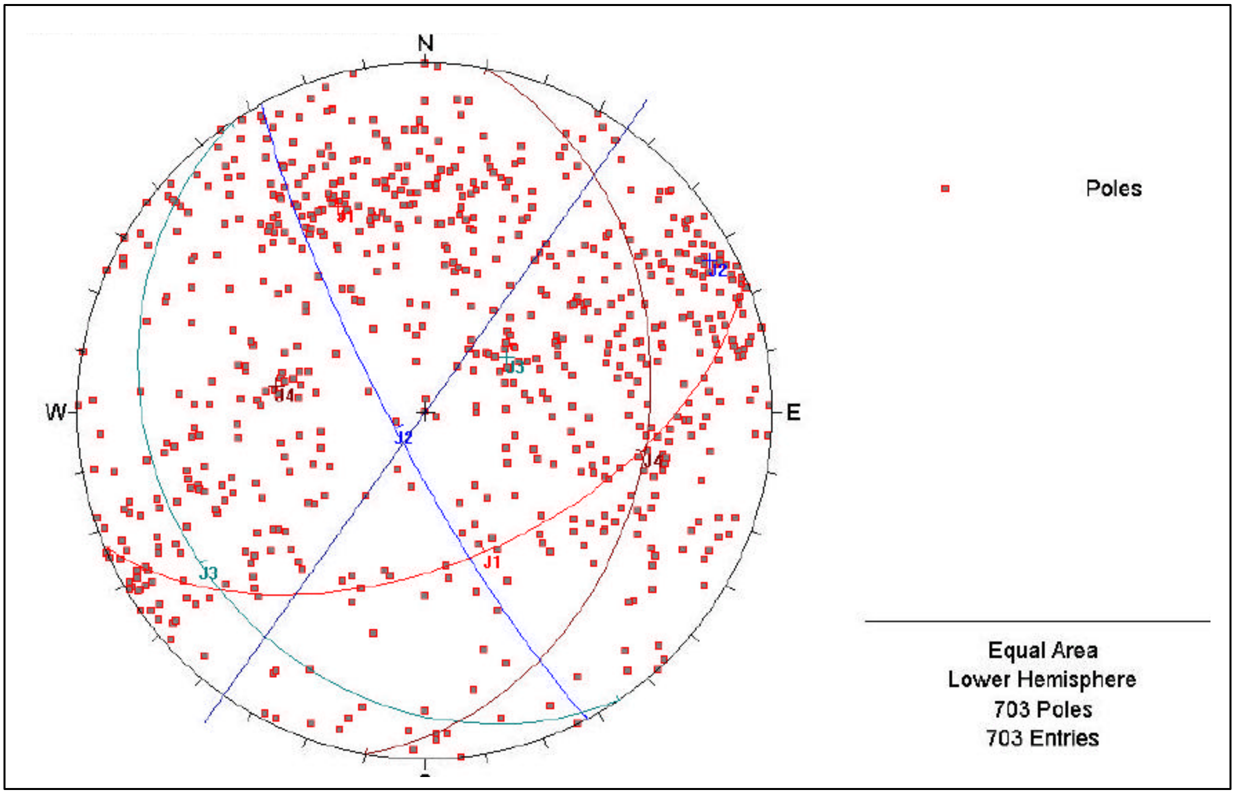
**Fig. A3.2.5:** Union Park Outcrop Areas 7,8,9,10; Distribution of poles to joint surfaces and pole concentration with identification of joint sets.

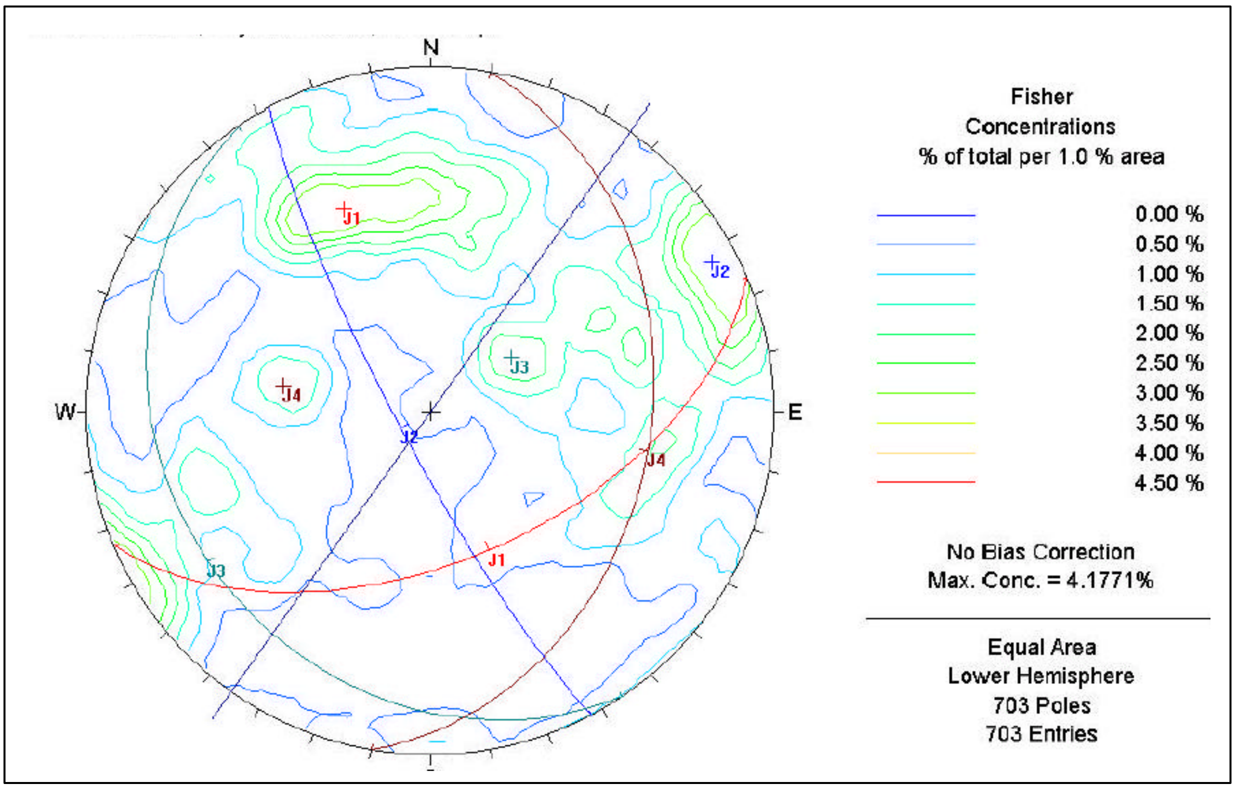



| Mean orientation of joint sets at outcrop areas 2 to 10 combined (dip/dip direction in degrees): | | |
|---|---|---|
| J1 | J2 | J3 |
| 54/164 | 87/246 | 44/239 |

**Fig. A3.2.6:** Union Park Dam, Outcrop Areas 2 to 10 combined; Distribution of poles to joint surfaces and pole concentration with identification of joint sets.




| Mean orientation of joint sets at outcrop areas 1 to 10 combined (dip/dip direction in degrees): | | | |
|---|---|---|---|
| J1 | J2 | J3 | J4 |
| 54/157 | 82/242 | 23/236 | 36/100 |

**Fig. A3.2.7:** Union Park Dam, Outcrop Areas 1 to 10 combined; Distribution of poles to joint surfaces and pole concentration with identification of joint sets.

**Table A3.2:** 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. | |

**Table A3.3:** Terms to describe structural type and surface condition [7]

| | STRUCTURE |
|---|---|
| | BLOCKY – very well interlocked undisturbed rock mass consisting of cubical blocks formed by three orthogonal discontinuity sets |
| | VERY BLOCKY – interlocked. partially disturbed rock mass with multifaceted angular blocks formed by four or more discontinuity sets |
| | BLOCKY/SEAMY – folded and faulted with many intersecting discontinuities forming angular blocks |
| | CRUSHED – partly interlocked, heavily broken rock mass with a mixture of angular and rounded blocks |

| SURFACE CONDITION |
|---|
| VERY GOOD<br>Very rough, unweathered, surface |
| GOOD<br>Rough, slightly weathered, Iron stained surface |
| FAIR<br>Smooth, moderately weathered or altered sur-faces |
| POOR<br>Slickensided, highly weathered surfaces with compact coatings of fillings containing angular rock fragments |
| VERY POOR<br>Slickensided, highly weathered surfaces with soft clay coatings or fillings |

**Table A3.4:** Terms to describe joint spacing and block size [7]

| **Term** | **Spacing mm** |
|---|---|
| Very wide | greater than 2000 |
| Wide | 2000 to 600 |
| Medium | 600 to 200 |
| Close | 600 to 60 |
| Very close | 60 to 20 |
| Extremely close | less than 20 |

| **Term** | **Average length of block edges** |
|---|---|
| Very large | greater than 2 m |
| Large | 600 mm to 2 m |
| Medium | 200 mm to 600 mm |
| Small | 60 mm to 200 mm |
| Very small | less than 60 mm |

**Table A3.5:** Terms for the description of discontinuity aperture [7]

| **Feature description term** | **Aperture size term** | **Aperture** |
|---|---|---|
| “ Closed” features | Very fight | Less than 0.1 mm |
| | Tight | 0.1 to 0.25 mm |
| | Partly open | 0.25 to 0.5 mm |
| “ Gapped” features | Open | 0.5 to 2.5 mm |
| | Moderately wide | 2.5 to 10 mm |
| | Wide | Greater than 10 mm |
| “Open” features | Very wide | 1 to 10 cm |
| | Extremely wide | 10 to 100 cm |
| | Cavernous | Greater than 1 m |

**Table A3.6:** Extent of joints and dimension of rock blocks

| **Term** | **Extent of discontinuity** | **Term** | **Average length of block edges** |
|---|---|---|---|
| Small | < 1 m | Very large | Greater than 2 m |
| Large | 1 – 10 m | Large | 600 mm to 2 m |
| Very large | 10 – 100 m | Medium | 200 mm to 600 mm |
| Extremely large | > 100 m | Small | 60 mm to 200 mm |
| | | Very small | Less than 60 mm |

**Table A3.7:** Non-oriental features of rock mass and discontinuities

| Outcrop Area | Structural type | Surface condition of joints | Joint spacing | Joint aperture | Extent of joints / block size | Comments |
|---|---|---|---|---|---|---|
| 1 | blocky – very blocky | fair occasion. Poor | medium to close | tight, partly open | small / medium | rock mass is intensively dissected, in part of the area many internal fractures with coating; quartz veined fracture zone parallel to J2 joints |
| | very blocky | | close to very close | partly open to open | small / very small | |
| 2 & 3 | blocky, locally very blocky | Fair | medium | tight to partly open | small to large/ medium to large | quartz veins and fracture-zone parallel to J2 through middle of outcrop area some joints of large extent |
| 4 | blocky, locally very blocky | fair | medium | tight to partly open | small to large/ medium to large | some large joints of considerable continuity parallel to J2 (fracture zones) and to J1 joint sets; foliation parallel in strike to joint set J2 |
| 5 & 6 | blocky, locally very blocky | fair | medium to wide | tight to partly open | small to large/ medium to very large | |
| 7, 8, 9,10 | blocky, locally very blocky | fair | medium to very wide, locally close to very close | tight to partly open | small to very large/ medium to very large | quartz veins and fracture zones parallel to joint set J2; some very large joints (sets J2 and J1) foliation parallel in strike to joint set J2 |

# Union Park Dam

# Appendix A 4

# Photographic Documentation

Photographs referenced in the report:

Photo 2.1 Upper reaches of Union Canyon downstream view; talus material and alluvial deposits, historic mining activities.

Photo 2.2 Recent rock fall material

Photo 2.3 Large scale blocks from recent rock fall, Outcrop Area 5.

Photo 2.4 Foliation and minor fold structures in gneissic rock

Photo 2.5 Closely spaced fractures; Outcrop Area 1.

Photo 2.6 Closely spaced fractures, Outcrop Area 1.

Photo 2.7    Joints of joint sets J1 and J2, Outcrop Area 9

Photo 2.8    Large scale joints, joint set J1, Outcrop Area 8

Photo 2.9 Joints of set J3 dipping towards the valley and opening of joints of set J2 due to gravitational effects on steep cliffs.

Photo 2.10 Avalanche and debris flow channel on left valley slope in uppermost part of Union Canyon

Photo 2.11 Avalanche and debris flow channel, close to mouth of Union Canyon.

Photo 2.12 View of Union Park reservoir area.

Photo 2.13 View of Union Park reservoir area.

Photographs related to Appendix A3:

Photo A3.1 Joint sets J1 and J2 with large scale joints.

Photo A3.2 Major sets J2, J3 and J1.

Photo A 3.3 Large scale joints, major joint set.

# Union Park Dam

# Appendix A5

# Preliminary Earthquake Hazard Evaluation

# Preliminary Earthquake Hazard Evaluation

Ueblacker Associates completed a preliminary study to evaluate the seismic hazard for the proposed Union Park Dam. The purpose of the study was to determine the Maximum Credible Earthquake (MCE) for design of Union Park Dam, and obtain estimates of the mean horizontal and vertical peak ground acceleration (PGA) generated by the MCE at the site of the dam, and estimates of the dam's acceleration response.

## U.S. Bureau of Reclamation (USBR) Seismotectonic Study

In 1998, the USBR completed a seismotectonic study for the existing Taylor Park Dam, which is located in close proximity to the proposed Union Park Dam. The Bureau's study includes all pertinent information on geologic structures and the seismicity of the region used in this evaluation.

The USBR concluded that Taylor Park Dam could be subjected to moderate to strong, earthquake–induced ground motions.

As in the USBR study, our evaluation focused on potential seismic sources that might produce significant ground shaking at the site of the proposed Union Park Dam, including both late Quaternary fault sources and background or random seismicity not associated with known surface faults.

Our study included the two major late-Quaternary faults in the Rio Grande Rift, the southern Sawatch fault and the northern Sangre de Cristo fault [24], which have potential significance to seismic hazard analysis of the dam. However, although the faults are considered capable of generating earthquakes in the magnitude range Mw 7.0-7.25, they are located at considerable distances from the site (35 and 75 km) thereby reducing the effects of ground shaking.

The USBR performed a probabilistic seismic hazard analysis to estimate earthquake loading parameters for Taylor Park Dam, incorporating fault and random sources. The effect of the Sangre de Cristo fault on the hazard at Taylor Park Dam was found to be negligible; therefore, only contributions of the Sawatch fault were considered in the final results.

In addition to earthquakes that occur on mapped faults, earthquakes not associated with known geologic structures also present hazards. Colorado is a region of low to moderate earthquake activity. The existing Taylor Park Dam and the proposed Union Park Dam are in the western mountain seismotectonic province of Colorado, which is described as an area of few late-Cenozoic faults, but a relatively high rate of occurrence of felt and/or instrumentally recorded earthquakes compared to other parts of Colorado.

In the USBR study, an area encompassing the Southern Rocky Mountain province of western and central Colorado and northern New Mexico was defined for seismicity investigations of Taylor Park Dam. Mw 6.5 was selected as the maximum magnitude of the background earthquake, based on observations of source parameters of earthquakes in the western US in the magnitude range of 6.0-6.5. A data set of 38 events identified as independent after declustering, elimination of explosions, and correction for completeness was used to calculate recurrence parameters. The calculated recurrence interval for Mw 6.0-6.5 earthquakes in the study area is 500 years, with upper and lower 95% confidence bounds of 1750 and 143 years, respectively.

Hazard curves for peak horizontal acceleration and 1-second spectral acceleration were developed for the Sawatch fault and for random seismicity, incorporating uncertainties in attenuation and seismicity rates, for three magnitude intervals: Mw 5.0-6.5, 5.0-6.0, and 6.0-6.5. It was found that background seismicity dominates the hazard at Taylor Park Dam for all return periods calculated; the fault source (Sawatch fault) has only a very small effect on the total hazard. Therefore, only the random source has been considered in developing design ground motions. The mean horizontal peak ground acceleration value '(Ah)-mean' for Taylor Park Dam for earthquakes in the magnitude range of 5.0-6.5 (return period of 50,000 years) is <u>0.38g</u>.

## MCE and Response Spectrum for Union Park Dam

In developing the response spectrum for Union Park Dam (Fig. A5.1), we found that a random earthquake of magnitude Mw 6.0 or 6.5 at a source-to-site distance of R = 5.0 or 7.7 km respectively (Fig. A5.2), will subject the structure to a mean horizontal peak ground acceleration of 0.38g. As illustrated in the attached spreadsheet calculations (Table A5.1 and A5.2) and graph showing the acceleration response spectrum developed for Union Park Dam (Fig. A5.1), the resulting mean horizontal spectral or mean horizontal dynamic response acceleration '(SAh)-mean' within the vibration period range of 0.1-0.5 seconds for concrete dams, can be as high as <u>0.75g</u>.

The dynamic finite element analysis (Appendix 6) was conducted using 0.38g and 0.19g for the horizontal and vertical components of the mean PGA, respectively. Considering the effects of damping due to interaction of the dam with the ground and reservoir during an earthquake, natural periods of vibration of >0.5 seconds have been calculated for the dam resulting in lower dynamic response acceleration values used in the analysis [25]. The spread sheet calculations (Table A5.1 and A5.2) for determining ground motion parameters are based on new attenuation relationships developed by Ken Campbell [26]. The simplified fault model used with the spread sheet calculations is illustrated in Fig. A5.2. Campbell's most recent equations [27] will be used in completing Phase 2 of this study, which may lead to structural optimization of the main RCC gravity dam and savings in concrete.

We believe that the decision made by the USBR to select a Mw 6.5 as the maximum magnitude of the background earthquake for developing design ground motions for Taylor Park Dam is reasonable and applies equally to Union Park Dam. This decision is based on observations of earthquakes in the magnitude range of 6.0-6.5, which generally have stress drops between 30 and

100 bars and fault dips of about 60 degrees. Given these parameters and the maximum seismogenic depth in the western US of 12-15 km, it is possible for events of Mw 6.5 to occur without intersecting the surface. Larger magnitude earthquakes in general cause surface ruptures. As may be observed along the southern section of the Sawatch fault and northernmost segment of the Sangre de Cristo fault, Colorado‘s most active fault, these features can be 29 km and 90 km long, respectively [24].

As of the completion of this study, additional site specific, geologic, geotechnical, and hydrologic information is needed to develop the design of a safe dam and water storage impoundment for Union Park Reservoir. The seismic loading parameters developed in this study suggest that Union Park Dam could be subjected to moderate to strong earthquake-induced ground motions. However, we believe that the estimated ground motion parameters appear to be reasonable and their anticipated effects on the stability of the dam can be effectively accommodated during the detailed design process. Also, there are no known active faults in close proximity to the dam that would render the structure unsafe.

## Table A5.1

**Estimate of Earthquake Loading for Union Park Dam for Maximum Credible Earthquake *(MCE) of Magnitude Mw 6.5; Source: Random* Earthquake; Source-to-Site Distance: R= 7.70 km, Rseis=8.23 km. Estimate based on Empirical Near-Source Attenuation Relationship for Horizontal Component of Peak Ground Acceleration (Ah), Kenneth W. Campbell, 1997***

*Spreadsheet Calculations by Horst Ueblacker, P.E., Ueblacker Associates, Lakewood, Colorado, July 31, 2003*

***Variables:***

| | | |
|---|---|---|
| Earthquake magnitude (M=Mw) | **6.50** | |
| Closest distance to vertical projection of fault rupture (R) | **7.70** | Km |
| Top seismogenic crust (Htop) | **3.00** | km |
| Bottom seismogenic crust (Hbot) | **15.00** | km |
| Dip of fault plane (alpha) | **60.00** | degrees |
| Selected down-dip dimension of fault rupture (W)select | **11.75** | km |
| Selected average depth to seismogenic rupture (min.2-4km) (Dseis)select | **3.00** | km |
| Style of faulting (F) | **0.50** | |
| Local site conditions (Ssr) | **0.00** | |
| Local site conditions (Shr) | **1.00** | |

F=0 for strike-slip faulting;
F=1 for reverse, thrust faulting;
F=1 for reverse-oblique & thrust-oblique faulting;
F=0.5 for normal faulting.
Ssr, Shr =0 for alluvium/firm soil;
Ssr=1, Shr=0 for soft rock;
Ssr=0, Shr=1 for hard rock.

Horizontal Component of PGA (Ah):
ln(Ah)=A+B+C+D+E, Ah in units of g (g=981cm/sec^2)
A=-3.512+(0.904*M)
B=-1.328*LN((Rseis^2+(0.149*EXP(0.647*M))^2)^0.5)
C=(1.125-0.112*LN(Rseis)-0.0957*M)*F
D=(0.440-0.171*LN(Rseis))*Ssr
E=(0.405-0.222*LN(Rseis))*Shr

Standard Error Estimate of ln(Ah):

| | | |
|---|---|---|
| M<7.4, sigma=0.889-(0.0691*M) | sig1 | 0.440 |
| M>=7.4, sigma=0.38 | sig2 | **0.380** |

Shortest Distance to Top of Seismogenic Rupture
(Rseis) = SQRT(R^2+(Dseis)select^2) 8.26 km

Selected Average Depth to Seismogenic Rupture
when (Dseis)select=>Htop:
(Dseis)select = 1/2*(Hbot-Htop-(Wselect*sinAlpha))*Htop
2.74 km

***Computation:***

Computed Averages of (W) and (Dseis)

(W)=!0^(-1.01+(0.32*M)) 11.75 km

(Dseis)=1/2*(Hbot -Htop-(W*sinALPHA))*Htop 2.74 km

| A | B | C | D | E | ln(Ah) | (Ah)mean | |
|---|---|---|---|---|---|---|---|
| 2.364 | -3.40275 | 0.13321 | 0.00 | -0.06 | -0.96938 | 0.38 | g-units |
| | | | | | | 372.11 | cm/sec^2 |

| ln(Ah)+sig1 | ln(Ah)+sig2 | (Ah)1max | (Ah)2max |
|---|---|---|---|
| -0.52953 | -0.589 | 0.59 | 0.55 |
| **ln(Ah)-sig1** | **ln(Ah)-sig2** | **(Ah)1min** | **(Ah)2min** |
| -1.40923 | -1.349 | 0.24 | 0.26 |

*Seismological Research Letters Volume 68, Number 1 January/February 1997, p. 156,164

## Table A5.2

**Estimate of Earthquake Loading for Union Park Dam for Maximum Credible Earthquake (MCE) of Magnitude Mw6.5; Source: Random Earthquake; Source-to-Site Distance: R=7.70 km, Rseis=8.26 km. Estimate based on Empirical Near-Source Attenuation Relationship for Horizontal Component of 5% - Damped Pseudo-Absolute Acceleration (Sah), Kenneth W. Campbell, 1997*.**

*Spreadsheet Calculations by Horst Ueblacker, P.E., Ueblacker Associates, Lakewood, Colorado, July 24, 2003*

***Variables:***

| | | |
|---|---|---|
| Earthquake magnitude (M=Mw) | **6.50** | |
| Source-to-site distance (Rseis) | **8.26** | km |
| Style of faulting (F) | **0.50** | |
| Dip of fault plane (alpha) | **60.00** | degrees |
| Local site conditions (Ssr) | **0.00** | |
| Local site conditions (Shr) | **1.00** | |
| Depth to basement rock (D) | **0.00** | km |
| fSA(D) | **0.0000** | |

F=0 for strike-slip faulting;
F=1 for reverse, thrust faulting;
F=1 for reverse-oblique & thrust-oblique faulting;
F=0.5 for normal faulting.
Ssr, Shr =0 for alluvium/firm soil;
Ssr=1, Shr=0 for soft rock;
Ssr=0, Shr=1 for hard rock.

Horizontal Component of PSA (SAh):
ln(SAh)=A+B+C, SAh in units of g
A=LN(Ah)+C1+C2*(TANH(C3*(M-4.7))
B=(C4+(C5*M))*Rseis+(0.5*C6*Ssr)+(C6*Shr)
C=(C7*TANH(C8*D))*(1-Shr)+fSA(D)

where when:

| | | |
|---|---|---|
| D=>1 km: | fSA(D)=0 | |
| D<1 km: | fSA(D)=(C6*(1-Shr)*(1-D))+(0.5*C6*(1-D)*Ssr) | **0.0000** |

| | | |
|---|---|---|
| Mean horizontal component of PGA (Ah) | **0.38** | g |
| Period | **0.15** | sec |

| | | | |
|---|---|---|---|
| C1 | **0.72** | C5 | **-0.00027** |
| C2 | **0** | C6 | **-0.02** |
| C3 | **0** | C7 | **0** |
| C4 | **-0.001** | C8 | **0** |

Standard Error Estimate of ln(SAh):

| | | |
|---|---|---|
| M<7.4: | sig1=((0.889-(0.0691*M))^2+0.27^2)^0.5 | 0.516 |
| M=>7.4: | sig2=((0.38)^2+(0.27)^2)^0.5 | 0.466 |

***Computation:***

| A | B | C | ln(SAh) | (SAh)mean | |
|---|---|---|---|---|---|
| -0.2476 | -0.0428 | 0.0000 | -0.2903 | 0.75 | g-units |
| | | | | 733.80 | cm/sec^2 |

| ln(SA)+sig1 | (SAh)1max | ln(SA)+sig2 | (SAh)2max |
|---|---|---|---|
| 0.2258 | 1.2533 | 0.1758 | 1.1922 |
| ln(SA)-sig1 | (SAh)1min | ln(SA)-sig2 | (SAh)2min |
| -0.8064 | 0.4464 | -0.7565 | 0.4693 |

*Seismological Research Letters Volume 68, Number 1 January/February 1997, p.170-171

**Fig. A5.1: Response Spectrum for Random (MCE) Earthquakes**
**Mw 6.0, R = 5.0 km and Mw 6.5, R = 7.7 km**

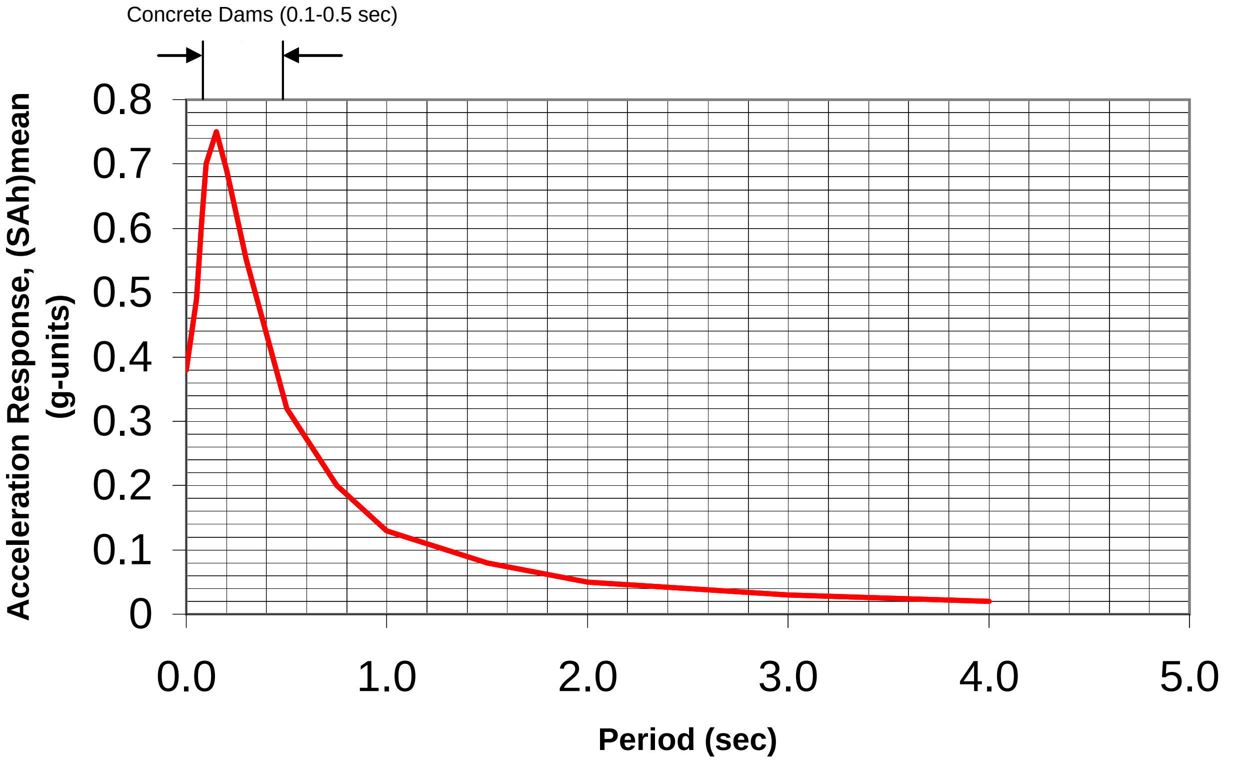
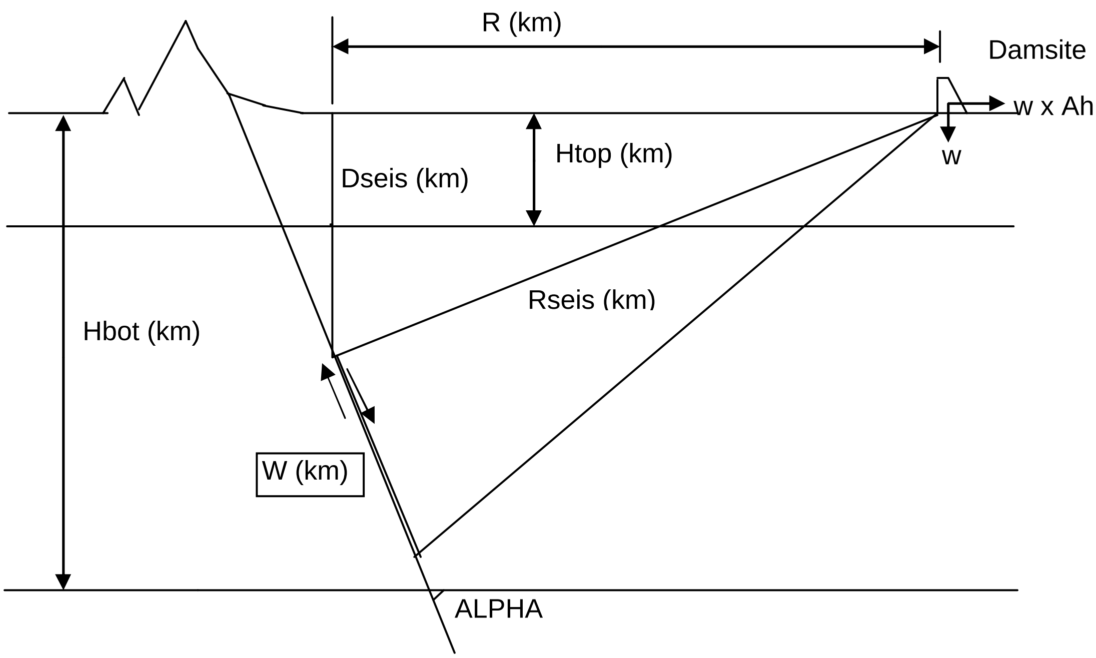


**Fig. A5.2: Illustration of Simplified Fault Model used in Estimating Earthquake Loading Parameters for Union Park Dam**




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

*Horst Ueblacker, P.E.*
*18-Feb-04*

# Union Park Dam

# Appendix A6

# Stability Evaluation and Conceptual Design Of Main RCC Gravity Dam

**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), [20] |
| Average P-Wave Velocity Left Abutment | Vp | = | 4,252 m/s |
| | Em | = | 17.81 GPa |
| | Em | = | 2,582,410 psi |

(see Fig. 3.3: 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) = <u>1.80E-05</u>

### 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), <u>Sa = 0.578 g</u>, (for Ground A [25]: 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; (see [25] Felsbau 14, 1996, Nr. 5, page 260-261).

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

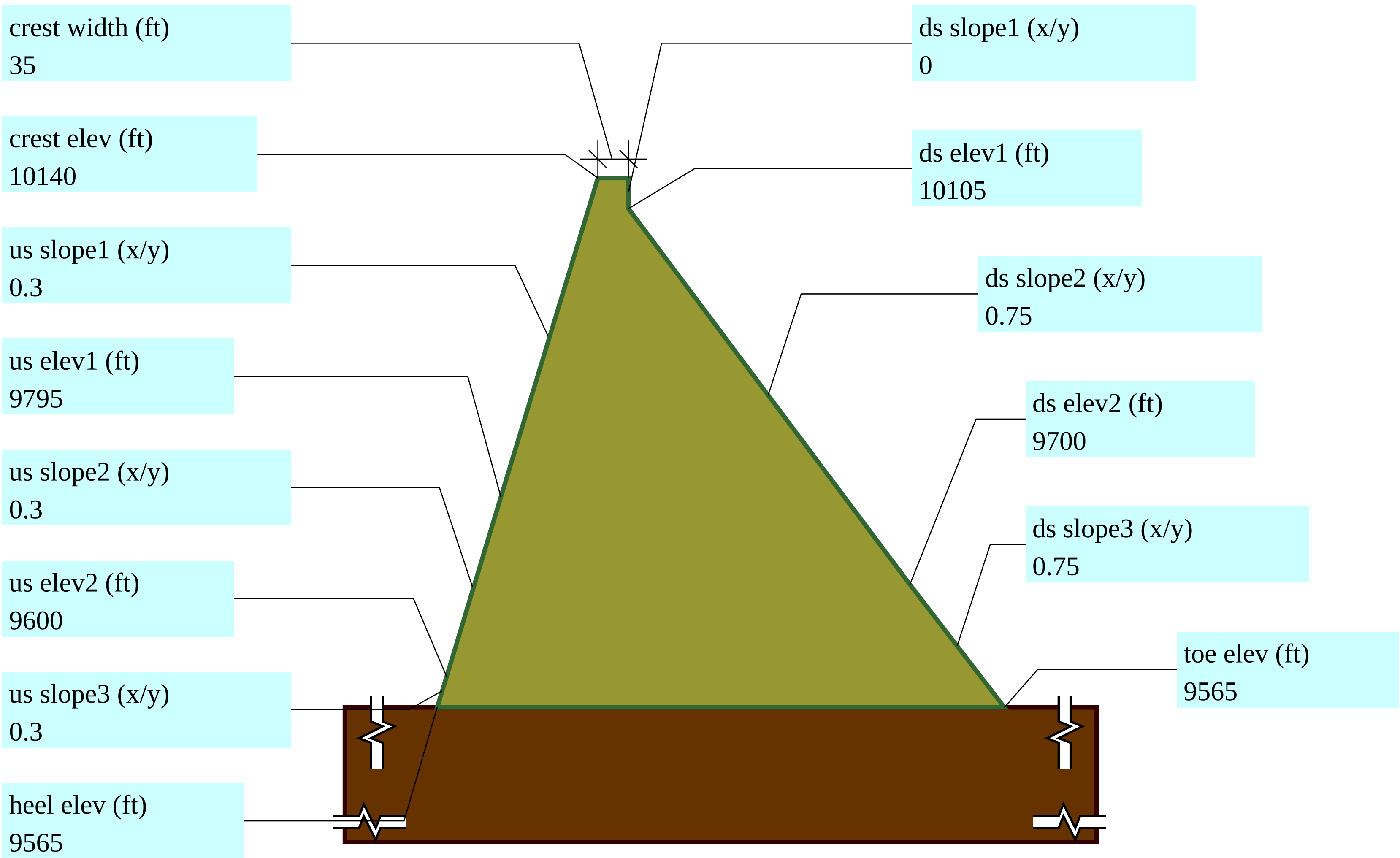

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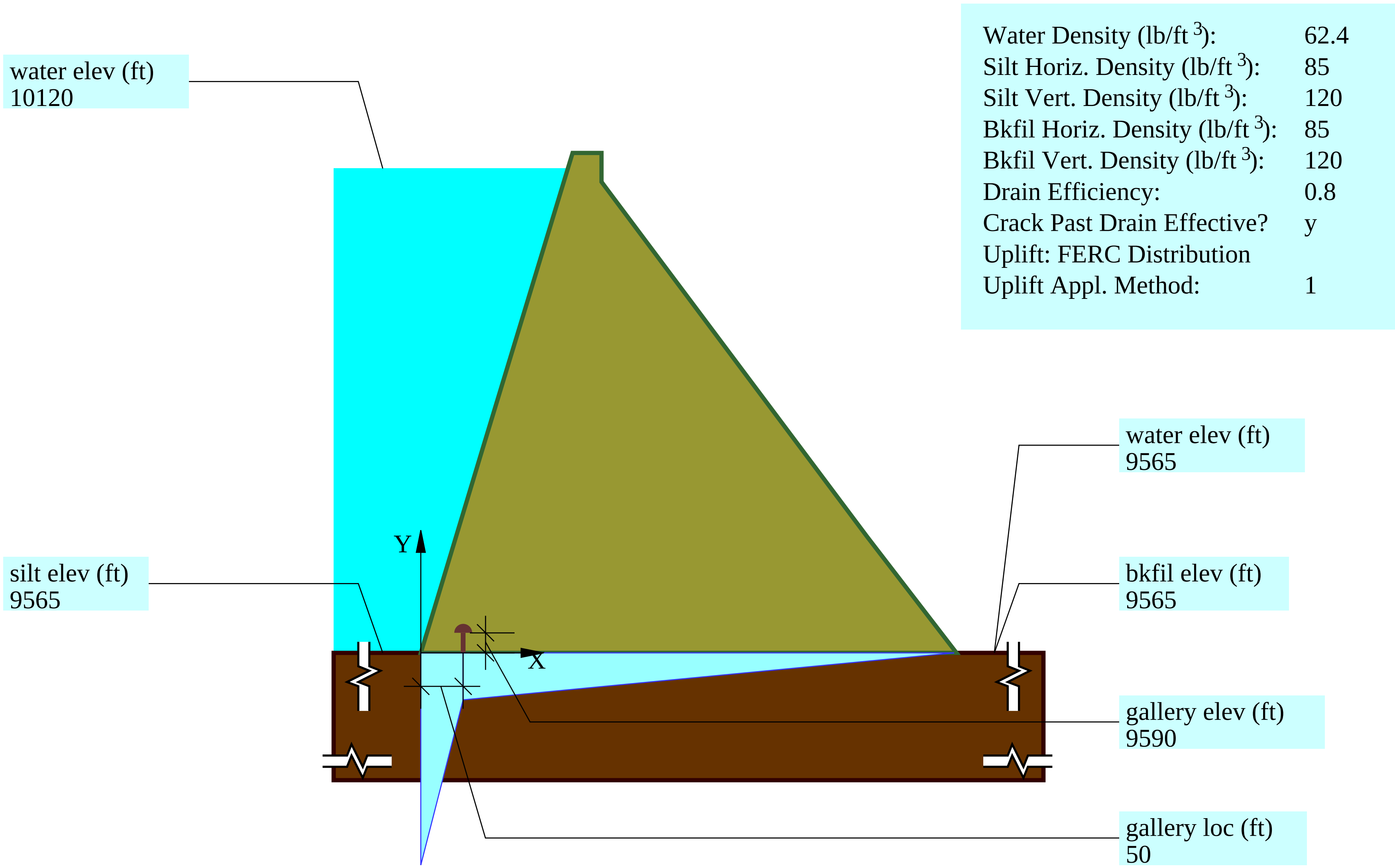
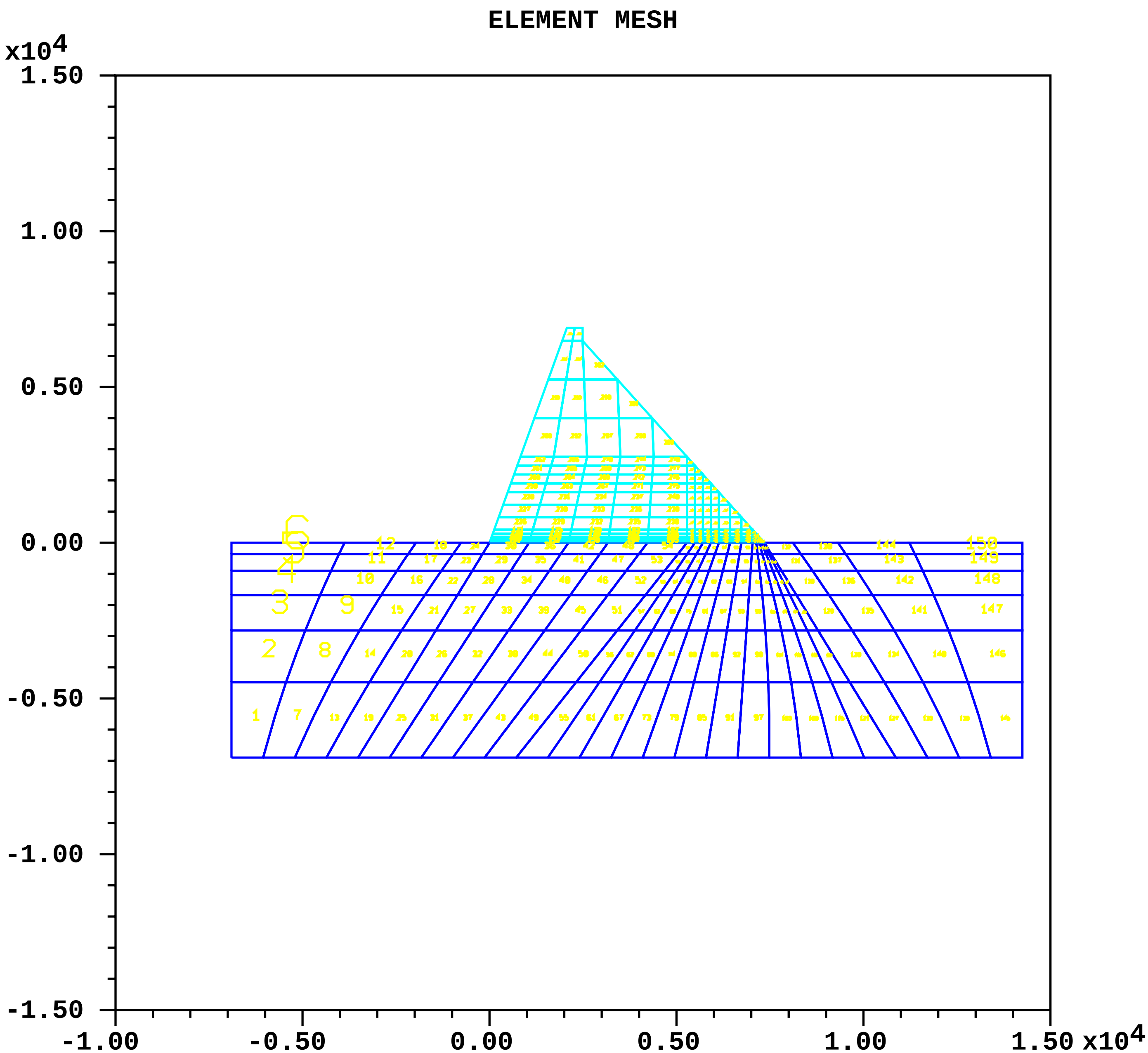
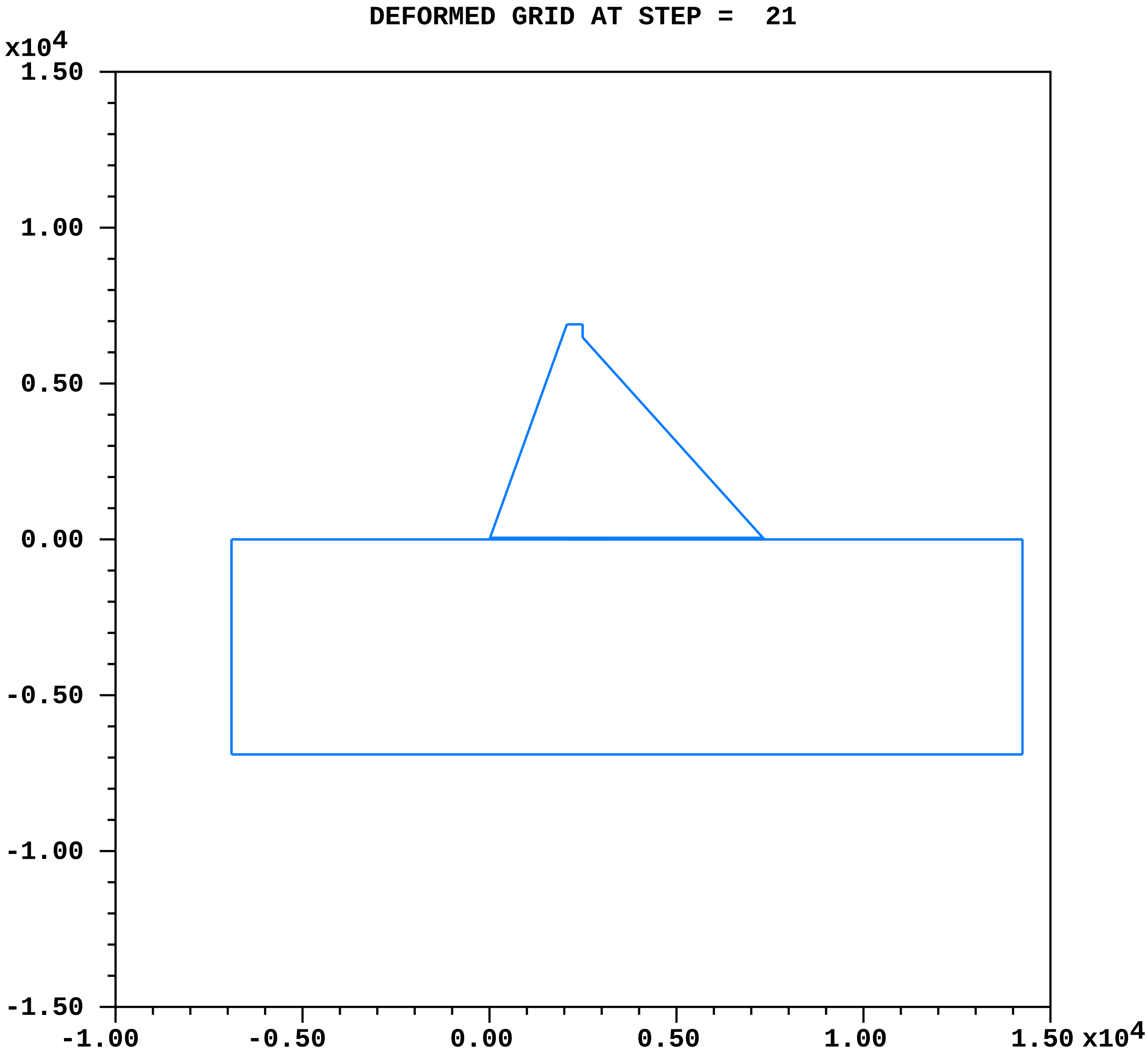
Fig. A6.2: Union Park Dam Modified Non-overflow Section w/Hydrostatic Loading






Frame 1 TAPE50

CG-DAMS PC-2.2.0 02/27/04 11:49:13.63

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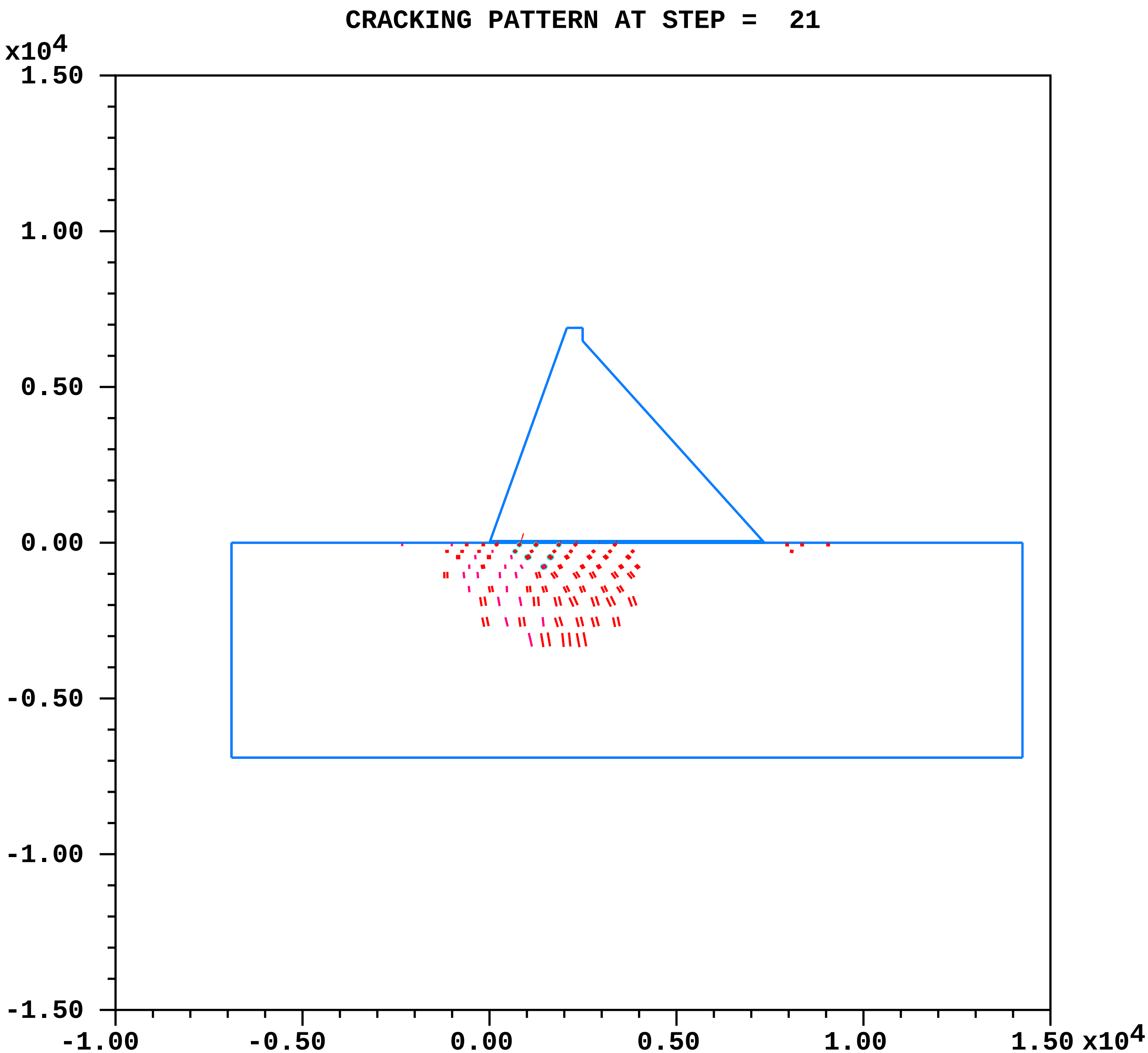


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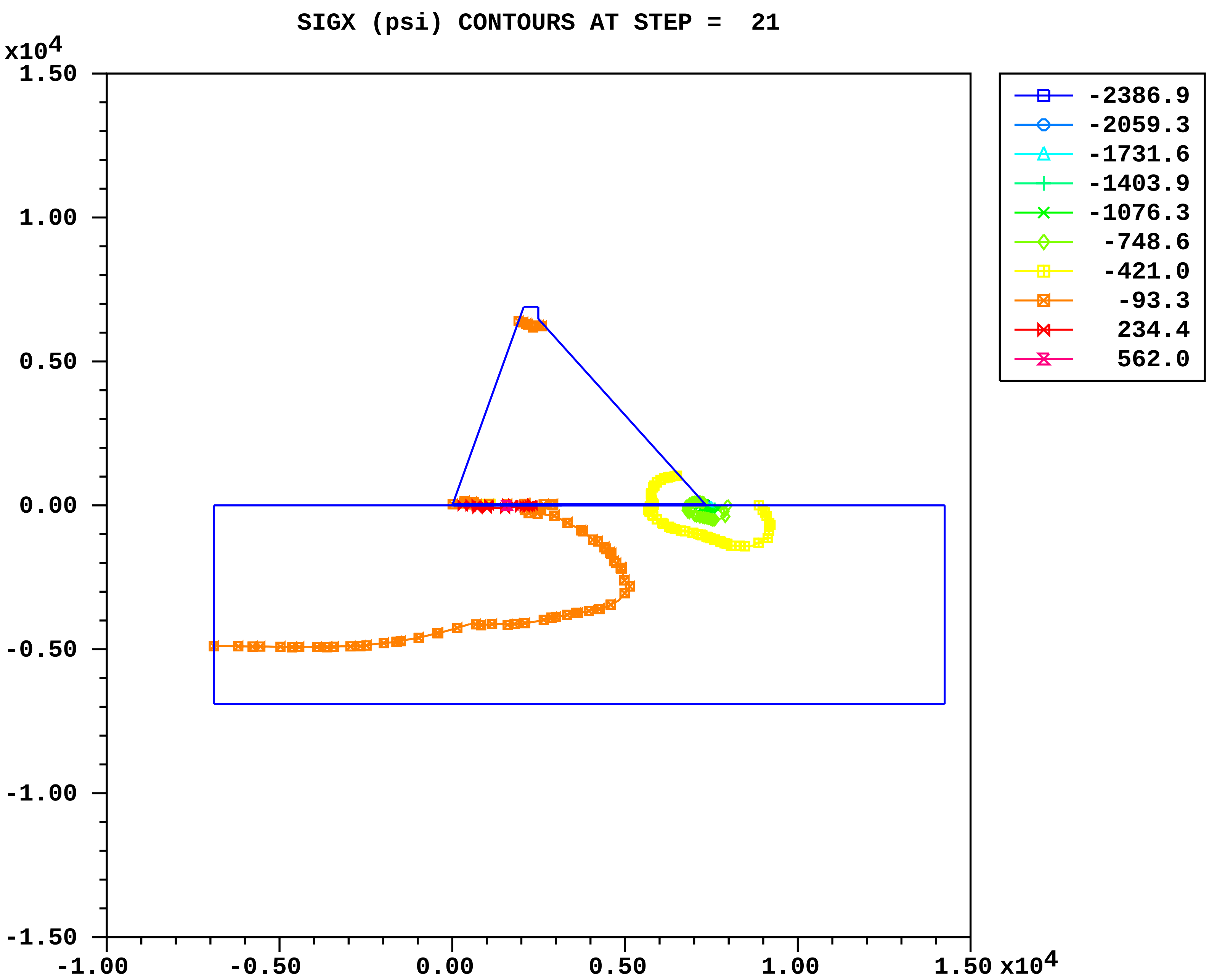
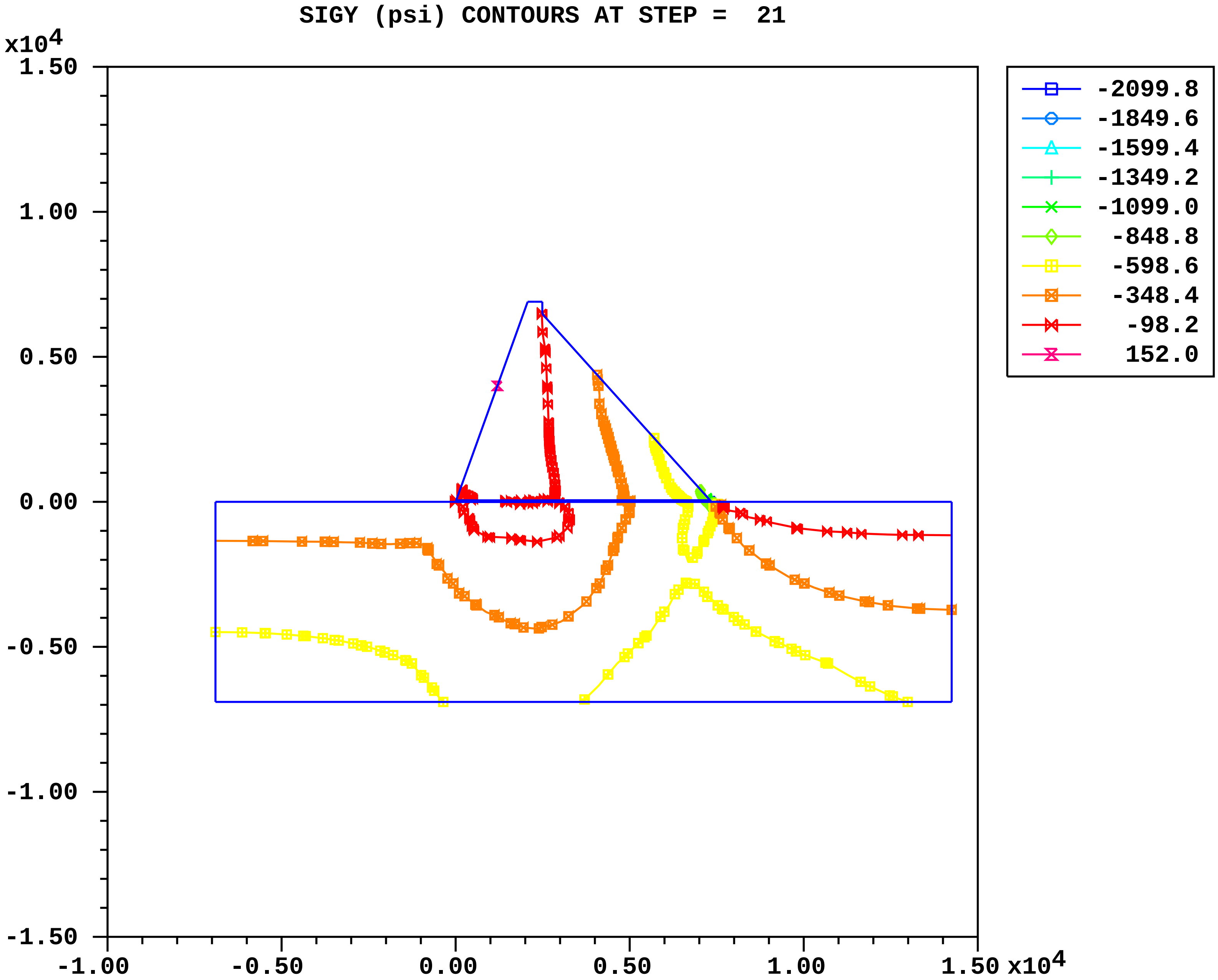


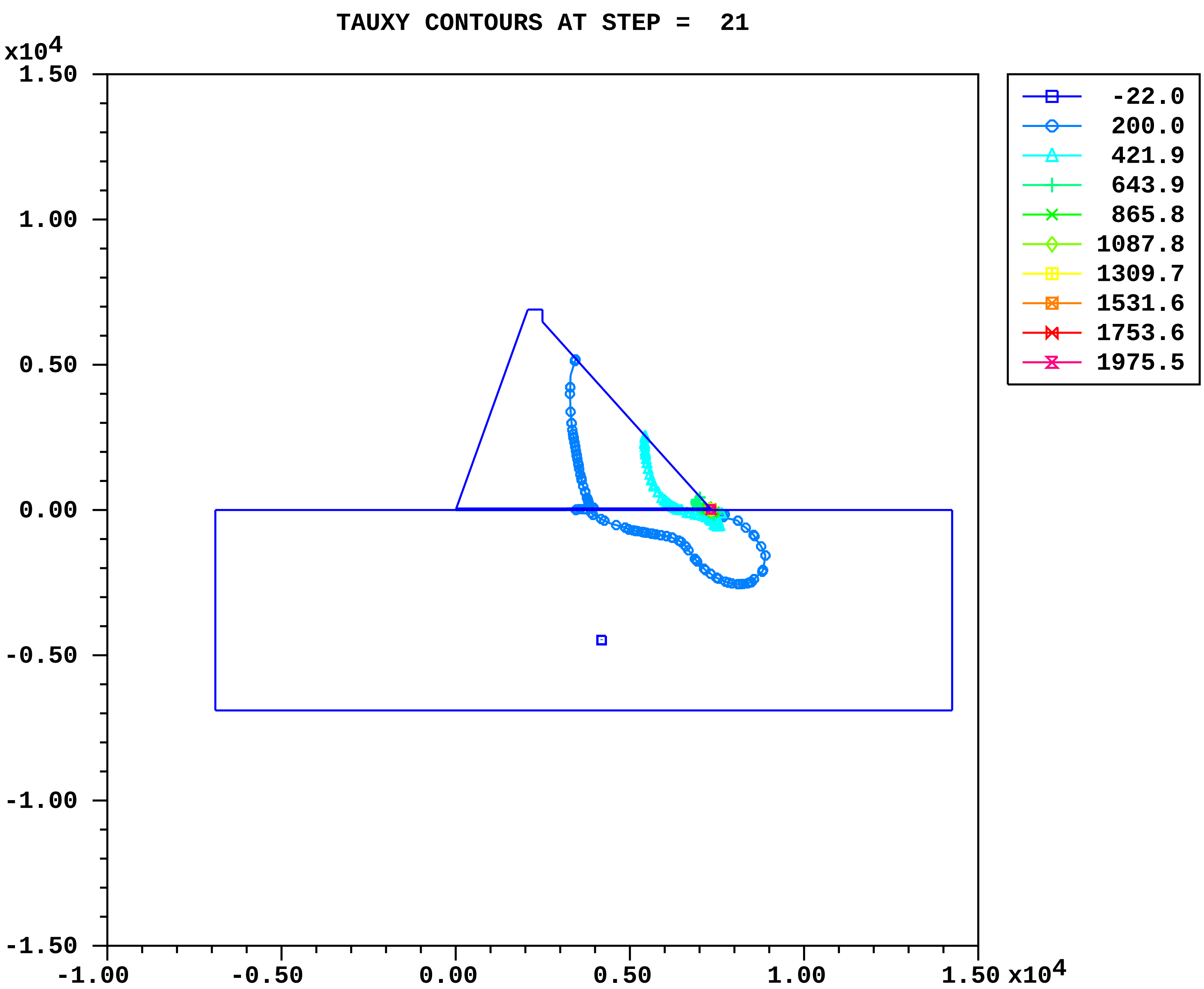

Frame 4 TAPE50

CG-DAMS PC-2.2.0 02/27/04 11:49:13.63



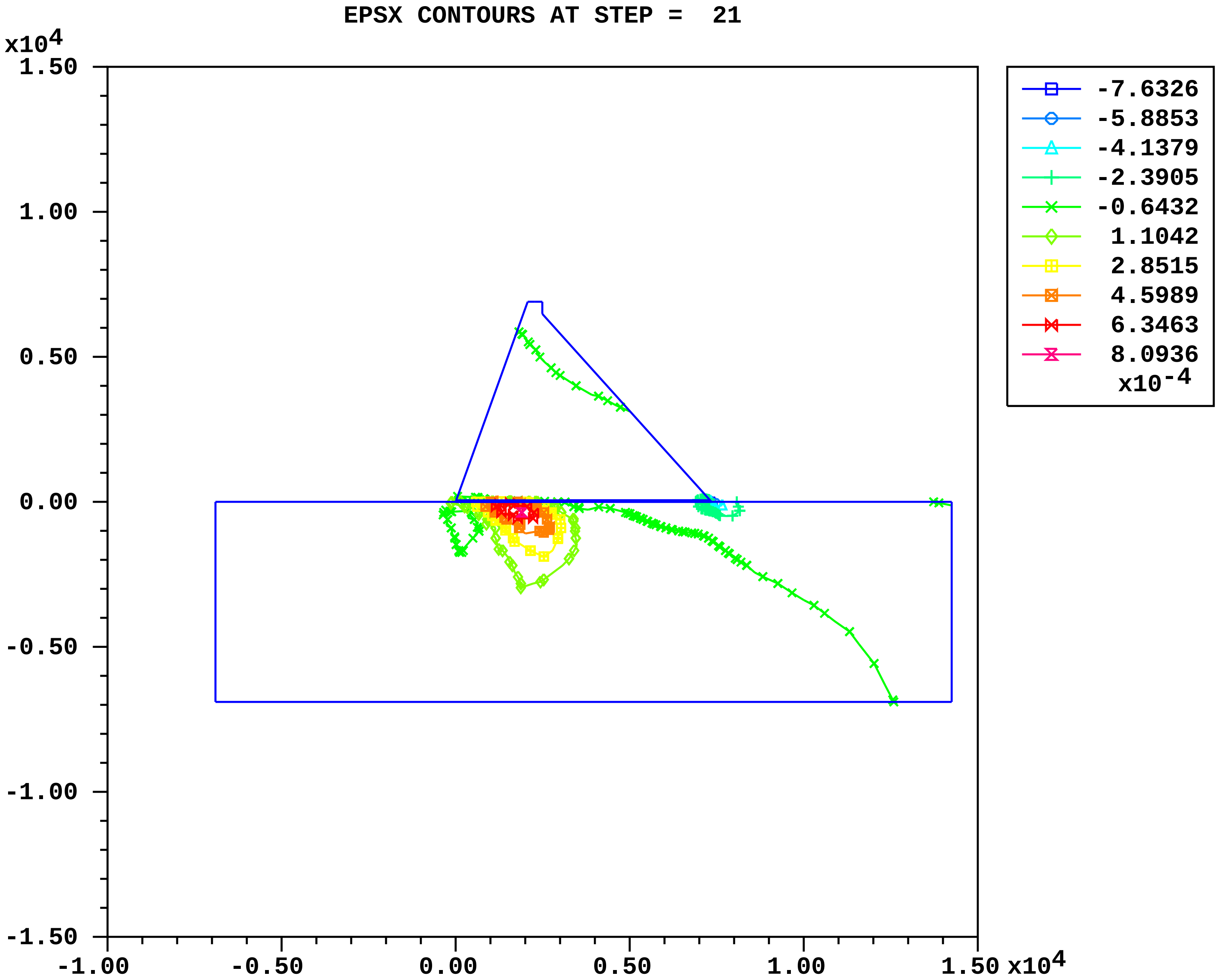

Frame 5 TAPE50

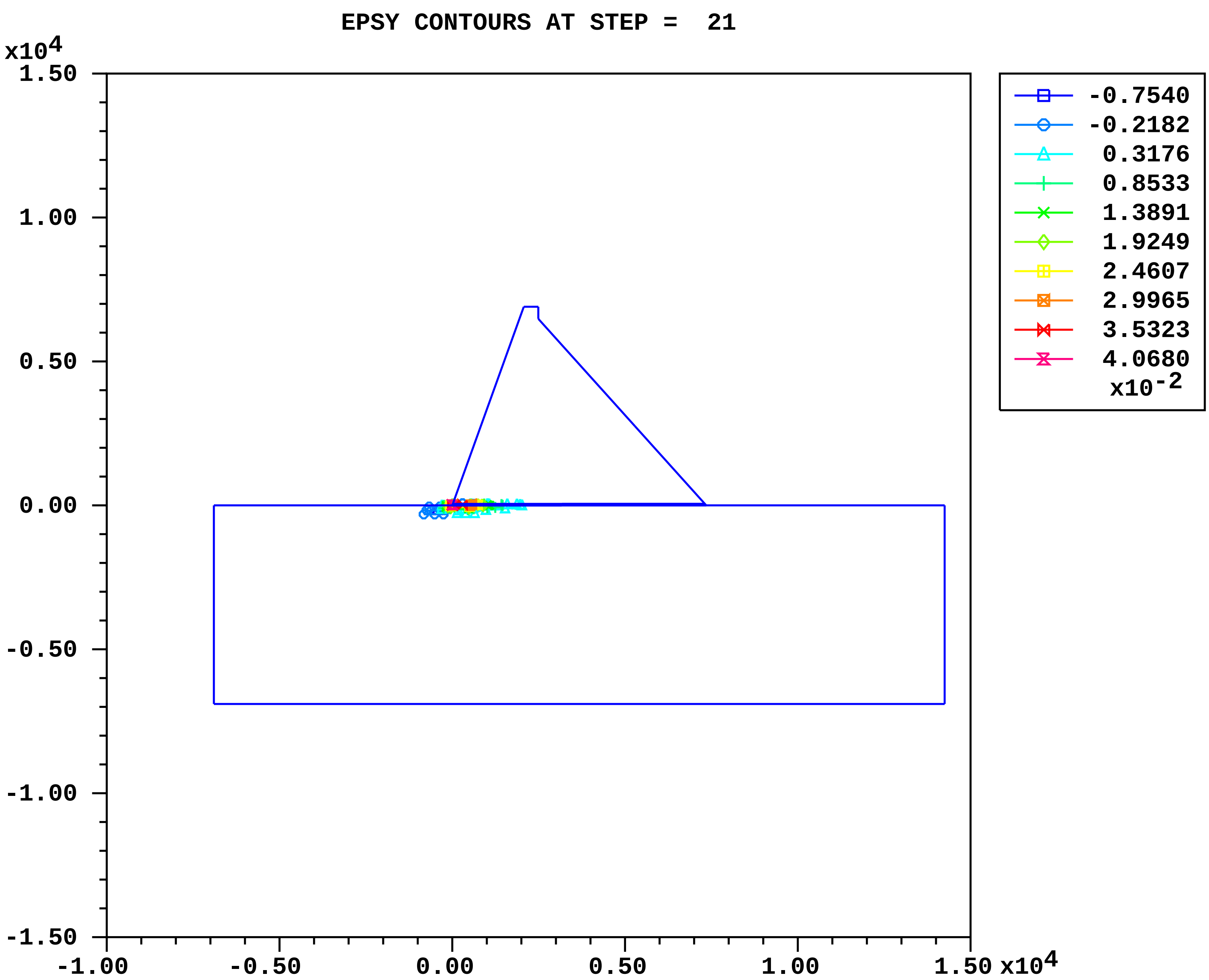
CG-DAMS PC-2.2.0 02/27/04 11:49:13.63




Frame 6 TAPE50

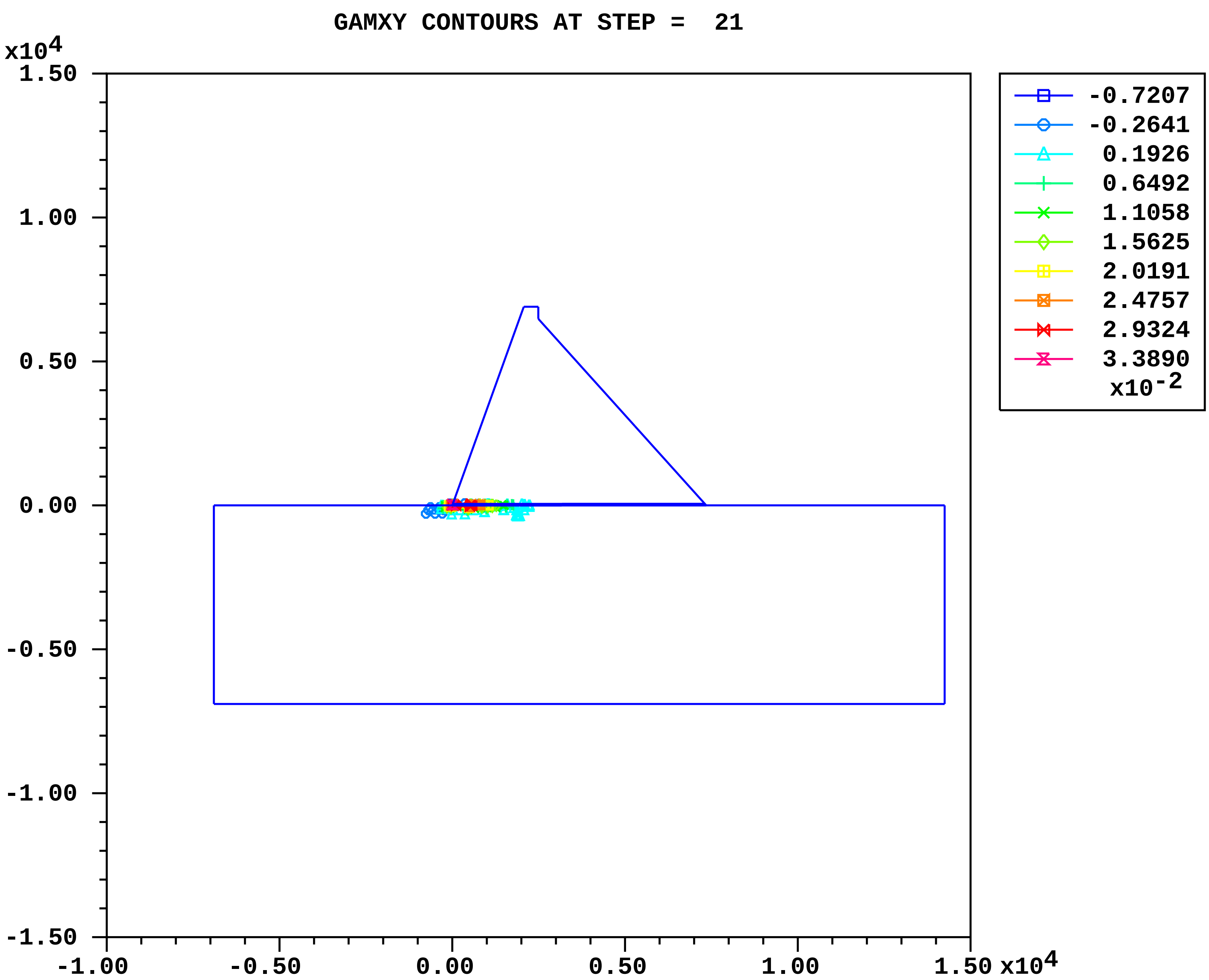
CG-DAMS PC-2.2.0 02/27/04 11:49:13.63

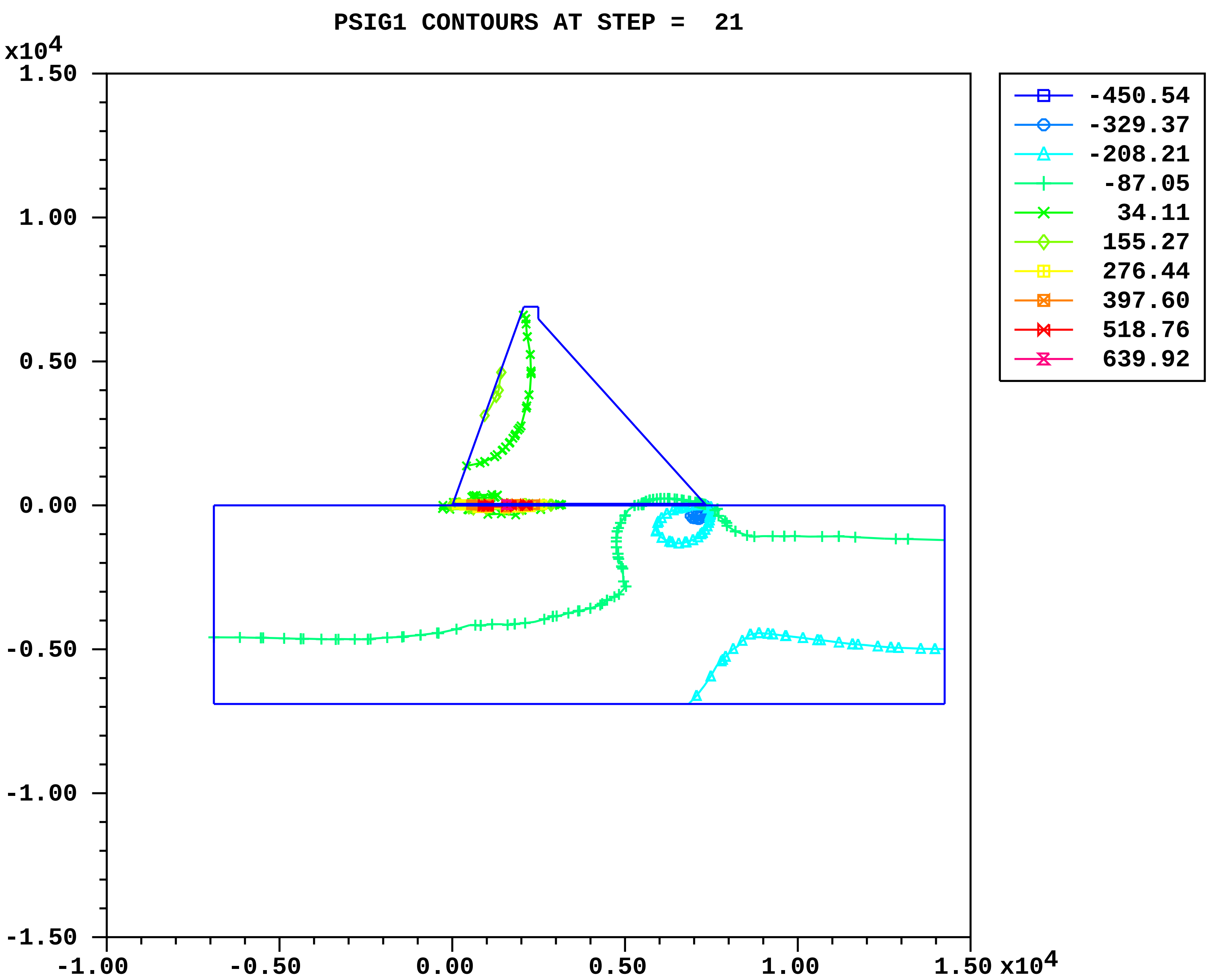

EPSX CONTOURS AT STEP = 21
x10^4
1.50
1.00
0.50
0.00
-0.50
-1.00
-1.50
-1.00
-0.50
0.00
0.50
1.00
1.50 x10^4
-7.6326
-5.8853
-4.1379
-2.3905
-0.6432
1.1042
2.8515
4.5989
6.3463
8.0936
x10^-4




Frame 8 TAPE50

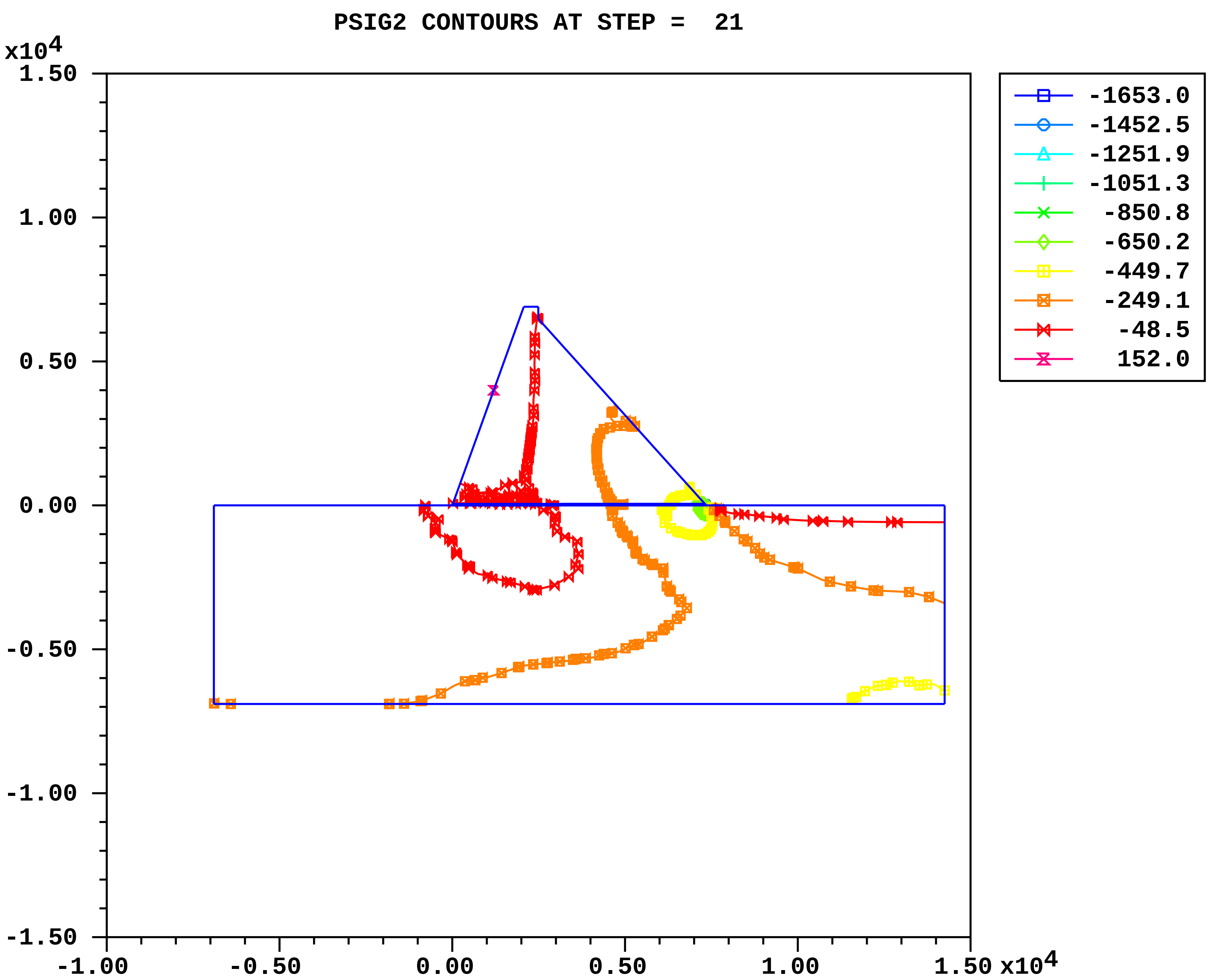
CG-DAMS PC-2.2.0 02/27/04 11:49:13.63


GAMXY CONTOURS AT STEP = 21
x10^4
1.50
1.00
0.50
0.00
-0.50
-1.00
-1.50
-1.00
-0.50
0.00
0.50
1.00
1.50 x10^4
-0.7207
-0.2641
0.1926
0.6492
1.1058
1.5625
2.0191
2.4757
2.9324
3.3890
x10^-2



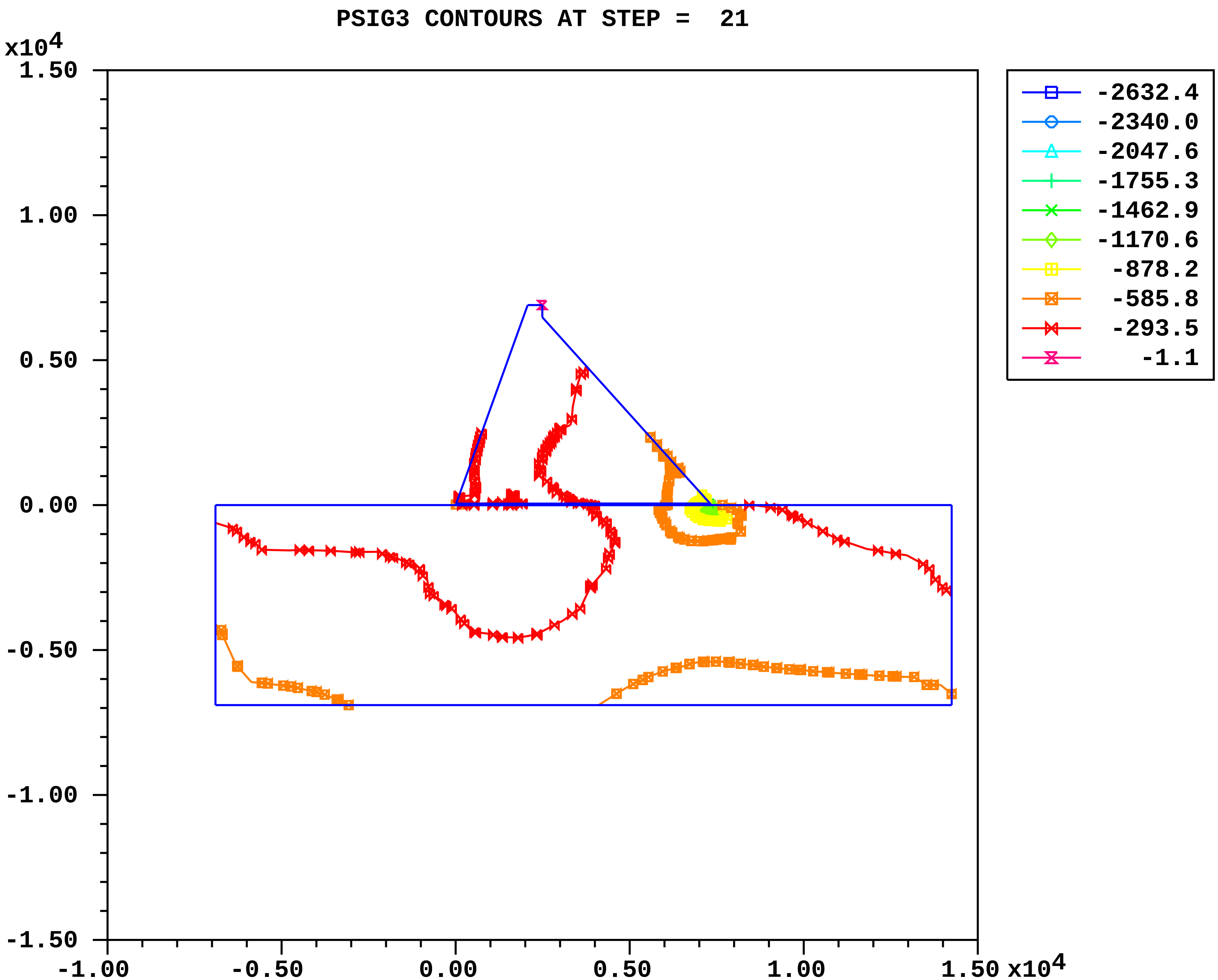

Frame 10 TAPE50

CG-DAMS PC-2.2.0 02/27/04 11:49:13.63




Frame 11 TAPE50

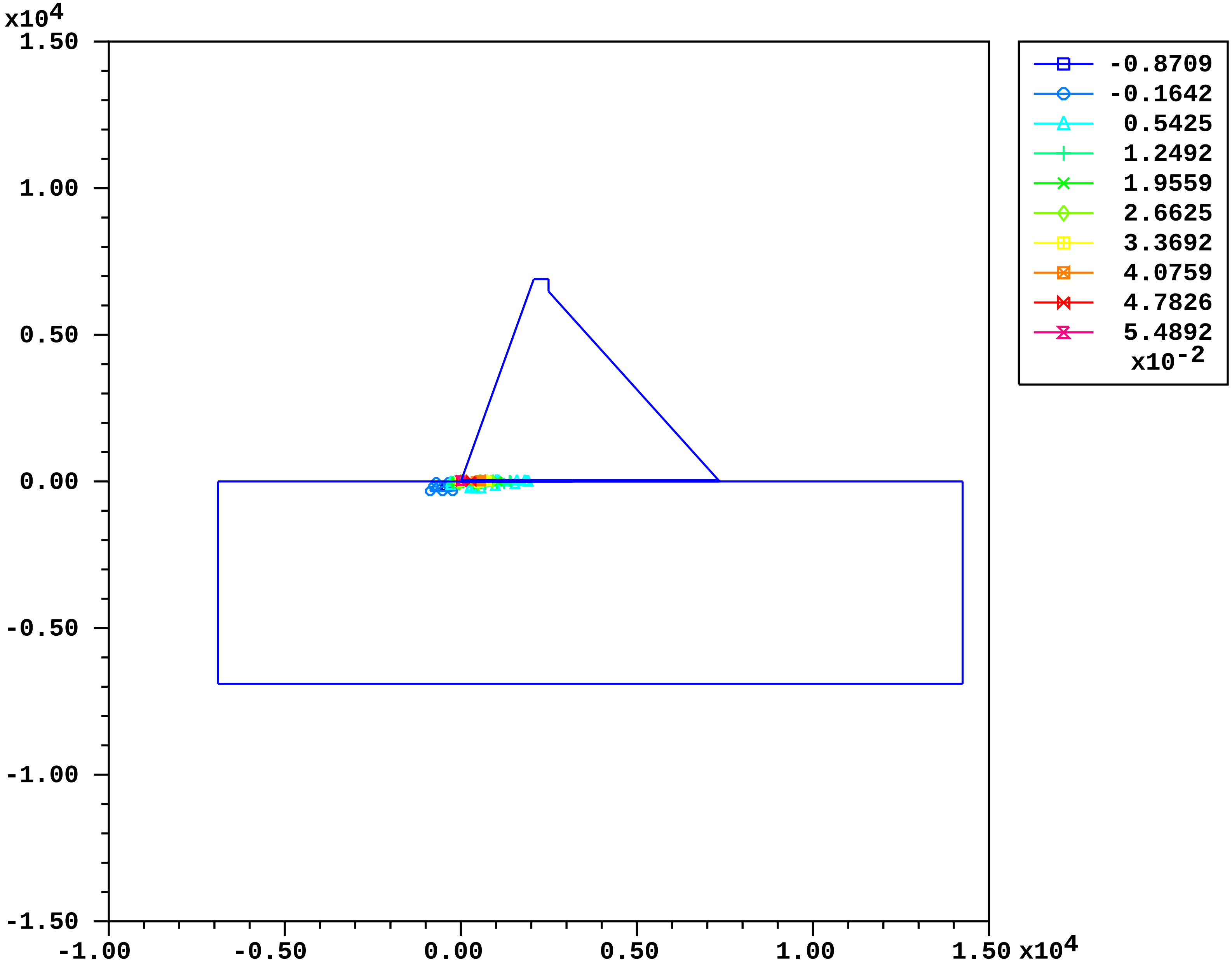
CG-DAMS PC-2.2.0 02/27/04 11:49:13.63




Frame 12 TAPE50

CG-DAMS PC-2.2.0 02/27/04 11:49:13.63

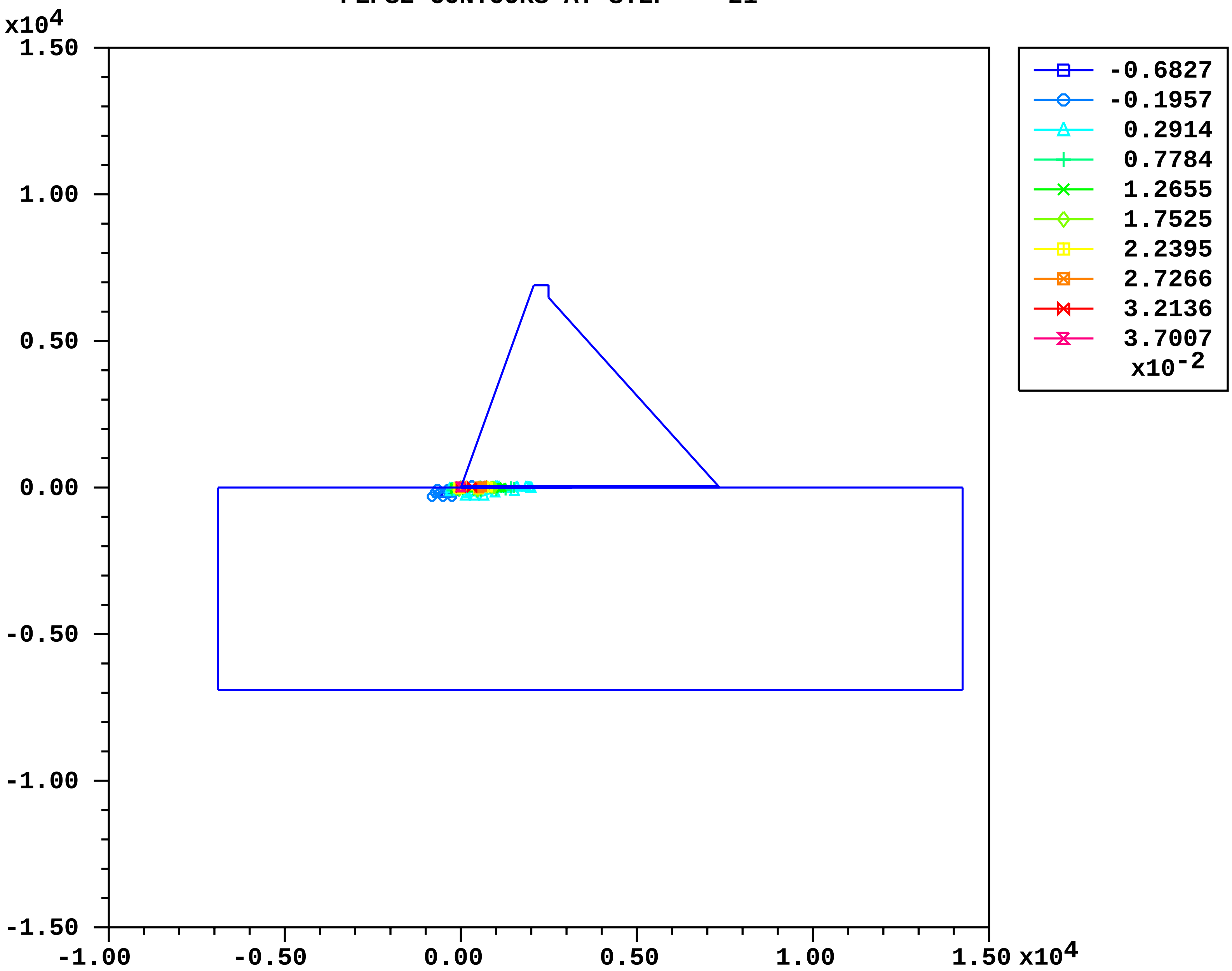
PEPS1 CONTOURS AT STEP = 21




Frame 13 TAPE50

CG-DAMS PC-2.2.0 02/27/04 11:49:13.63

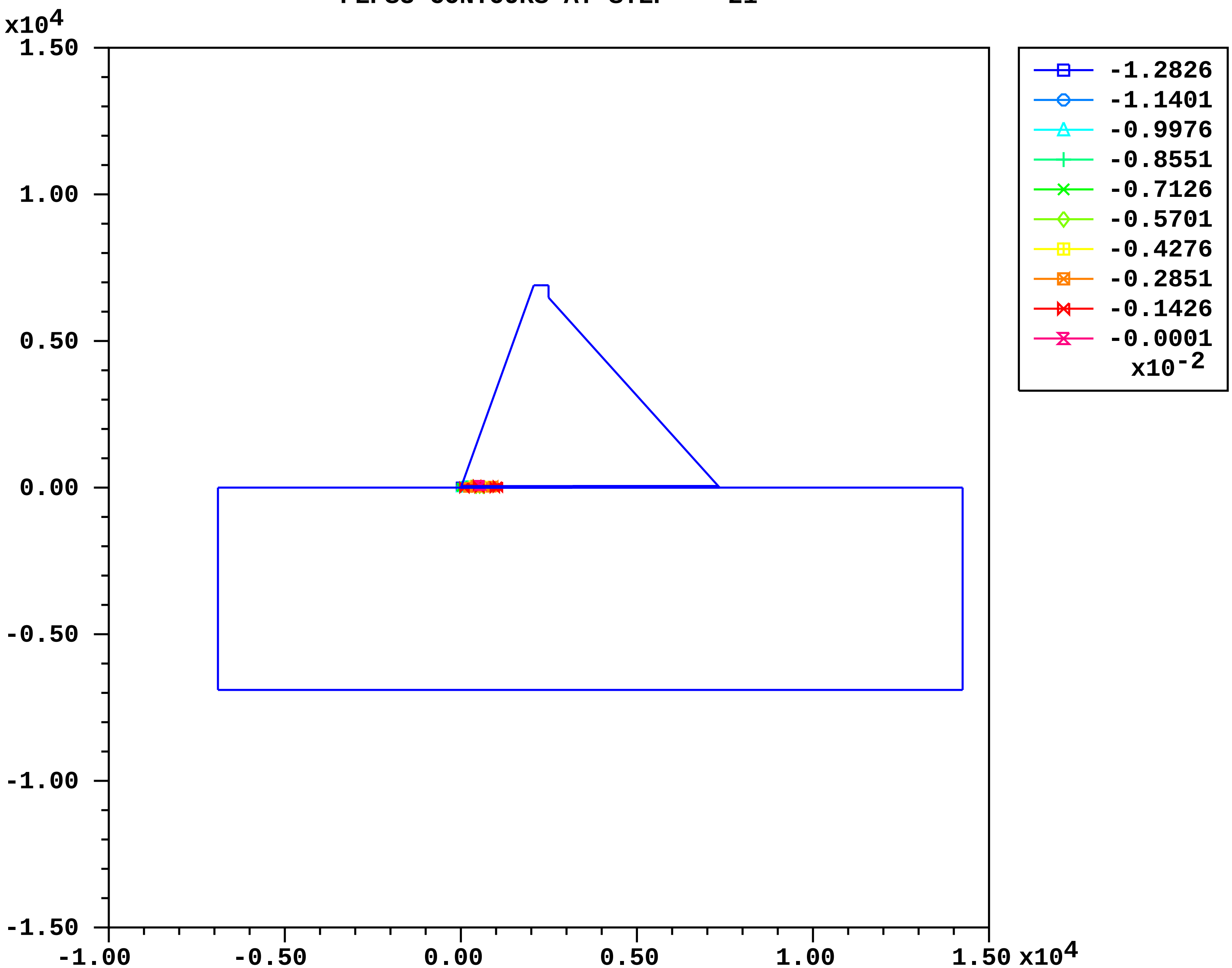
PEPS2 CONTOURS AT STEP = 21




Frame 14 TAPE50

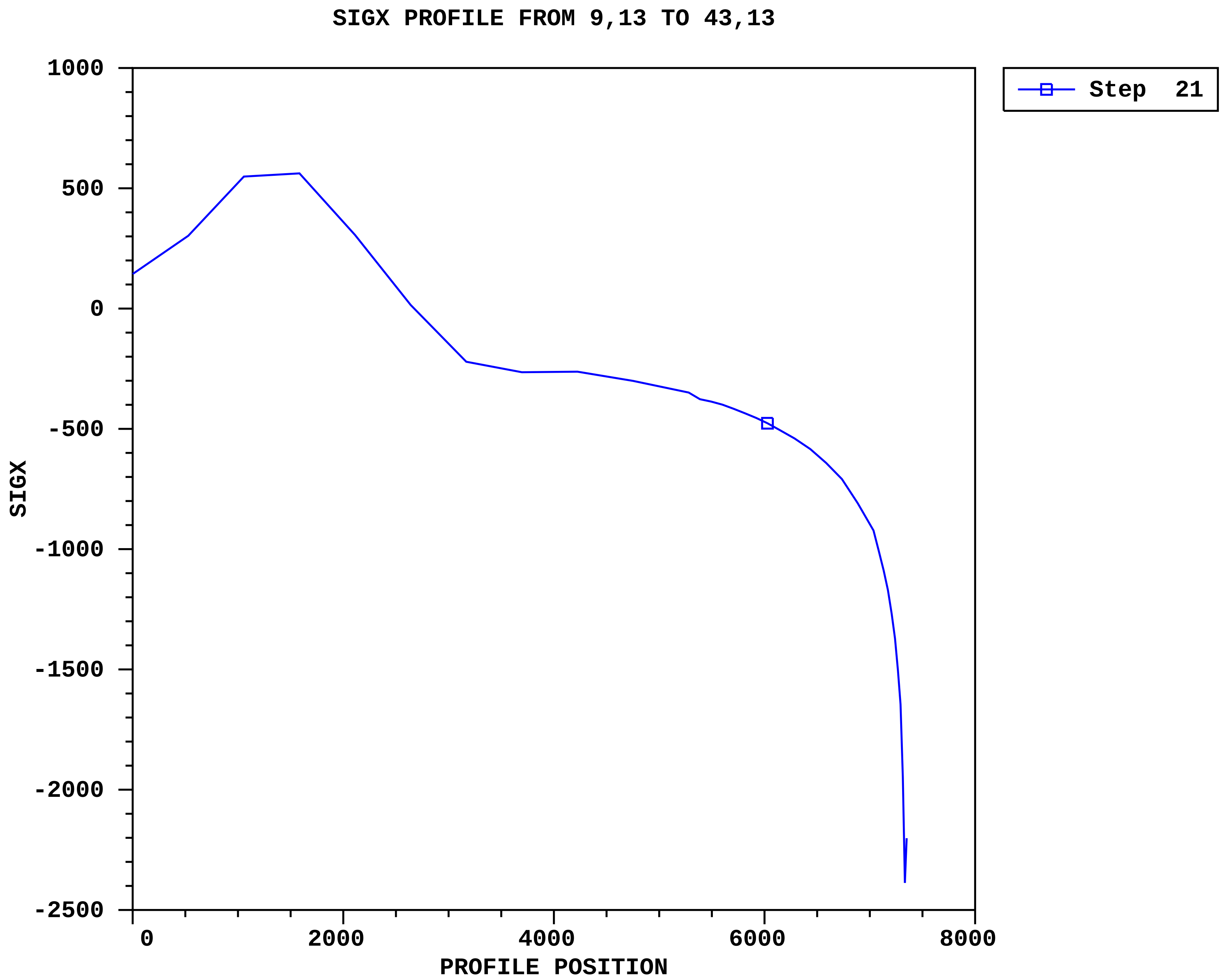
CG-DAMS PC-2.2.0 02/27/04 11:49:13.63

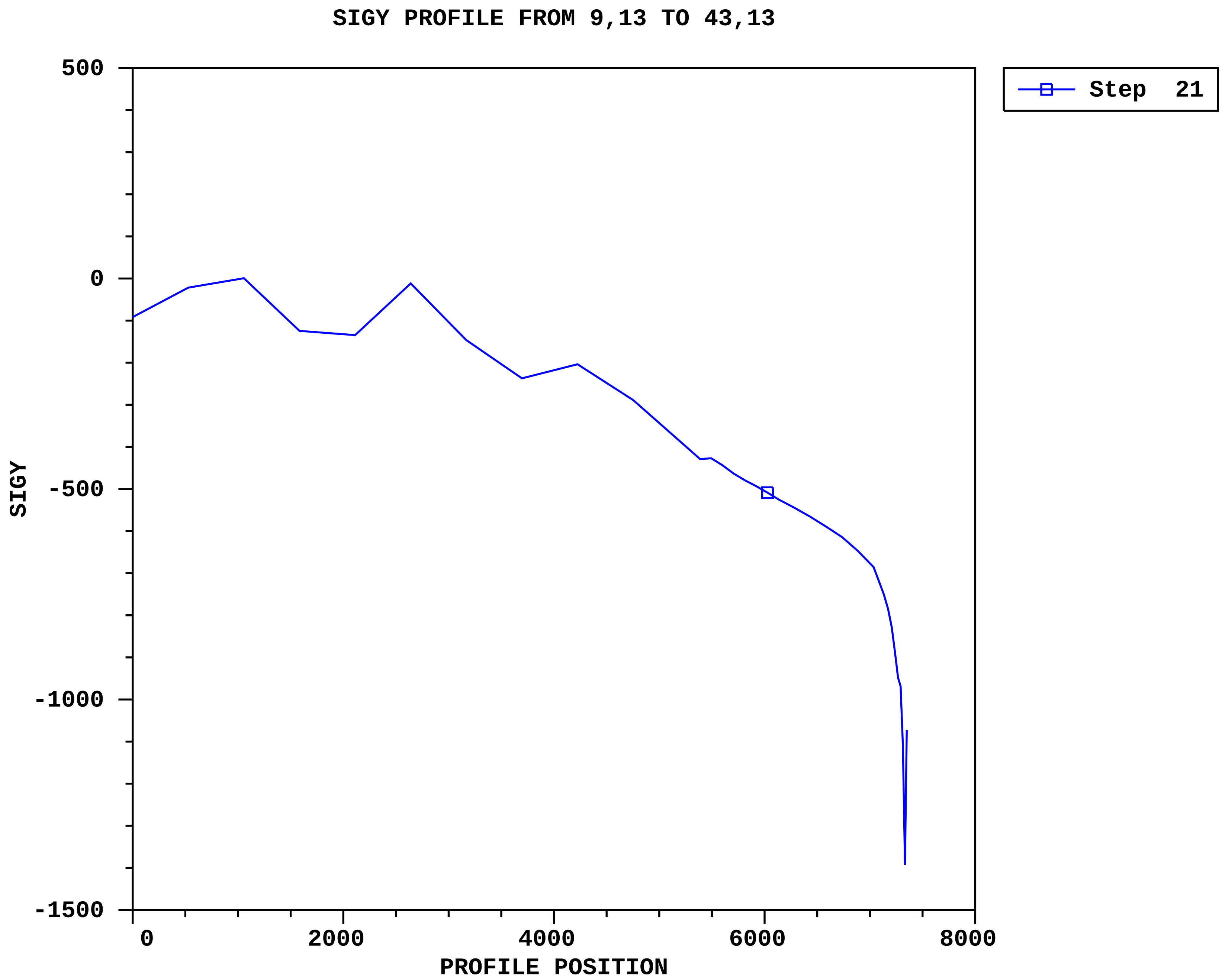
PEPS3 CONTOURS AT STEP = 21

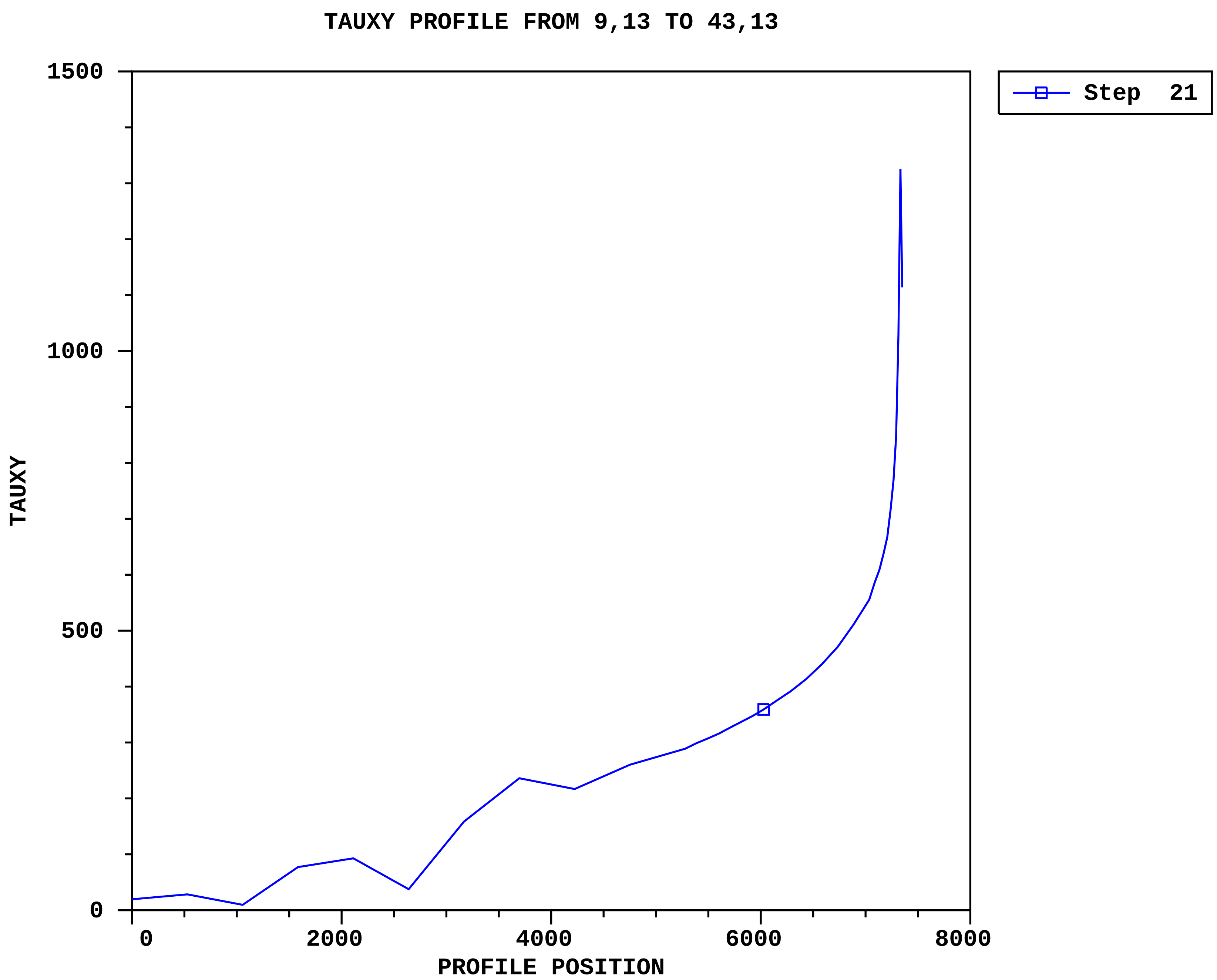



Frame 15 TAPE50

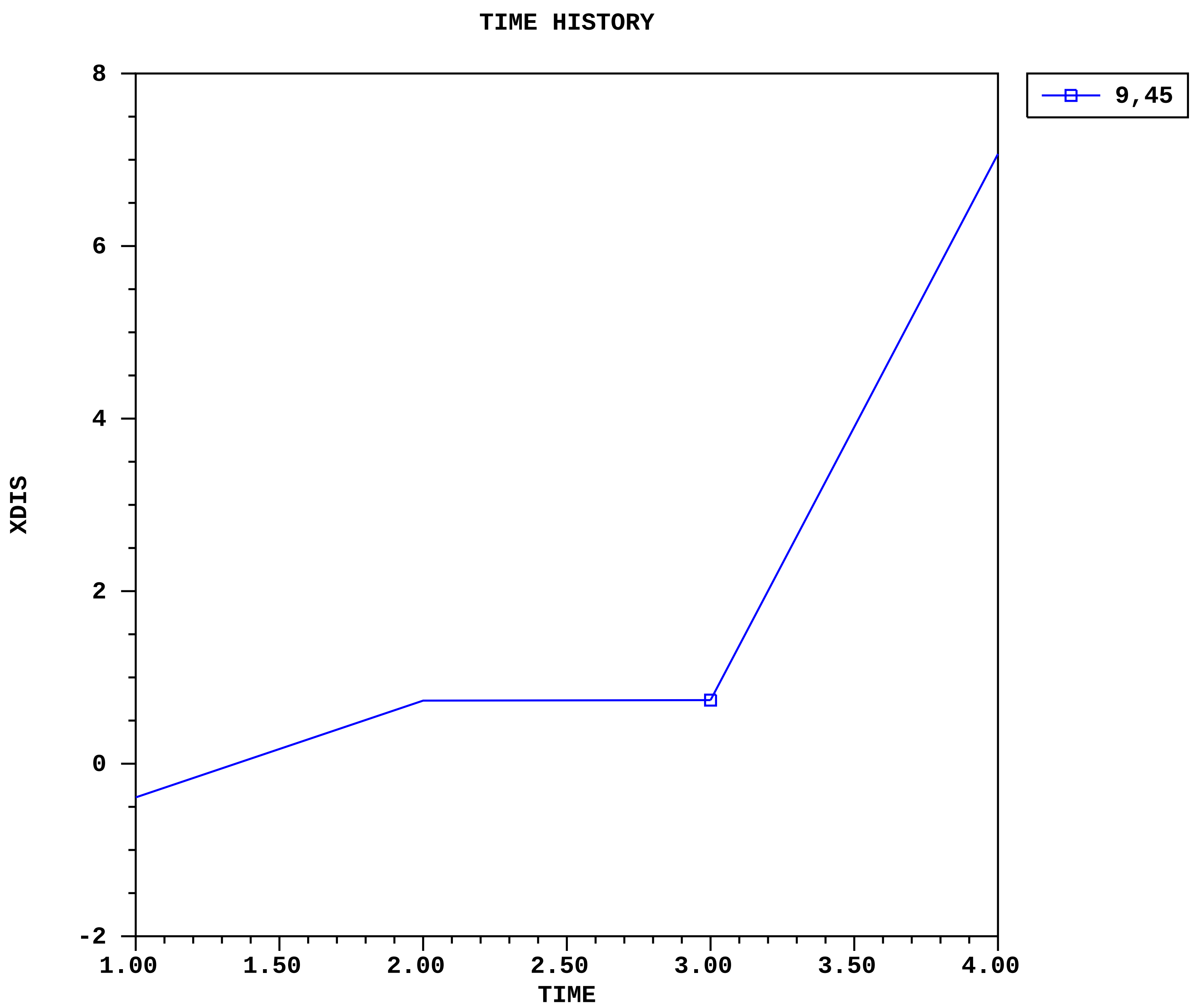
CG-DAMS PC-2.2.0 02/27/04 11:49:13.63

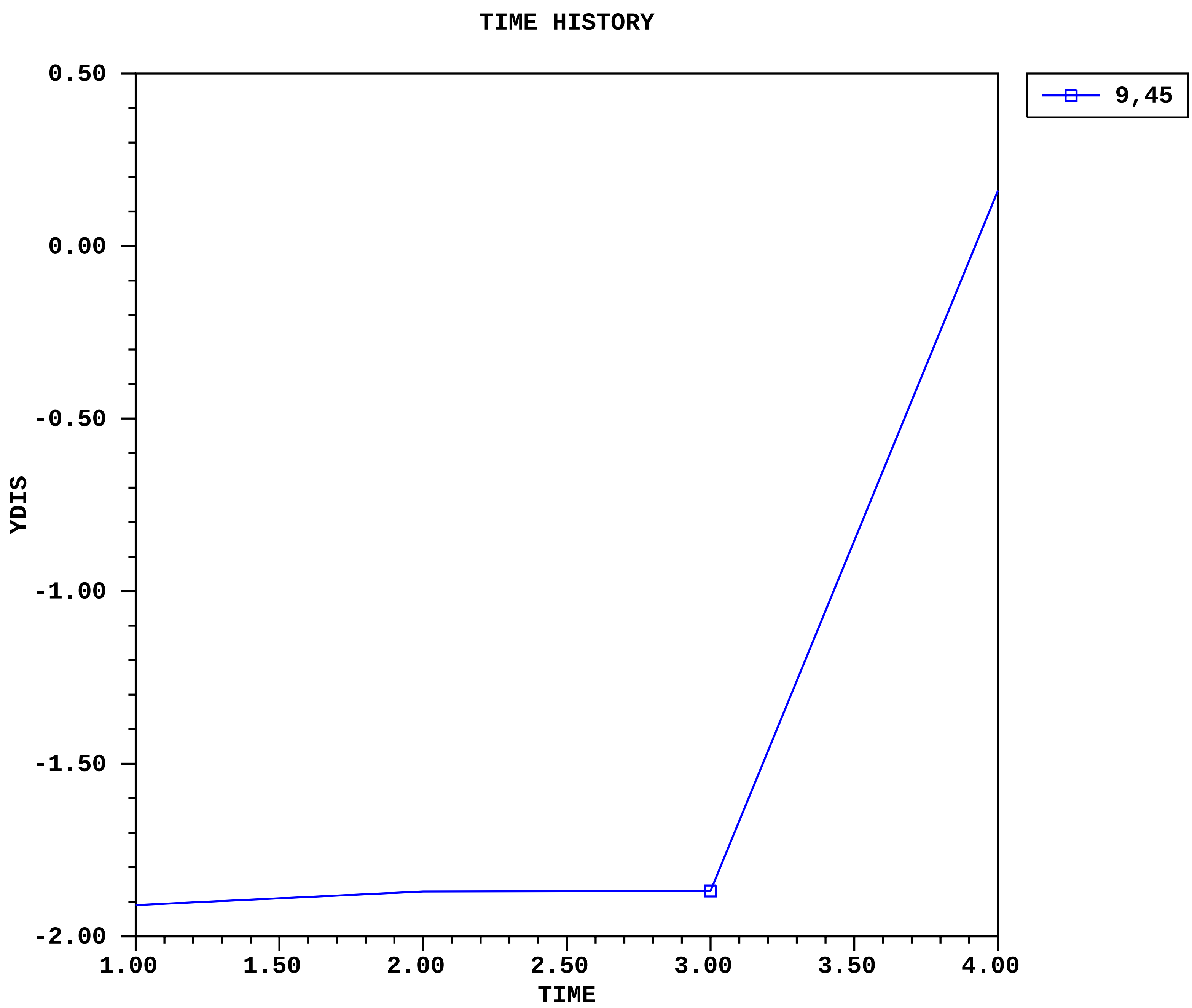

SIGX PROFILE FROM 9,13 TO 43,13
Step 21
1000
500
0
-500
-1000
-1500
-2000
-2500
SIGX
0
2000
4000
6000
8000
PROFILE POSITION

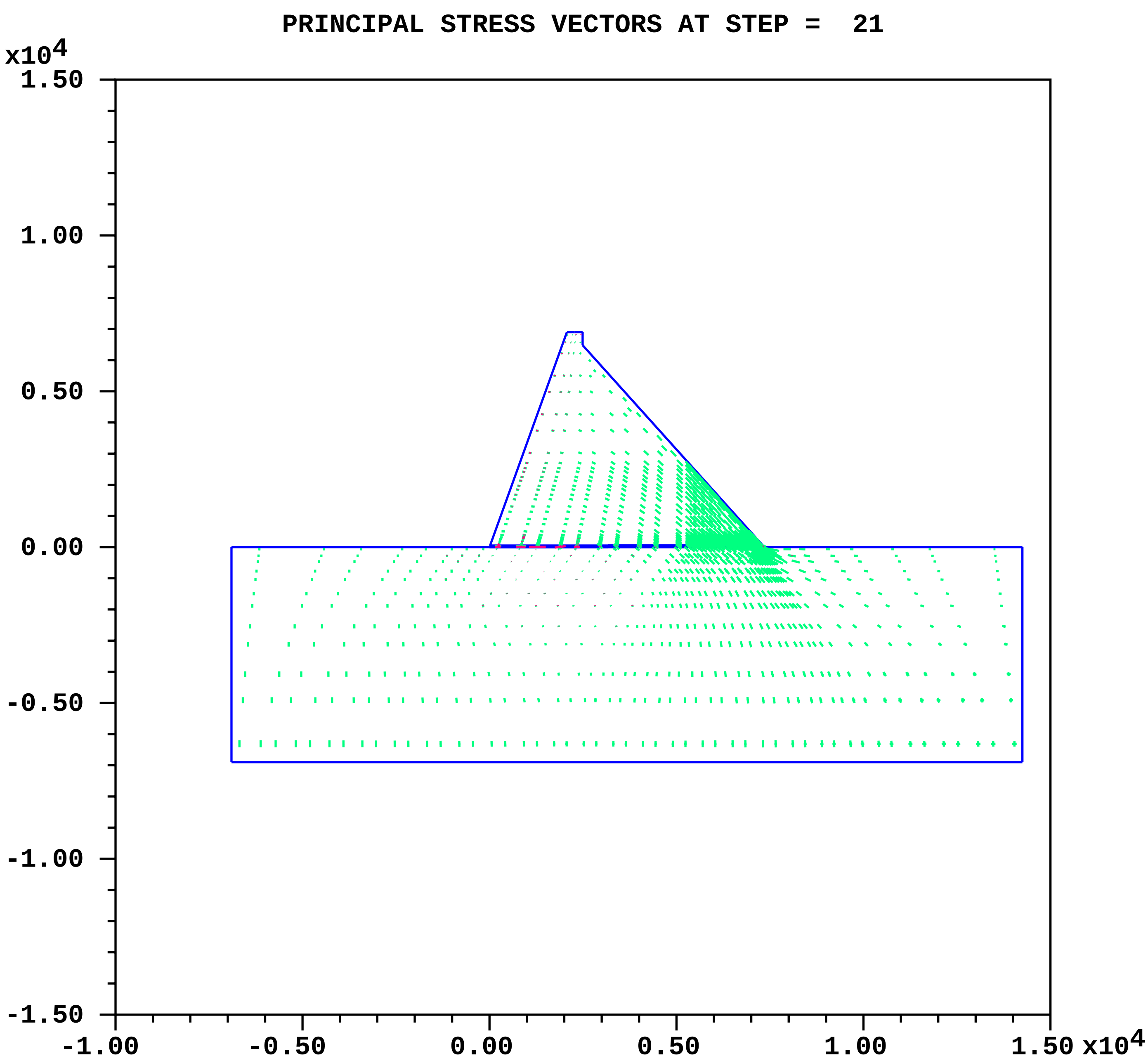

SIGY PROFILE FROM 9,13 TO 43,13
500
0
-500
-1000
-1500
SIGY
0
2000
4000
6000
8000
PROFILE POSITION
Step 21




Frame 18 TAPE50 CG-DAMS PC-2.2.0 02/27/04 11:49:13.63

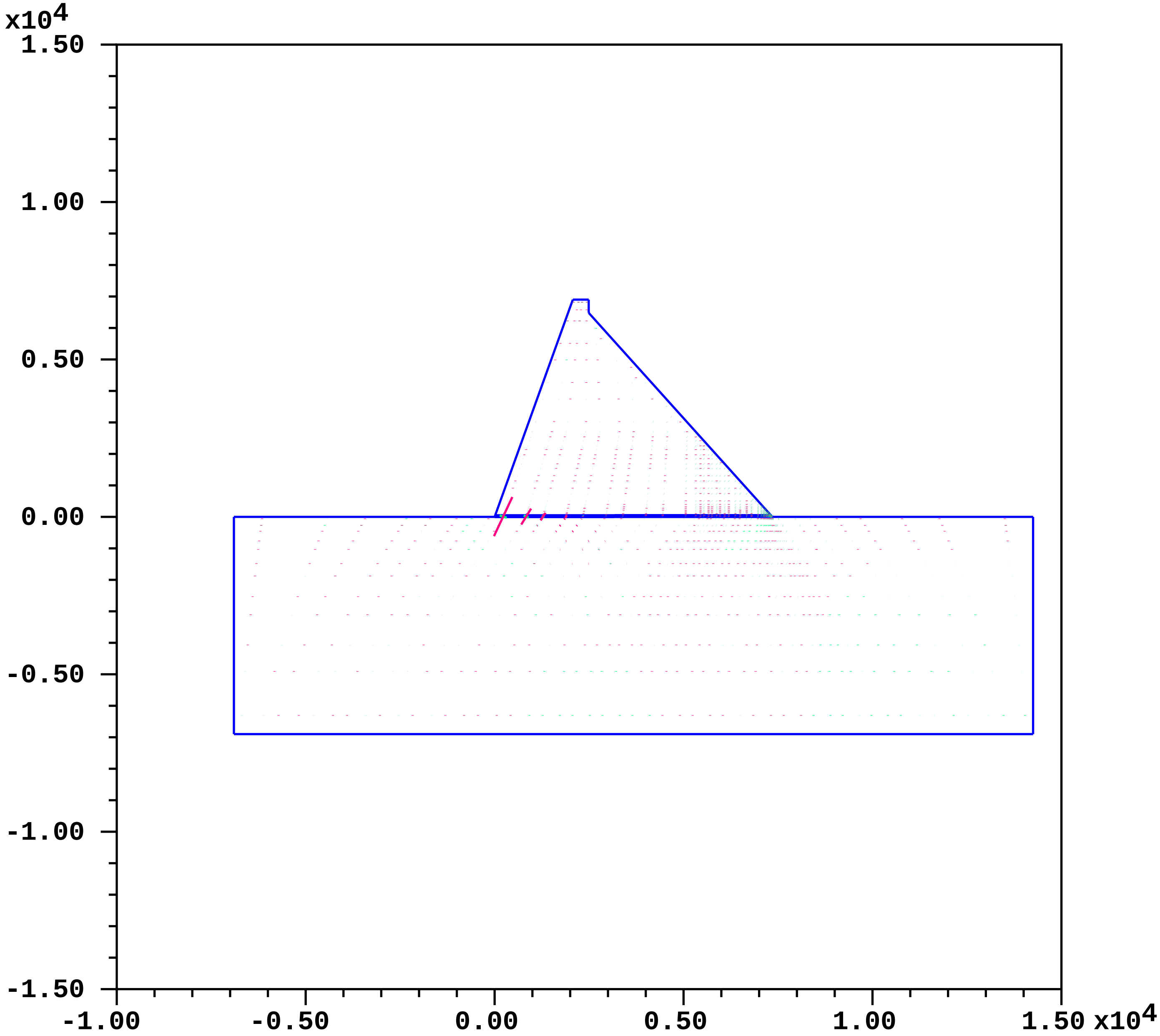

TIME HISTORY
8
6
4
2
0
-2
XDIS
9,45
1.00
1.50
2.00
2.50
3.00
3.50
4.00
TIME


TIME HISTORY
0.50
0.00
-0.50
-1.00
-1.50
-2.00
YDIS
1.00
1.50
2.00
2.50
3.00
3.50
4.00
TIME
9,45

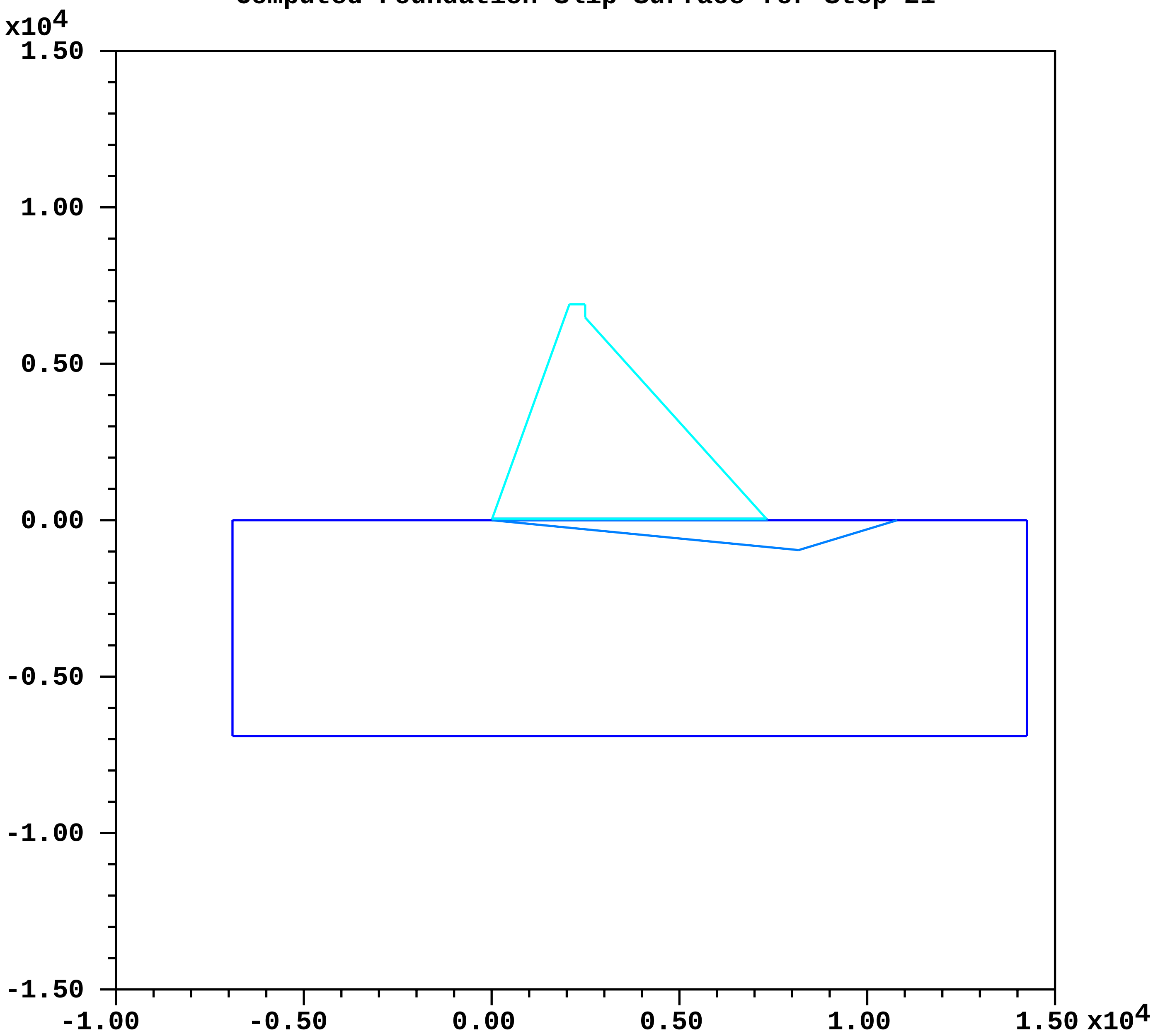



Frame 21 TAPE50 CG-DAMS PC-2.2.0 02/27/04 11:49:13.63

PRINCIPAL STRAIN VECTORS AT STEP = 21



Computed Foundation Slip Surface for Step 21




Foundation Shear Friction Factor of Safety: 2.400

02/27/04

# Union Park Dam Modified Non-overflow Section Crack-Dynamic Analysis, Extreme Loading, UPD17.SMT

| **Dam Geometry** | **Results** | **Units** |
|---|---|---|
| Crest elevation | 10140.00 | ft |
| Base elevation at heel | 9565.00 | ft |
| Base length | 612.50 | ft |
| | | |
| **Water Elevations** | | |
| Reservoir surface elevation | 10120.00 | ft |
| Silt elevation | 9565.00 | ft |
| Tailwater surface elevation | 9565.00 | ft |
| Backfill elevation | 9565.00 | ft |
| | | |
| **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 = c | 70.00 | psi |
| Internal friction angle = phi | 57.00 | deg |
| | | |
| **Crack Length** | | |
| Cracked length = t | 308.00 | ft |
| Uncracked length = l | 304.50 | ft |
| % of base cracked | 50.29 | |

| **Uplift Force (First Appl. Method)** | **Results** | **Units** |
|---|---|---|
| Initial uplift at start of analysis | -3203.85 | kip/ft |
| **Final uplift at end of analysis = U** | **-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 = n** | **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 = v** | **19889.76** | **kip/ft** |
| | | |
| **Shear Friction Factor of Safety** | | |
| Q=(cl + (n+U)tan(phi))/v | 1.82 | |

# Union Park Dam

# Appendix B1

# Seismic Refraction Surveys

!GEOPHYSICA

SEISMIC REFRACTION SURVEY
UNION PARK DAM
GUNNISON COUNTY, COLORADO

Prepared for
Ueblacker Associates
600 South Carr Street
Lakewood, CO 80226-0880

Job. No. 03-6
September 24, 2003

1840 MT. ZION DRIVE GOLDEN,CO 80401 (303) 587-2959

## INTRODUCTION

GEOPHYSICA was contacted by Ueblacker Associates to conduct a seismic refraction survey along the proposed Union park Dam axis to be located near the mouth of Lottis Creek in Gunnison County, Colorado. The proposed dam site is situated in a steep canyon just west of the upper mouth of Union Canyon where it exits Union Park. The seismic refraction survey was conducted to determine depth to bedrock and as a preliminary estimate of rock quality along the proposed dam axis as part of a feasibility study.

## FIELD PROCEDURES

The seismic refraction data were collected between September 9 and 11, 2003 using a Geometrics Seisview 24-channel, signal enhancement seismograph and 8-Hz vertical geophones spaced 10 feet apart. Lines 1, 2 and 3 were conducted from southwest to northeast along a line starting just north of Lottis Creek and extending up the talus slope to an area near the proposed right abutment. Lines 4, 5, 6 and 7 were conducted from northeast to southwest along a line starting on the south edge of Lottis Creek (across from Line 1) and extending up the slope to an area near the proposed left abutment. The seven lines form an almost continuous section along the proposed dam alignment. The uppermost sections on either side of the canyon were not profiled due to weather conditions and time constraints. Line 8 was conducted at right angles to the dam axis in the valley bottom to the south and approximately parallel to the road. Elevations along each line were determined with a hand level and the beginning elevation of 9603 feet at Geophone 1 on Line 1 was estimated based on the site map provided. Seismic refraction stationing was constructed to match the stationing on the maps supplied by the client. The ends of each line were marked in the field for future reference.

Geophones were firmly pushed into the ground surface using the standard spike bases where the ground surface was soft enough, or wedged into rock crevices on the talus slopes in order to obtain the best ground coupling possible. Seismic energy was generated using a 16 pound sledgehammer impacted on a convenient boulder or an aluminum plate. Multiple blows were stacked until clear data were obtained along the length of the seismic line. Energy points were located at the end of each line, offset from the end of each line and in the middle of each line. Records were inspected in the field to determine record quality which varied considerably based on the energy source, wind and rain. Record quality varied from very good to acceptable.

## DATA PROCESSING AND INTERPRETATION

The digital data are stored on the hard drive built into the seismograph and are downloaded to diskettes for archiving and to a computer for processing. The data are displayed and first arrival times picked by the interpreter using a program called FIRSTPIX (© Interpex). Site coordinates and hand-elevation data are also input to Firstpix which interpolates elevations and coordinates between known points. The data are then input to GREMIX which is another industry standard program from Interpex. Gremix uses the Generalized Reciprocal Method of seismic refraction interpretation which is commonly referred to as GRM analysis. The program requires a good understanding of seismic refraction interpretation in order to provide reasonable inputs during a number of operator interfaces required by the program. The interpretation results are output as XYZ data files and imported to AutoCadLT98© which was used to construct the figures.

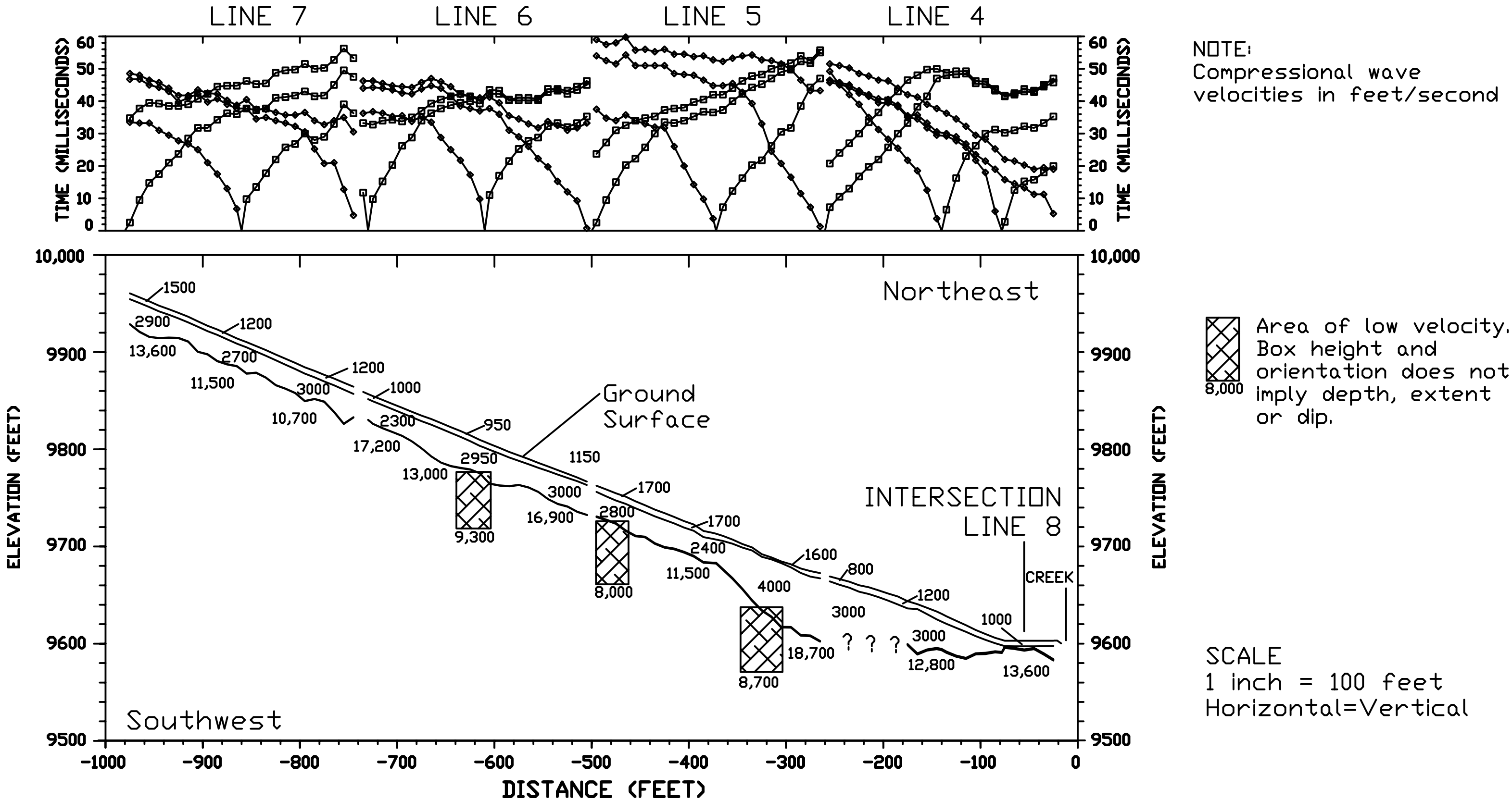
## RESULTS

The results of the seismic refraction survey are shown on Figures 1, 2 and 3. Figure 1 shows the results from Lines 4, 5, 6 and 7 along the proposed dam axis and extending from near the left abutment down the slope to Lottis Creek. The interpretation indicates three seismic layers. Velocities in the upper layer range from about 800 fps to 1700 fps. Compressional wave velocities less than 1000 fps are typical of talus and loose surface soils. This layer averages less than 10 feet thick along the entire line. The intermediate layer has an average velocity of about 2900 fps along the upper part of the line which could indicate denser soils (colluvium) or very weathered/fractured bedrock. This layer is typically about 30 feet thick except near the bottom of the slope where it thickens to between 50 and 60 feet thick and the velocity increases slightly to about 3500 fps. The bedrock velocities vary considerably from about 8,000 fps to over 17,000 fps. Overall bedrock velocity on this side of the valley averages about 12,700 fps which is typical of hard, relatively sound bedrock. The individual velocities shown on the section are averages and result from a detailed analysis of the velocity functions obtained from each shotpoint. Areas of bedrock velocity less than 10,000 fps are marked on the figure and are only intended to be used as an indication of the approximate lateral extent of lower velocity. These areas may indicate weak bedrock zones or areas of different material.

In the valley bottom adjacent to the creek, the seismic data indicate a two layer case (as does Line 8 which starts at -55 on Line 4). However, the interpretation software does not allow for a transition from a two-layer to a three-layer case within a single seismic line. In addition to forcing three layers into the interpretation in the valley bottom, the data in this area are also complicated by changes in slope and materials at the transition from slope to valley bottom. The seismic data in this case do not allow the discrimination of a relatively thin intermediate layer. The depth to bedrock may be slightly in error in this area depending on whether the intermediate layer exists or not. At any rate, the depth to bedrock in the valley bottom is less than 20 feet.

Figure 2 shows the results from Lines 1, 2 and 3 which run northeast from the edge of the creek and up the right abutment slope. The seismic sections show that most of the right abutment can be characterized as a two layer case with an upper, low velocity layer which averages about 10 feet thick along most of the slope and thickens slightly on the upper slopes to about 20 feet thick. The upper layer ranges in velocity from 900 fps to 1800 fps. These velocities are consistent with very loose talus and surface soils to moderately dense colluvium/highly weathered bedrock. An intermediate layer was also observed where the valley intersects the slope, creating a similar problem with the interpretation. The intermediate layer has a velocity of 4600 fps which could represent either dense soils or weathered bedrock. The area close to the creek is also affected by the thin intermediate layer problem encountered on Line 4 resulting in "dropouts" in the depth to bedrock in some areas where the program has difficulty in determining which layer it is dealing with. It appears that the depth to bedrock is generally less than 20 feet across the valley bottom and may be deeper under the creek. This apparent increase in depth next to the creek was also observed on Line 4 (Figure 1). Bedrock velocities range from 7,700 fps to 12,800 fps and average about 9,900 fps. It is apparent that the overall bedrock velocity is lower on this side of the valley. This may be due to slightly different composition, more fracturing or more deep weathering. There is one area of very high velocity which is indicated on the figure. This could represent a seam of different material or a cemented zone.

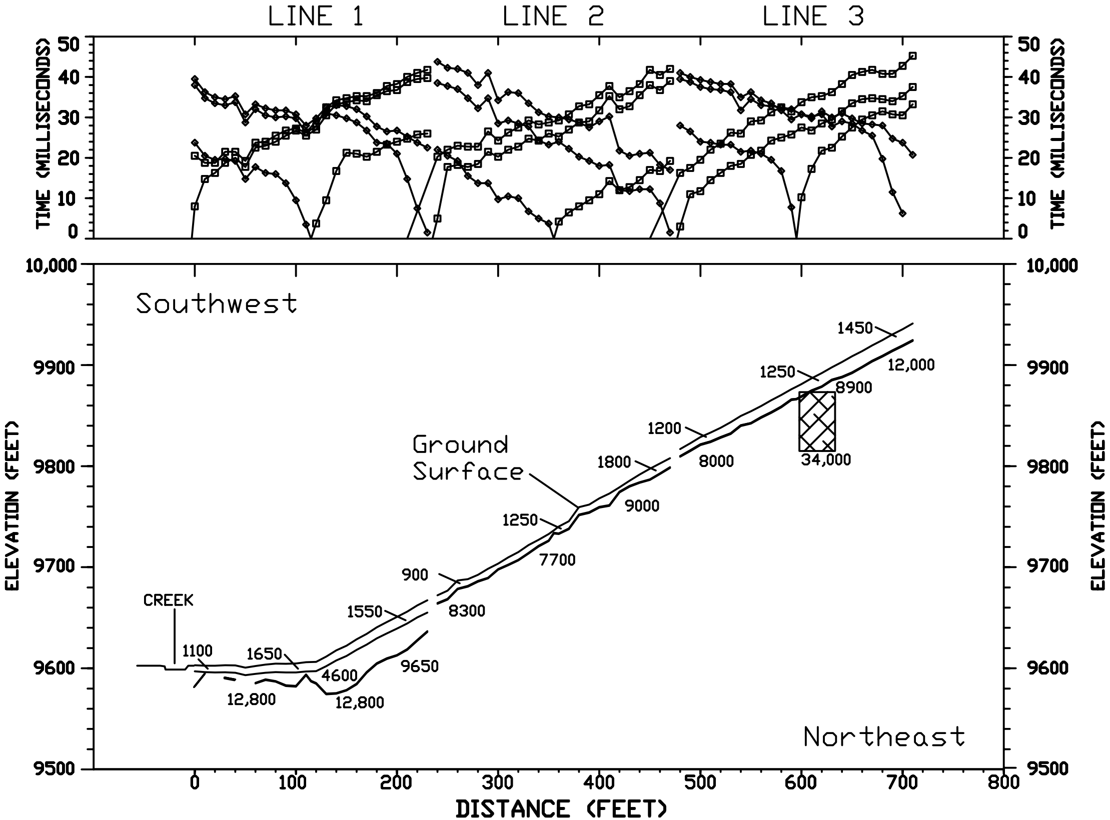
Figure 3 shows the results of Line 8 which was conducted roughly parallel to the dirt road south of Lottis Creek. This line has been interpreted using a two-layer model, although there are suggestions of an intermediate layer on the north end where the line crosses Line 4. Upper layer velocities range from 1100 to 1550 fps which are typical of unsaturated, loose to moderately dense soils. Depth to bedrock ranges from 10 to 20 feet. Bedrock velocities range from 14,400 to 17,400 fps which is indicative of sound bedrock. These velocities are somewhat higher than the corresponding bedrock velocity on Line 4, but this can be attributed to both a lack of topography on Line 8 (yielding more confidence in the values) and the fact that compressional wave velocities are often different when measured in a different direction due to the effects of inhomogeneity in the rock, fracture orientation and other factors such as stress direction.




**UNION PARK DAM**

SEISMIC REFRACTION PROFILE
LEFT ABUTMENT TO CREEK

| GEOPHYSICA | JOB NO. 03-6 | FIGURE 1 |
|---|---|---|

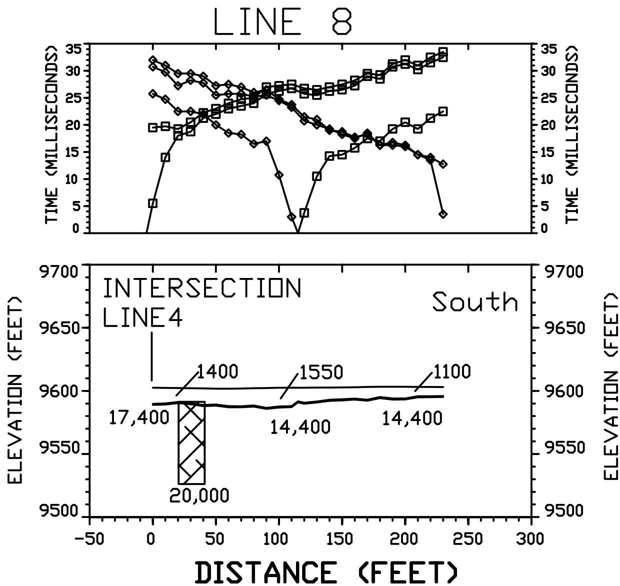


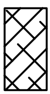

NOTE:
Compressional wave velocities in feet/second

34,000 Area of high velocity. Box height and orientation does not imply depth, extent or dip.

SCALE
1 inch = 100 feet
Horizontal=Vertical

| UNION PARK DAM | | |
|---|---|---|
| SEISMIC REFRACTION PROFILE CREEK TO RIGHT ABUTMENT | | |
| GEOPHYSICA | JOB NO. 03-6 | FIGURE 2 |





20,000

Area of high velocity. Box height and orientation does not imply depth, extent or dip.

SCALE
1 inch = 100 feet
Horizontal=Vertical

| UNION PARK DAM | | |
|---|---|---|
| SEISMIC REFRACTION PROFILE<br>VALLEY BOTTOM | | |
| GEOPHYSICA | JOB NO. 03-6 | FIGURE 3 |

# Union Park Dam

# Appendix B2

# Core Drilling Proposal

**Crux Subsurface, Inc.**

**16707 E. Euclid Ave**
**Spokane Valley, WA 99216**

REVISED
January 21, 2004

Ueblacker Associates
600 South Carr Street
Lakewood, CO 80226-0800

Crux Project Number:
**304-03-CO Ueblacker Union Park Dam Exploration**

Assumptions:

1. Crux will drill 18 borings. 9 borings will be drilled to 200 feet in depth on the abutments of the main dam location, 8 borings will be drilled to 150 feet in depth at the saddle dam location, and 1 boring will be drilled to 700 feet at the powerhouse location. All borings are to be drilled through talus colluvium and granodiorite bedrock. Crux will use HWT casing with spt's on 5 foot intervals, then core bedrock with HQ3 triple-tube core. Crux assumes a maximum of 80 feet of overburden. Crux will then conduct packer permeability tests, image and orient the features of the borings with COBL, then abandon the borings with bentonite backfill. In the powerhouse boring Hydro-fracture testing will be conducted.
2. Crux will access the 9 main dam borings with the use of a helicopter for moving the rigs and transporting crews. Crux will prepare one helicopter landing zone two-thirds the way up each abutment for safe crew transport. This may require some tree removal. The platform for the helicopter to set down on will be erected on the slope, minimizing environmental impact, and will be removed upon completion. The 8 saddle dam borings will be drilled with truck or track mounted equipment along an existing forest service road. The attached spreadsheet reflects that Crux will access the powerhouse boring by creating a rough road, sufficient for a track mounted rig. This will make hydro-fracture testing feasible, and minimize the cost of this boring. The road will be reclaimed by Crux upon completion of the exploration.
3. Crux is assuming packer permeability tests on 25 foot intervals in the bedrock.
4. Crux will provide two water trucks, two high pressure water pumps and high pressure waterline capable of 500 feet of head.
5. Ueblacker will provide utility locates, access permits, and permits to draw water from local sources.
6. Ueblacker will provide personnel to box and log the core, packer and insitu testing results. Crux will transport Ueblacker personnel to the sites via helicopter and transport the core to a truck accessible location upon the completion of each borehole.
7. Crux will provide three drill rigs and all necessary equipment to complete the project. All three rigs will be helicopter portable and at least one will have a track

carrier. While the helicopter work is being performed, all three rigs will work at the dam site. Either before or after the dam work is completed, one of the rigs will drill and test the Powerhouse Boring.

8. This is a reduced cost for COBL. The cost per foot is valid for this scope only.
9. Crux will work 10 days on (10 hours per day on the rig), 4 days off schedule. The three rig helicopter project will take 14 project days or 18 calendar days. The saddle dam borings will take 25 project days with one rig or 13 project days with two drills. The powerhouse hole will take approximately 21 project days or 25 calendar days. No helicopter or rental charges will be incurred while Crux is off site during crew breaks.
10. If all drilling is done concurrently a total of $19,600.00 will be saved in mobilization charges.

**DESCRIPTION**

| | | | |
|---|---|---|---|
| CLIENT | UEBLACKER ASSOC. | | |
| JOB NUMBER/NAME | 304-03-CO UNION MAIN DAM | | |
| # OF DRILLS | 3 | RIGS | |
| PRICE PER MOB PER DRILL | $ 7,000.00 | PER RIG | |
| TOTAL FOOTAGE | 1,800 | FEET | |
| ESTIMATED FTG / SHIFT | 75 | FEET | |
| EXPECTED RECOVERY | 100% | PERCENT | |
| NUMBER OF HOLES TYPE 1 | 9 | HOLES | |
| NUMBER OF HOLES TYPE 2 | | HOLES | |
| HOURS PER MOVE TYPE 1(INCL. H20) | 6.0 | HOURS | |
| HOURS PER MOVE TYPE 2(INCL. H20) | | HOURS | |
| HOURLY RATE | $ 175.00 | PER HOUR | |
| FEET OF PIEZO | | FEET | |
| FEET OF INCLIN. CASING | | FEET | 54 |
| FEET TO BE ABANDONED | 1,800 | FEET | |
| ABANDONMENT COST PER FOOT | $ 3.00 | PER FOOT | |
| # OF EXTRA MEN | 4 | MEN | |
| HOURS OF STANDBY | | HOURS | |
| DAYS WORKED PER WEEK | 7 | DAYS | |
| PRICE PER MAN DAY SUBSIS. | 85 | DOLLARS | |
| TRAVEL TIME ROUND TRIP PER DAY | 1.0 | HOURS | |
| # OF WATER TRUCKS | 2 | NUMBER | |
| # OF GROUT PLANTS | | NUMBER | |
| # OF BARGES | | NUMBER | |
| # OF JET BOATS | | NUMBER | |
| # OF TRAFFIC CONTROL SETS | | NUMBER | |
| # OF CRANES | | NUMBER | |
| # OF CRANE DAYS | | DAYS | |
| CRANE MOB DEMOB COST | | DOLLARS | |
| CRANE DAILY COST | | PER DAY | |
| # OF OTHER RENTAL EQUIPMENT | 2 | RENTALS | |
| OTHER RENTAL EQUIPMENT MOB DEMOB | $ 1,000.00 | DOLLARS | |
| OTHER RENTAL EQUIPMENT DAYS | | DAYS | |
| OTHER RENTAL EQUIPMENT DAILY COST | $ 100.00 | DOLLARS | |
| NUMBER OF XTRA SPT SAMPLES | | SAMPLES | |
| NUMBER OF SHELBY OR PISTON SAMPLES | | TUBES | |
| NUMBER OF PACKER TESTS | 54 | TESTS | |
| NUMBER OF PACKER SETS | 3 | SETS | |
| AMOUNT OF WATERLINE OVER 1000' | $ 3,000.00 | DOLLARS | |
| COST OF CORE BOXES | $ 8.00 | DOLLARS | |
| PRICE FOR MISC. MATERIALS | $ 2,500.00 | PER LUMP | |

| | |
|---|---|
| CLIENT CONTACT | |
| START DATE | |
| DRILL RIGS | |
| CASING ADV Y/N | |
| HQ3 CORING Y/N | |
| 4 1/4" AUGER Y/N | |
| DEPTH SHALLOWEST | |
| DEPTH DEEPEST | |
| SPT INTERVALS | |
| WATER SOURCE | |
| BY CLIENT Y/N | |
| HAUL LENGTH | |
| PUMP DISTANCE | |
| LODGING | |
| UTILITIES BY CLIENT | |
| TRAFFIC BY CLIENT | |
| BIT CLAUSE Y/N | |
| LOST TOOLS Y/N | |
| BOXES BILLABLE | |
| MUD&MISC BILLABLE | |

UEBLACKER ASSOC.
304-03-CO UNION MAIN DAM

| DESCRIPTION | QUANTITY | UNIT | QUANTITY EXT | UNIT | COST | TOTAL |
|---|---|---|---|---|---|---|
| M/D | 3 | RIG | | | $ 7,000.00 | $ 21,000.00 |
| DRILLING | 1800 | FEET | | | $ 36.70 | $ 66,060.00 |
| MOVES TYPE 1 INCL H20 LINE | 54 | HOURS | | | $ 175.00 | $ 9,450.00 |
| MOVES TYPE 2 INCL H20 LINE | 0 | HOURS | | | $ 175.00 | $ - |
| EXTRA MEN | 4 | MEN | 13 | DAYS | $ 350.00 | $ 18,200.00 |
| PIEZO INSTALATION | 0 | FEET | | | $ 14.00 | $ - |
| INCLINOMETER CASING INSTALLATION | 0 | FEET | | | $ 20.00 | $ - |
| EXTRA SPTS | 0 | SAMPLES | | | $ 35.00 | $ - |
| SHELBY OR PISTON SAMPLES | 0 | SAMPLES | | | $ 50.00 | $ - |
| ABANDONMENT | 1800 | FEET | | | $ 3.00 | $ 5,400.00 |
| SUBSISTENCE | 10 | MEN | 13 | DAYS | $ 85.00 | $ 11,050.00 |
| STANDBY | 0 | HOURS | | | $ 100.00 | $ - |
| TRAVEL TIME | 3 | CREW HRS | 13 | DAYS | $ 100.00 | $ 3,900.00 |
| WATER TRUCK | 13 | DAYS | 2 | TRUCKS | $ 200.00 | $ 5,200.00 |
| GROUT PLANT | 13 | DAYS | 0 | PLANTS | $ 40.00 | $ - |
| BARGE | 13 | DAYS | 0 | BARGES | $ 360.00 | $ - |
| JET BOAT | 13 | DAYS | 0 | BOATS | $ 250.00 | $ - |
| TRAFFIC CONTROL MATERIALS | 13 | DAYS | 0 | SETS | $ 30.00 | $ - |
| CRANE MOB/DEMOB. | 1 | MOB/DEMOB | | | $ - | $ - |
| CRANE DAILY | 0 | DAYS | 0 | CRANES | $ - | $ - |
| HIGH PRESSURE SUPPLY PUMPS M/D | 2 | MOB/DEMOB | | | $ 1,150.00 | $ 2,300.00 |
| HIGH PRESSURE SUPPLY PUMPS DAILY | 2 | RENTALS | 0 | DAYS | $ 115.00 | $ - |
| HELICOPTER COST PLUS 15% | | | | | | $ 65,958.25 |
| PACKER TESTING HOURS | 54 | TESTS | 2 | HOURS | $ 175.00 | $ 18,900.00 |
| PACKER INFLATIONS | 54 | INFLATIONS | | | $ 75.00 | $ 4,050.00 |
| PACKER RENTAL | 3 | SETS | 13 | DAYS | $ 50.00 | $ 1,950.00 |
| EXCESS WATERLINE MATERIALS | 1 | LUMP | | | $ 3,000.00 | $ 3,000.00 |
| CORE BOXES 10 FEET PER BOX | 225 | BOXES | | | $ 8.00 | $ 1,800.00 |
| MISC. MATERIALS | 1 | LUMP | | | $ 2,500.00 | $ 2,500.00 |
| CASE OVERBURDON W/PVC FOR COBL & GEOPH | 450 | FEET | | | $ 5.00 | $ 2,250.00 |
| COBL MOB/DEMOB | 1 | M/D | | | $ 2,800.00 | $ 2,800.00 |
| COBL DATA ACQUISITION AND REPORT | 1350 | FEET | | | $ 21.00 | $ 28,350.00 |
| COBL RIG TIME | 0 | HRS | | | $ 150.00 | $ - |
| **TOTAL PROJECT** | | | | | | **$ 274,118.25** |
| | | | | | | |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| INCLUSIVE COST PER FOOT | | | | | | $ | 152.29 | $/FT |
| TOTAL PROJECT DAYS | | | | | | | 13.40 | DAYS |
| TOTAL CALENDAR DAYS | | | | | | | 13 | DAYS |

| HELICOPTER CHARGES | QUANTITY | UNIT | COST | | TOTAL |
|---|---|---|---|---|---|
| HELICOPTER MOB/DEMOB. | | LUMP | | | $ - |
| HELICOPTER DAILY | 15 | DAY | $ 3,600.00 | | $ 54,000.00 |
| HELICOPTER FUEL | | GALLON | | | $ - |
| HELICOPTER PILOT SUBSISTENCE | 13 | DAY | $ 85.00 | | $ 1,105.00 |
| HELICOPTER RENTAL CAR | | DAY | | | $ - |
| HELICOPTER FUEL TRUCK | 15 | DAY | $ 150.00 | | $ 2,250.00 |
| HELICOPTER RAW COST | | | | | $ 57,355.00 |

**DESCRIPTION**

| | | | | | |
|---|---|---|---|---|---|
| CLIENT | UEBLACKER ASSOC. | | | CLIENT CONTACT | |
| JOB NUMBER/NAME | 304-03-CO UNION POWERHOUSE | | | START DATE | |
| # OF DRILLS | 1 | RIGS | | DRILL RIGS | |
| PRICE PER MOB PER DRILL | $ 7,000.00 | PER RIG | | CASING ADV Y/N | |
| TOTAL FOOTAGE | 700 | FEET | | HQ3 CORING Y/N | |
| ESTIMATED FTG / SHIFT | 75 | FEET | | 4 1/4" AUGER Y/N | |
| EXPECTED RECOVERY | 100% | PERCENT | | DEPTH SHALLOWEST | |
| NUMBER OF HOLES TYPE 1 | 1 | HOLES | | DEPTH DEEPEST | |
| NUMBER OF HOLES TYPE 2 | | HOLES | | SPT INTERVALS | |
| HOURS PER MOVE TYPE 1(INCL. H20) | 10.0 | HOURS | | WATER SOURCE | |
| HOURS PER MOVE TYPE 2(INCL. H20) | | HOURS | | BY CLIENT Y/N | |
| HOURLY RATE | $ 175.00 | PER HOUR | | HAUL LENGTH | |
| FEET OF PIEZO | | FEET | | PUMP DISTANCE | |
| FEET OF INCLIN. CASING | | FEET | | LODGING | |
| FEET TO BE ABANDONED | 700 | FEET | 27 | UTILITIES BY CLIENT | |
| ABANDONMENT COST PER FOOT | $ 3.00 | PER FOOT | | TRAFFIC BY CLIENT | |
| # OF EXTRA MEN | 1 | MEN | | BIT CLAUSE Y/N | |
| HOURS OF STANDBY | | HOURS | | LOST TOOLS Y/N | |
| DAYS WORKED PER WEEK | 7 | DAYS | | BOXES BILLABLE | |
| PRICE PER MAN DAY SUBSIS. | 85 | DOLLARS | | MUD&MISC BILLABLE | |
| TRAVEL TIME ROUND TRIP PER DAY | 1.0 | HOURS | | | |
| # OF WATER TRUCKS | 1 | NUMBER | | | |
| # OF GROUT PLANTS | | NUMBER | | | |
| # OF BARGES | | NUMBER | | | |
| # OF JET BOATS | | NUMBER | | | |
| # OF TRAFFIC CONTROL SETS | | NUMBER | | | |
| # OF CRANES | | NUMBER | | | |
| # OF CRANE DAYS | | DAYS | | | |
| CRANE MOB DEMOB COST | | DOLLARS | | | |
| CRANE DAILY COST | | PER DAY | | | |
| # OF OTHER RENTAL EQUIPMENT | 1 | RENTALS | | | |
| OTHER RENTAL EQUIPMENT MOB DEMOB | $ 600.00 | DOLLARS | | | |
| OTHER RENTAL EQUIPMENT DAYS | 2 | DAYS | | | |
| OTHER RENTAL EQUIPMENT DAILY COST | $ 1,500.00 | DOLLARS | | | |
| NUMBER OF XTRA SPT SAMPLES | | SAMPLES | | | |
| NUMBER OF SHELBY OR PISTON SAMPLES | | TUBES | | | |
| NUMBER OF PACKER TESTS | 27 | TESTS | | | |
| NUMBER OF PACKER SETS | 1 | SETS | | | |
| AMOUNT OF WATERLINE OVER 1000' | | DOLLARS | | | |
| COST OF CORE BOXES | $ 8.00 | DOLLARS | | | |
| PRICE FOR MISC. MATERIALS | $ 550.00 | PER LUMP | | | |

UEBLACKER ASSOC.
304-03-CO UNION POWERHOUSE

| DESCRIPTION | QUANTITY | UNIT | QUANTITY EXT | UNIT | COST | TOTAL |
|---|---|---|---|---|---|---|
| M/D | 1 | RIG | | | $ 7,000.00 | $ 7,000.00 |
| DRILLING | 700 | FEET | | | $ 36.70 | $ 25,690.00 |
| MOVES TYPE 1 INCL H20 LINE | 10 | HOURS | | | $ 175.00 | $ 1,750.00 |
| MOVES TYPE 2 INCL H20 LINE | 0 | HOURS | | | $ 175.00 | $ - |
| EXTRA MEN | 1 | MEN | 21 | DAYS | $ 350.00 | $ 7,350.00 |
| PIEZO INSTALATION | 0 | FEET | | | $ 14.00 | $ - |
| INCLINOMETER CASING INSTALLATION | 0 | FEET | | | $ 20.00 | $ - |
| EXTRA SPTS | 0 | SAMPLES | | | $ 35.00 | $ - |
| SHELBY OR PISTON SAMPLES | 0 | SAMPLES | | | $ 50.00 | $ - |
| ABANDONMENT | 700 | FEET | | | $ 3.00 | $ 2,100.00 |
| SUBSISTENCE | 3 | MEN | 21 | DAYS | $ 85.00 | $ 5,355.00 |
| STANDBY | 0 | HOURS | | | $ 100.00 | $ - |
| TRAVEL TIME | 1 | CREW HRS | 21 | DAYS | $ 100.00 | $ 2,100.00 |
| WATER TRUCK | 21 | DAYS | 1 | TRUCKS | $ 200.00 | $ 4,200.00 |
| GROUT PLANT | 21 | DAYS | 0 | PLANTS | $ 40.00 | $ - |
| BARGE | 21 | DAYS | 0 | BARGES | $ 360.00 | $ - |
| JET BOAT | 21 | DAYS | 0 | BOATS | $ 250.00 | $ - |
| TRAFFIC CONTROL MATERIALS | 21 | DAYS | 0 | SETS | $ 30.00 | $ - |
| CRANE MOB/DEMOB. | 1 | MOB/DEMOB | | | $ - | $ - |
| CRANE DAILY | 0 | DAYS | 0 | CRANES | $ - | $ - |
| ROAD BUILDING EQUIPMENT MOB/DEMOB. | 2 | MOB/DEMOB | | | $ 690.00 | $ 1,380.00 |
| ROAD BUILDING EQUIPMENT DAILY | 1 | RENTALS | 2 | DAYS | $ 1,725.00 | $ 3,450.00 |
| HELICOPTER COST PLUS 15% | | | | | | |
| PACKER TESTING HOURS | 27 | TESTS | 2 | HOURS | $ 175.00 | $ 9,450.00 |
| PACKER INFLATIONS | 27 | INFLATIONS | | | $ 75.00 | $ 2,025.00 |
| PACKER RENTAL | 1 | SETS | 21 | DAYS | $ 50.00 | 1050 |
| EXCESS WATERLINE MATERIALS | 1 | LUMP | | | $ - | $ - |
| CORE BOXES 10 FEET PER BOX | 88 | BOXES | | | $ 8.00 | $ 704.00 |
| MISC. MATERIALS | 1 | LUMP | | | $ 550.00 | $ 550.00 |
| HYDRO FRAC EQUIP M/D | 1 | M/D | | | $ 6,000.00 | $ 6,000.00 |
| HYDRO FRAC TESTING HOURS | 54 | HOURS | | | $ 210.00 | $ 11,340.00 |
| CRUX RIG HOURS WHILE HYDRO FRAC TESTING | 0 | HOURS | | | $ 175.00 | $ - |
| CASE OVERBURDON W/PVC FOR COBL & GEOPH | 25 | FEET | | | $ 5.00 | $ 125.00 |
| COBL MOB/DEMOB | 1 | M/D | | | $ 2,800.00 | $ 2,800.00 |
| COBL DATA ACQUISITION AND REPORT | 675 | FEET | | | $ 21.00 | $ 14,175.00 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| COBL RIG TIME | 0 | HRS | | | $ 150.00 | $ - | |
| **TOTAL PROJECT** | | | | | | **$ 108,594.00** | |
| | | | | | | | |
| INCLUSIVE COST PER FOOT | | | | | | $ 155.13 | $/FT |
| TOTAL PROJECT DAYS | | | | | | 21.13 | DAYS |
| TOTAL CALENDAR DAYS | | | | | | 21 | DAYS |

**DESCRIPTION**

| | | | | | | |
|---|---|---|---|---|---|---|
| CLIENT | UEBLACKER ASSOC. | | | CLIENT CONTACT | |
| JOB NUMBER/NAME | 304-03-CO UNION SADDLE DAM | | | START DATE | |
| # OF DRILLS | 1 | RIGS | | DRILL RIGS | |
| PRICE PER MOB PER DRILL | $ 7,000.00 | PER RIG | | CASING ADV Y/N | |
| TOTAL FOOTAGE | 1,200 | FEET | | HQ3 CORING Y/N | |
| ESTIMATED FTG / SHIFT | 85 | FEET | | 4 1/4" AUGER Y/N | |
| EXPECTED RECOVERY | 100% | PERCENT | | DEPTH SHALLOWEST | |
| NUMBER OF HOLES TYPE 1 | 8 | HOLES | | DEPTH DEEPEST | |
| NUMBER OF HOLES TYPE 2 | | HOLES | | SPT INTERVALS | |
| HOURS PER MOVE TYPE 1(INCL. H20) | 2.5 | HOURS | | WATER SOURCE | |
| HOURS PER MOVE TYPE 2(INCL. H20) | | HOURS | | BY CLIENT Y/N | |
| HOURLY RATE | $ 175.00 | PER HOUR | | HAUL LENGTH | |
| FEET OF PIEZO | | FEET | | PUMP DISTANCE | |
| FEET OF INCLIN. CASING | | FEET | 130 | LODGING | |
| FEET TO BE ABANDONED | 1,200 | FEET | 5.2 | UTILITIES BY CLIENT | |
| ABANDONMENT COST PER FOOT | $ 3.00 | PER FOOT | 41.6 | TRAFFIC BY CLIENT | |
| # OF EXTRA MEN | 1 | MEN | | BIT CLAUSE Y/N | |
| HOURS OF STANDBY | | HOURS | | LOST TOOLS Y/N | |
| DAYS WORKED PER WEEK | 7 | DAYS | | BOXES BILLABLE | |
| PRICE PER MAN DAY SUBSIS. | 85 | DOLLARS | | MUD&MISC BILLABLE | |
| TRAVEL TIME ROUND TRIP PER DAY | 1.0 | HOURS | | | |
| # OF WATER TRUCKS | 1 | NUMBER | | | |
| # OF GROUT PLANTS | | NUMBER | | | |
| # OF BARGES | | NUMBER | | | |
| # OF JET BOATS | | NUMBER | | | |
| # OF TRAFFIC CONTROL SETS | | NUMBER | | | |
| # OF CRANES | | NUMBER | | | |
| # OF CRANE DAYS | | DAYS | | | |
| CRANE MOB DEMOB COST | | DOLLARS | | | |
| CRANE DAILY COST | | PER DAY | | | |
| # OF OTHER RENTAL EQUIPMENT | | RENTALS | | | |
| OTHER RENTAL EQUIPMENT MOB DEMOB | | DOLLARS | | | |
| OTHER RENTAL EQUIPMENT DAYS | | DAYS | | | |
| OTHER RENTAL EQUIPMENT DAILY COST | | DOLLARS | | | |
| NUMBER OF XTRA SPT SAMPLES | | SAMPLES | | | |
| NUMBER OF SHELBY OR PISTON SAMPLES | | TUBES | | | |
| NUMBER OF PACKER TESTS | 42 | TESTS | | | |
| NUMBER OF PACKER SETS | 1 | SETS | | | |
| AMOUNT OF WATERLINE OVER 1000' | | DOLLARS | | | |
| COST OF CORE BOXES | $ 8.00 | DOLLARS | | | |
| PRICE FOR MISC. MATERIALS | $ 1,000.00 | PER LUMP | | | |

UEBLACKER ASSOC.
304-03-CO UNION SADDLE DAM

| DESCRIPTION | QUANTITY | UNIT | QUANTITY EXT | UNIT | COST | | TOTAL | |
|---|---|---|---|---|---|---|---|---|
| M/D | 1 | RIG | | | $ | 7,000.00 | $ | 7,000.00 |
| DRILLING | 1200 | FEET | | | $ | 32.40 | $ | 38,880.00 |
| MOVES TYPE 1 INCL H20 LINE | 20 | HOURS | | | $ | 175.00 | $ | 3,500.00 |
| MOVES TYPE 2 INCL H20 LINE | 0 | HOURS | | | $ | 175.00 | $ | - |
| EXTRA MEN | 1 | MEN | 25 | DAYS | $ | 350.00 | $ | 8,750.00 |
| PIEZO INSTALATION | 0 | FEET | | | $ | 14.00 | $ | - |
| INCLINOMETER CASING INSTALLATION | 0 | FEET | | | $ | 20.00 | $ | - |
| EXTRA SPTS | 0 | SAMPLES | | | $ | 35.00 | $ | - |
| SHELBY OR PISTON SAMPLES | 0 | SAMPLES | | | $ | 50.00 | $ | - |
| ABANDONMENT | 1200 | FEET | | | $ | 3.00 | $ | 3,600.00 |
| SUBSISTENCE | 3 | MEN | 25 | DAYS | $ | 85.00 | $ | 6,375.00 |
| STANDBY | 0 | HOURS | | | $ | 100.00 | $ | - |
| TRAVEL TIME | 1 | CREW HRS | 25 | DAYS | $ | 100.00 | $ | 2,500.00 |
| WATER TRUCK | 25 | DAYS | 1 | TRUCKS | $ | 200.00 | $ | 5,000.00 |
| GROUT PLANT | 25 | DAYS | 0 | PLANTS | $ | 40.00 | $ | - |
| BARGE | 25 | DAYS | 0 | BARGES | $ | 360.00 | $ | - |
| JET BOAT | 25 | DAYS | 0 | BOATS | $ | 250.00 | $ | - |
| TRAFFIC CONTROL MATERIALS | 25 | DAYS | 0 | SETS | $ | 30.00 | $ | - |
| CRANE MOB/DEMOB. | 1 | MOB/DEMOB | | | $ | - | $ | - |
| CRANE DAILY | 0 | DAYS | 0 | CRANES | $ | - | $ | - |
| OTHER RENTAL EQUIPMENT MOB/DEMOB. | 0 | MOB/DEMOB | | | $ | - | $ | - |
| OTHER RENTAL EQUIPMENT DAILY | 0 | RENTALS | 0 | DAYS | $ | - | $ | - |
| HELICOPTER COST PLUS 15% | | | | | | | | |
| PACKER TESTING HOURS | 42 | TESTS | 2 | HOURS | $ | 175.00 | $ | 14,700.00 |
| PACKER INFLATIONS | 42 | INFLATIONS | | | $ | 75.00 | $ | 3,150.00 |
| PACKER RENTAL | 1 | SETS | 25 | DAYS | $ | 50.00 | $ | 1,250.00 |
| EXCESS WATERLINE MATERIALS | 1 | LUMP | | | $ | - | $ | - |
| CORE BOXES 10 FEET PER BOX | 150 | BOXES | | | $ | 8.00 | $ | 1,200.00 |
| MISC. MATERIALS | 1 | LUMP | | | $ | 1,000.00 | $ | 1,000.00 |
| CASE OVERBURDON W/PVC FOR COBL AND GEO | 200 | FEET | | | $ | 5.00 | $ | 1,000.00 |
| COBL MOB/DEMOB | 1 | M/D | | | $ | 2,800.00 | $ | 2,800.00 |
| COBL DATA ACQUISITION AND REPORT | 1000 | FEET | | | $ | 21.00 | $ | 21,000.00 |
| **TOTAL PROJECT** | | | | | | | **$** | **121,705.00** |
| | | | | | | | | |
| INCLUSIVE COST PER FOOT | | | | | | | $ | 101.42 $/FT |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| TOTAL PROJECT DAYS | | | | | | 24.52 | DAYS |
| TOTAL CALENDAR DAYS | | | | | | 25 | DAYS |