SUBMITTAL #1

Feb 14, 1986

REPORT

GEOTECHNICAL INVESTIGATION
PROPOSED FLOODWATER RETARDING DIKE
UNIT IV OF AHWATUKEE
MARICOPA COUNTY, ARIZONA

FOR PRESLEY DEVELOPMENT COMPANY

D.E. CLARK

Geotechnical Engineer

Registered Professional Engineer
CERTIFICATE NO.
04905
DONALD E.
CLARK
Date Signed 1/30/86
ARIZONA, U.S.A.

REPORT

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D.E. CLARK

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10329 Campana Drive
Sun City, Arizona 85351
(602) 972-0404

January 30, 1986
Our Job No. 15A

Brooks, Hersey & Associates, Inc.
5246 South 40th Street
Phoenix, Arizona 85040

Attention: William L. Clark
Project Engineer

Gentlemen:

This letter transmits our report of this date titled "Geotechnical Investigation, Proposed Floodwater Retarding Dike, Unit IV of Ahwatukee, Maricopa County, Arizona, For Presley Development Company."

The work was done in accordance with our proposal of November 15, 1985.

The report develops four alternate sections for a dike of roller compacted concrete, and develops suggested construction specifications. Since RCC is a fairly new technology, the main concern has been construction procedures. The report can be used to negotiate a construction contract, with the intent that the contractor can be asked to contribute thoughts on alternate designs and construction procedures that would particularly fit his equipment and experience.

The report completes our presently contemplated services. If you so desire, we would be pleased to participate in negotiations with a contractor.

Yours very truly,

D. E. CLARK
Geotechnical Engineer

Donald E. Clark

Donald E. Clark

DEC/dc

(6 sent)

Registered Professional Engineer (Civil)
CERTIFICATE NO.
04905
DONALD E.
CLARK
Date Signed 1/30/86
ARIZONA, U.S.A.

CONTENTS

January 30, 1986

REPORT
GEOTECHNICAL INVESTIGATION
PROPOSED FLOODWATER RETARDING DIKE
UNIT IV OF AHWATUKEE
MARICOPA COUNTY, ARIZONA
FOR PRESLEY DEVELOPMENT COMPANY

INTRODUCTION

This report presents results of our geotechnical investigation for a proposed floodwater retarding dike in Unit IV of Ahwatukee, Maricopa County, Arizona. Plate 1, Vicinity Map, shows the location of the site relative to streets of Ahwatukee. Plate 2, Plot Plan, shows ground contours, existing improvements, and proposed dike locations.

The purposes of the investigation were to evaluate depth of overburden, to evaluate suitability of bedrock to support a dike of roller compacted concrete, to develop alternate dike sections, and to develop suggested construction specifications.

The report describes our understanding of site conditions and proposed construction, describes the investigation services, interprets subsurface conditions, discusses four alternate sections, discusses construction procedures, and presents our conclusions.

SITE CONDITIONS

The site of the dike is a steep-sided notch cut through a bedrock ridge by a wash that carries water only immediately after rainfall. Here the bottom of the wash is roughly 30 feet wide and the top is roughly 100 feet wide. General horizontal to vertical slope ratios are approximately 1.6:1 on the west abutment and approximately 1.1:1 on the east abutment.

The sides of the wash are bedrock, the bottom is covered with alluvial sand and boulders, and the surrounding area is covered with colluvial bouldery soils. There is some rather sparse desert vegetation in the surrounding area; virtually none in the sides or bottom of the wash.

The wash is considerably wider upstream from the dike site. The entrance to a concrete lined channel is just downstream. This newly constructed channel lining is designed to carry a flow of 1,600 cubic feet per second.

PROPOSED CONSTRUCTION

The dike was initially conceived as an earth embankment with a 60-foot-wide crest intended to carry a road. Later it was decided that the dike would be built of roller compacted concrete, would be placed along an alignment where the crest length would be minimum, and would be designed with the minimum section that would be stable. The plot plan shows both alignments.

Flow considerations dictate a crest elevation of 1379 feet. The channel bottom is at elevation 1355 feet, so the height of the dike is to be 24 feet.

A conduit through the dike is sized to pass the 1,600 cfs flow for which the downstream channel was designed. With this size of outlet conduit, a 100-year, one-hour rainfall event will create a reservoir, and a maximum probable rainfall event will go over the dike. The dike will need to back up water for periods up to two hours.

The ends of the dike are to stop where the crest intersects the natural ground at elevation 1379 feet. To handle the overflow condition, the dike is to be extended on each end with one-foot-wide cut-off walls taken down to bedrock. The combined length of the two cut-off walls is estimated to be about 30 feet.

INVESTIGATION SERVICES

Bedrock is exposed over much of the area that will be covered by the dike. The dike will need to back up water only for periods up to two hours. Leakage is of concern only if it can erode the dike or supporting bedrock. Under these conditions, investigation needs are considerably different than would be the case for a dike that is to retain water for a prolonged period of time. The reasons for borings are absent: the strength of the foundation bedrock is sufficiently known, and field permeability tests are not needed.

Kenneth M. Euge, R.G., of the firm Geological Consultants, performed a geologic reconnaissance of the site and surrounding area, and performed a seismic refraction survey of the site. His report is presented as Appendix A, Geologic Reconnaissance and Seismic Refraction Survey. Salient features are summarized in the following section of this report.

We performed analyses using procedures outlined in Pages 329 through 339 of the Bureau of Reclamation's book "Design of Small Dams," 2nd edition, revised 1977. The forces considered were horizontal water pressure on the upstream face, vertical water pressure upward from the foundation, vertical and lateral earthquake loads, and weight of the structure. We considered the forces' effects on overturning, sliding, and overstressing within the structure or foundation. Earthquake and water pressures were not considered to act at the same time. The designs were governed by overturning: weight of the structure vs. water pressure on the upstream face and water pressure upward from the foundation or within the structure at various elevations. The computation sheets are presented as Appendix B, Computations.

Also the effect of the upstream fill on the stability of the structure has to be included if the downstream fill is to be allowed to wash out during overtopping.

D.E. CLARK

INTERPRETATION OF SUBSURFACE CONDITIONS

This interpretation of subsurface conditions is based on our field observations and the contents of Appendix A.

The site has granitic bedrock with tight joints and fractures. There are no minerals likely to go into solution upon wetting, and the joints are unlikely to leak, especially considering the minimal periods of time when the dike is to back up water.

The bedrock is weathered near the surface, and is the source material for the bouldery colluvium that is present above the bedrock.

To reach bedrock sufficiently unweathered to support the dike, it looks necessary to strip a maximum of roughly six feet, and an average of roughly four feet of colluvium and alluvium.

There is no indication of springs. After times when the wash carries water, there will be water in the alluvium in the bottom of the wash.

Since there is only one foundation material, and much of that is exposed, interpretative error relative to the foundation material is less likely than with most investigations. Any error is likely to be in the evaluation of the depths of alluvium and colluvium that need to be removed.

ALTERNATE SECTIONS

We have developed four alternate dike sections. Each has a base extending from the bedrock up to an elevation of 1355 feet, the present channel bottom, creating a level pad about 50 feet long between the exposed bedrock in the stripped abutments. In each case, the base is made three feet wider than the dike section because the outer 18 inches or so cannot be well compacted, and because some extra space is needed as a safety factor against erosional undercutting of the dike. The cut-off walls that extend the dike are not shown. The outlet conduit is shown with dimensions of six by 15 feet, which may change during final design. The four sections have the following general features:

> Alternate 1 is designed with vertical sides, 15 feet apart, the minimum width at which this alternate has a computed safety factor of 1.2 against overturning. This alternate could be built between exterior tied-back walls, between cast-in-place curbs, between temporary forms (RCC imposes little pressure on forms because it sets up between placement of lifts), or between earth fills brought up as the RCC is placed.
>
> Alternate 2 has one vertical side and one side with a horizontal:vertical slope ratio of 0.8:1, the steepest slope ratio at which RCC is normally built without forms. The crest is eight feet wide because this is assumed to be the absolute minimum width on which construction equipment can work.
>
> Alternate 3 also has one vertical side, but the 0.8:1 side comes up to a top width of eight feet, six feet below the crest elevation, and the upper six feet is a cap of poured concrete six by eight feet in section. why not RCC ?
>
> Alternate 4 has both sides sloped at 0.8:1, and comes up to a crest width of eight feet.

The sections are illustrated on Plate 3, Longitudinal Section, and Plates 4 through 7, Cross Sections. The cross sections also show design estimates of material quantities.

CONSTRUCTION PROCEDURES

Mix Design

The contractor can be responsible for the mix design. we are thinking in terms of 1.5-inch-maximum size aggregate, no more that 15 percent fines, 60 percent cement and 40 percent Class F flyash, and voids in the coarse aggregate filled or nearly filled with mortar. We further suggest that the mix batches be compacted in a steel, six inch diameter mold, and be compacted with Modified Proctor equipment and compactive energy (ASTM D 1557).

The selected mix would be a workable mix which, when compacted to 95 percent compaction, has a compressive strength of 1,500 pounds per square inch at an age of 90 days (simulated by accelerated curing). This strength is higher than is needed for structural purposes, but is intended to result in a mix not excessively subject to weathering or erosion, and one that is not sensitive to some fluctuations in strength due to possible variations in cement content, mixing time, or compaction effectiveness.

?

Site Preparation

Stripping will generate bouldery and non-bouldery materials. Just upstream, where the channel is considerably wider, there is room to line the banks with the bouldery strippings, thus lessening the likelihood of erosion and instability of the natural banks when they become the edge of the reservoir. The non-bouldery strippings can be stockpiled for later use as backfill against the base of the dike.

After stripping, the exposed surface will need to be cleaned. Depending on the roughness of the exposed surface, the cleaning may be done with brushes or, more likely, with an air or water jet.

Just before placing each lift of RCC in the base and dike, the cleaned rock surface will need to be moistened and painted with grout to fill any cracks and aid bonding of the RCC to the rock.

Base Construction

We have in mind delivering the RCC to the east abutment by dump truck, chuting it down to the working area, moving it with a loader, and spreading it with a bulldozer, possibly laser controlled. Compared with pavements, levelness and waviness of lift surfaces are relatively unimportant in this application.

We think of the base construction having a secondary purpose as a test section from which to develop construction procedures and evaluate field strength of the RCC (as against the laboratory strength found during mix design).

We are thinking in terms of lifts with a compacted thickness of about eight inches, with initial compaction by the tracks of the spread bulldozer, and with final compaction by a flat-wheel roller. Ordinarily a roller for RCC compaction weighs 10 tons, is operated in a vibrating mode for about four passes, and is finished in a static mode with about two passes. With the initial track compaction, and the thin lifts, vibration may not be necessary to achieve 95 percent compaction, and a lighter weight roller may be satisfactory.

Depending on the air temperature, we would allow 1 to 1 1/2 hours between mixing in the water and completing the compaction. If necessary a set retarder can extend these times. We see no need for any other additives.

Even though some leakage of the dike would not impair its functioning, extra care is needed to aid bonding between lifts. The lift surfaces need to be protected, to be kept free of dirt and other contaminants, to be kept moist, and to be painted with grout if more that about five hours elapses between placement of lifts.

Lateral support for the base construction can come from backfilling as the construction proceeds, but it will still be necessary for the roller to stay a foot or so away from the edge.

Dike Construction

We see the dike being built in the same general manner as the base, using the experience developed and tests made as the base is built. We are advised that an unsupported side of the dike can be built with a horizontal:vertical slope ratio of 0.8:1. With equipment that can work in the small space available on the dike, we have not found a way to build a an unsupported vertical side.

Immediately after the dike is complete, it needs to be water cured continuously for at least seven days.

Observation and Testing

Observation of the stripping is needed to verify that sufficiently unweathered rock has been exposed, and that cleaning has been adequately done.

The moisture content of the RCC material needs to be tested as it is delivered, and the density needs to be tested to establish when sufficient compaction has been achieved. A nuclear moisture-density device can do these things quickly, operated in the backscatter mode for moisture testing and in the direct transmission mode for moisture or density testing.

At some time during construction of the base, cores will need to be taken and tested (with accelerated curing) for verification that the field procedures are resulting in concrete with a compressive strength of at least 1,500 pounds per square inches at 90 days. Once the procedures are established we would plan only to observe moistening and painting of the rock surface with grout just before each lift of RCC is placed, and to observe the spreading and compaction procedures, with record cores taken and tested at random every few lifts.

Specifications

Appendix C, Suggested Specifications, implements the things considered above, using where feasible the Standard Specifications of the Maricopa Association of Governments. As presented, the specifications consider only actual construction aspects of the work; things such as measurement and payment have not been included. The suggested specifications should be subject to modification during contract negotiations.

CONCLUSIONS

In our opinion:

- after stripping an average of roughly four feet of alluvium and colluvium, the bedrock at this site is suitable to support the RCC dike without appreciable settlement or leakage.
- Any of the proposed sections will be stable.
- Other sections and construction procedures should be considered if they better fit the equipment and experience of the selected contractor.

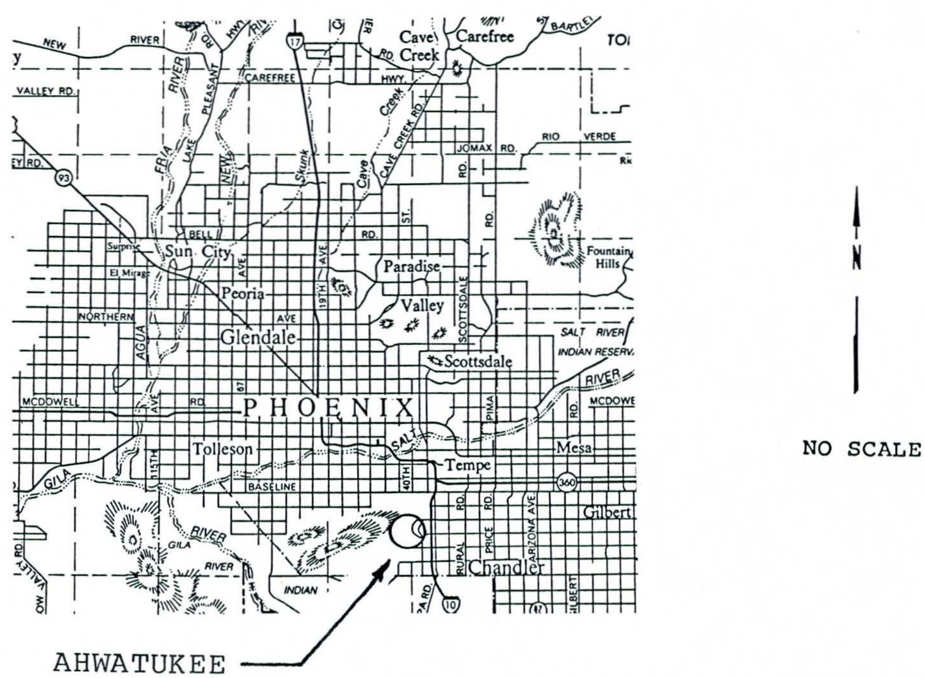
CLOSURE

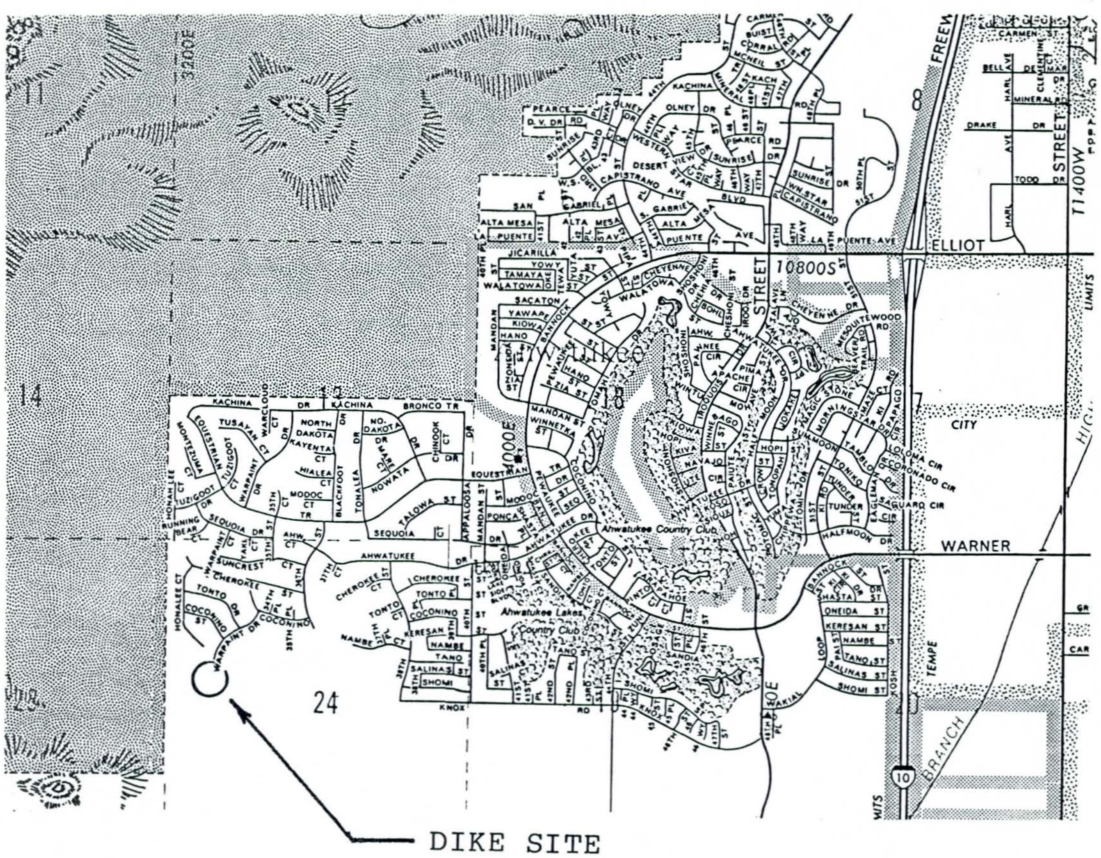
The seven plates and three appendices are attached and complete the report.

DEC/dc

JOB NO. 1517 CLIENT/OWNER Brooks Hersey/Presley Dev. Co JOB LOCATION Ahwatukee

PREPARED BY: D. Clark DRAFTED BY: D. Clark CHECKED BY: D. Clark DATE 1/29/86



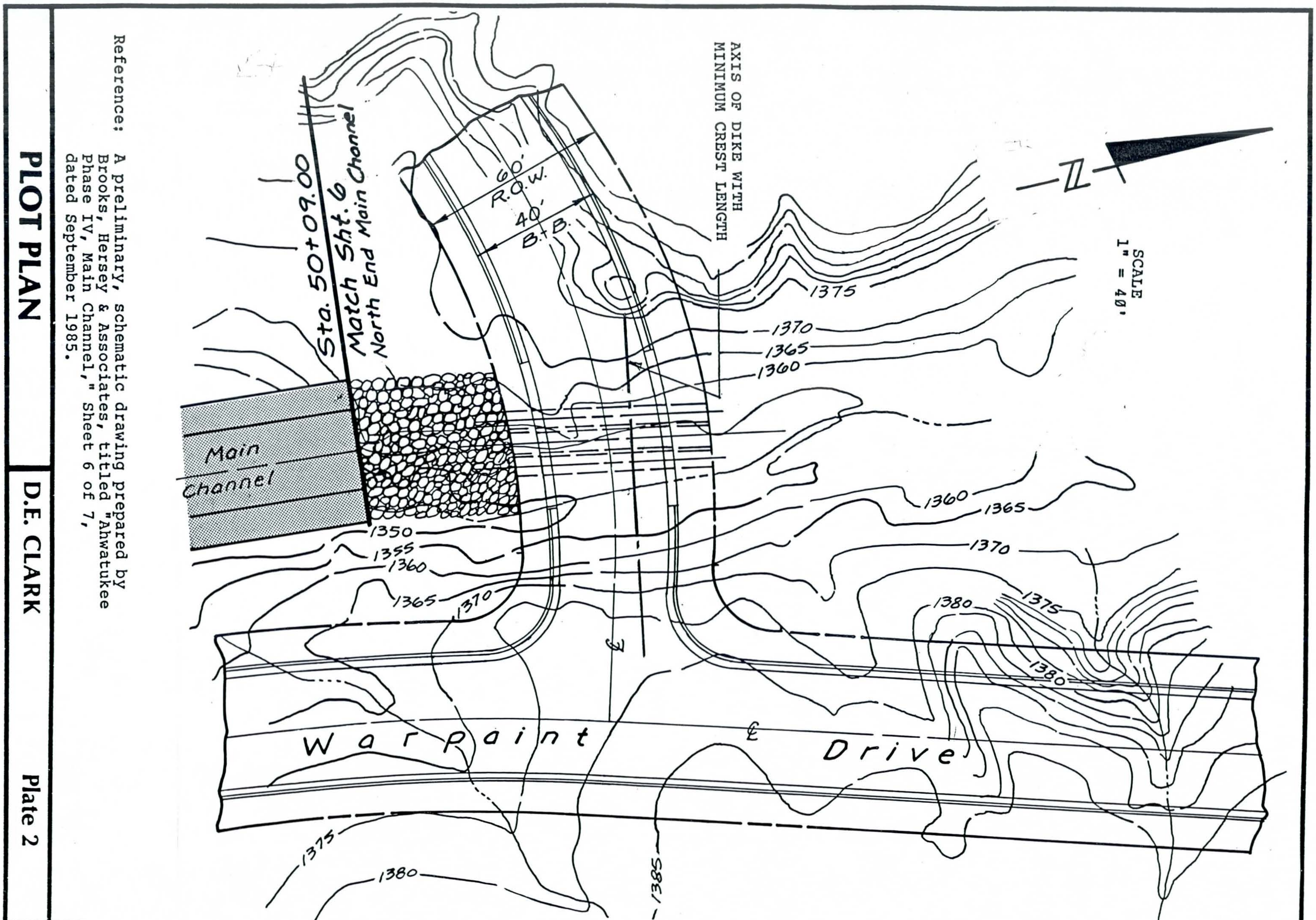


REFERENCE: Metropolitan Phoenix Street Atlas, Wide Wide World of Maps, Inc., 1985

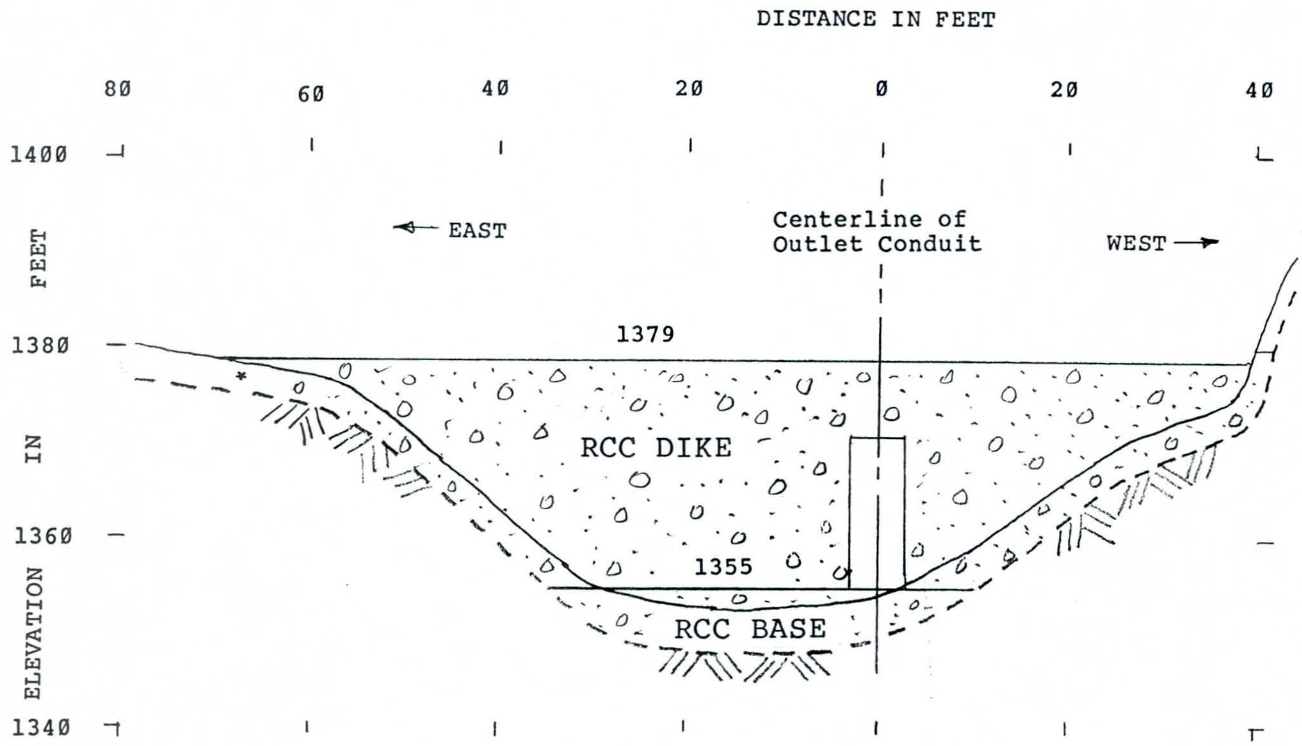
VICINITY MAP	D.E. CLARK	Plate 1

JOB NO. 13A CLIENT/OWNER Brooks Hersey / Presley Dev. Co. JOB LOCATION Ahwatukee

PREPARED BY: D. Clark DRAFTED BY: D. Clark CHECKED BY: D. Clark DATE 1/29/86



JOB NO. ISH CLIENT/OWNER Bro...s Hersey / Presley Dev. Co. JOB LOCATION Minwaukee
PREPARED BY: D. Clark DRAFTED BY: D. Clark CHECKED BY: D. Clark DATE 1/29/86



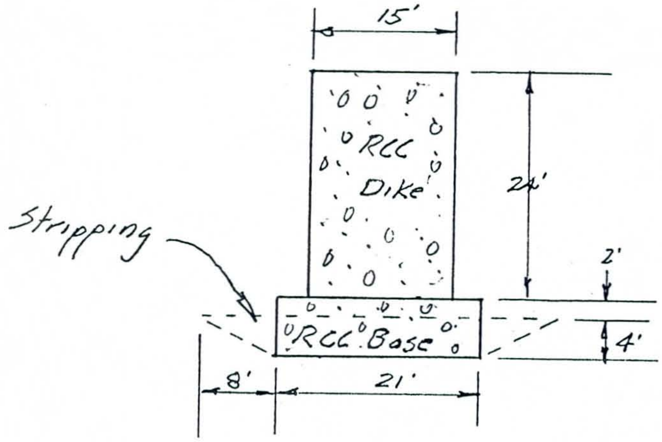
* The stripping depth averages roughly four feet

LONGITUDINAL SECTION

SCALE 1" = 20'

LONGITUDINAL SECTION	D.E. CLARK	Plate 3

JOB NO. 13H CLIENT/OWNER Brooks Hersey/Presley Dev Co. JOB LOCATION Milwaukee
PREPARED BY: D. Clark DRAFTED BY: D. Clark CHECKED BY: D. Clark DATE 1/29/86



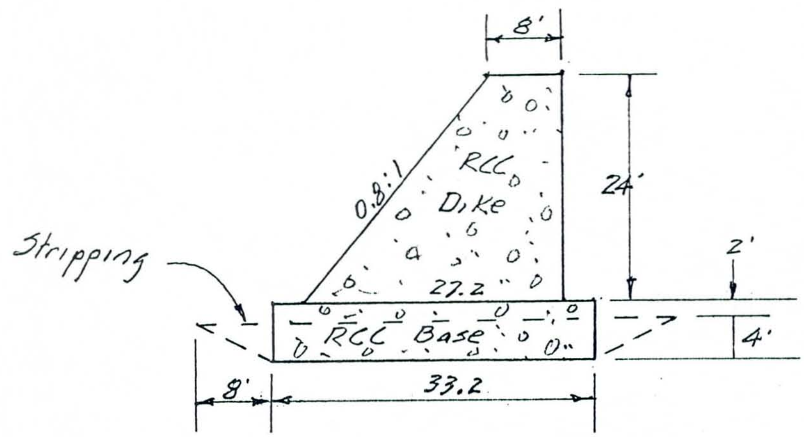
CROSS SECTION

ALTERNATE 1

VOLUME ESTIMATES

The following estimates include only the dike, not the conduit or cut-offs at the ends. The estimates were made for comparisons of alternate designs, and may not be sufficiently accurate for bid purposes.

Volume of Stripping	500 cu yd
Volume of RCC Base	200 cu yd
Volume of RCC Dike	1,000 cu yd



CROSS SECTION

ALTERNATE 2

VOLUME ESTIMATES

The following estimates include only the dike, not the conduit or cut-offs at the ends. The estimates were made for comparisons of alternate designs, and may not be sufficiently accurate for bid purposes.

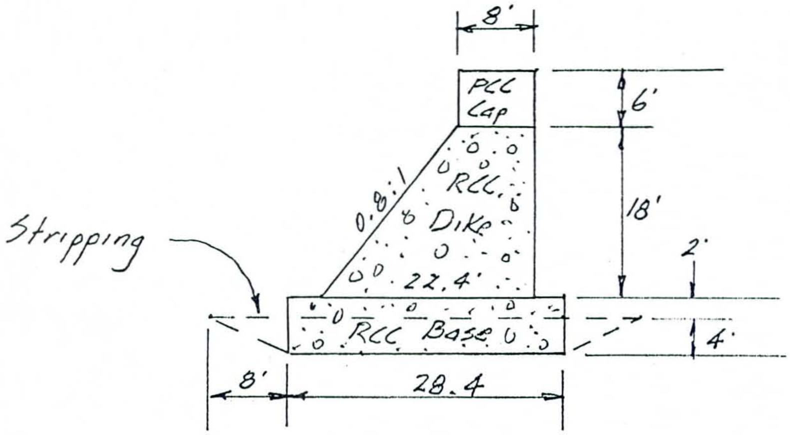
Volume of Stripping	700 cu yd
Volume of RCC Base	300 cu yd
Volume of RCC Dike	1,200 cu yd

JOB NO. 104 CLIENT/OWNER Brooks Persey/Presley Dev Co JOB LOCATION [illegible] DATE 1/29/86

PREPARED BY: D. Clark DRAFTED BY: D. Clark CHECKED BY: D. Clark

JOB NO. 124 CLIENT/OWNER Brooks Mersey/Presley Dev Co. JOB LOCATION Mhwatukee

PREPARED BY: D. Clark DRAFTED BY: D. Clark CHECKED BY: D. Clark DATE 1/29/86



CROSS SECTION

ALTERNATE 3

This Alternate is almost as economical as the one proposed and it is much more stable!

VOLUME ESTIMATES

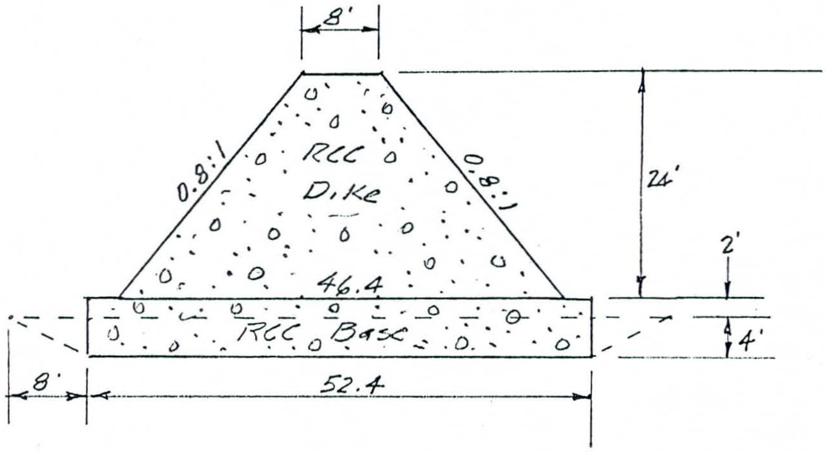
The following estimates include only the dike, not the conduit or cut-offs at the ends. The estimates were made for comparisons of alternate designs, and may not be sufficiently accurate for bid purposes.

Volume of Stripping	600 cu yd
Volume of RCC Base	300 cu yd
Volume of RCC Dike	700 cu yd
Volume of PCC Cap	200 cu yd

is there some reason why this could not be RCC ?

CROSS SECTION - ALT 3 | D.E. CLARK | Plate 6

JOB NO. 15A CLIENT/OWNER Brooks Mersey/Presley Dev Co JOB LOCATION Milwaukee
PREPARED BY: D. Clark DRAFTED BY: D. Clark CHECKED BY: D. Clark DATE 1/29/86



CROSS SECTION

ALTERNATE 4

VOLUME ESTIMATES

The following estimates include only the dike, not the conduit or cut-offs at the ends. The estimates were made for comparisons of alternate designs, and may not be sufficiently accurate for bid purposes.

Volume of Stripping	1,000 cu yd
Volume of RCC Base	500 cu yd
Volume of RCC Dike	1,800 cu yd

APPENDIX A

GEOLOGIC RECONNAISSANCE

AND

SEISMIC REFRACTION SURVEY

GEOLOGICAL CONSULTANTS

Kenneth M. Euge, R.G.

December 17, 1985

Mr. Donald E. Clark, P.E.
D.E. Clark
Geotechnical Engineer
10329 Campana Drive
Sun City, Arizona 85351

Subject: Geologic Reconnaissance and Seismic Refraction Survey, Proposed Dike Site
Unit IV of Ahwatukee, Maricopa County, Arizona
D.E. Clark Job No. 15A
Geological Consultants Project No. 85-129

Dear Mr. Clark:

As requested by you during our meeting at the subject site on November 21, 1985 and agreed to in our Independent Contractor Agreement of November 22, 1985, I have completed the geological reconnaissance and seismic survey at the proposed floodwater retarding dike site. The purpose of this study was to evaluate surficial site conditions from a geological standpoint and to characterize the subsurface bedrock using indirect seismic techniques.

Scope of Investigation

This study included the following activities:

- visual examination of the property;
- geologic data gathering from bedrock exposures at the site;
- five seismic refraction surveys;
- preparation and analysis of geologic and seismic profiles; and
- preparation of this letter report presenting my findings, conclusions and recommendations.

2822 West Northern Avenue, Suite B • Phoenix, Arizona 85021 • Telephone 602-864-1888

Proposed Construction

According to the plan sheet provided by you, an extension of a proposed residential street, supported on an earth fill, will traverse the existing natural drainage channel. The earth fill is to be designed to function on a flood retarding structure with large diameter outlet pipes to provide for controlled release of the probable maximum flood flow downstream into a concrete lined floodway. The configuration of the dike is not finalized, however the dike will be founded in competent bedrock. The maximum pool water level (calculated by others) is at elevation 1375 feet.

Site Conditions

The topography of the site consists of steeply sloping terrain bisected by a deeply entrenched ephemeral stream channel. Maximum relief at the site measured from the hilltop on the east side of the channel to the concrete-lined drainage floodway is about 120 feet. Relief in the immediate dike area measured from the channel bottom with nearly vertical side slopes to the nearest slope break above pool elevation is about 30 feet. Surface drainage is to the east and west over the slopes into the major southerly flowing stream channels. Vegetation consists of Palo Verde and Mesquite trees, creosotebush, cactus and grasses. Moss and lichen grow on the north faces of bouldery outcrops that are present throughout the site area. (Drawing No. 1)

Geologic Setting

The site area is underlain by granitic bedrock of mid-Tertiary Age which is covered locally by a thin verneer of colluvial soils. Recent alluvium is found in the main drainage channel. Refer to Drawing No. 2.

Granitic Bedrock

The bedrock, named the South Mountain granodiorite (Reynolds, 1985)[1] exposed at the site, consists of strongly foliated, hard, very highly fractured light gray granodiorite, with localized small inclusion blocks of very dark, fine grained igneous rock. The mineral composition is about 20% quartz, 35-40% plagioclase, 25% potassium feldspar and 10-15% biotite. The rock is deeply weathered in-place causing the unit to decompose and exhibit a reddish brown color in surface exposures. The weathering is caused by the alteration of the feldspar minerals to clays and the ferromagnesium minerals to the patina on the rock surface.

The fractures and joints exhibit close to very close spacing ranging from < 0.1 foot to about 1 foot. Joint separation is negligible except where small blocks have experienced displacements.

The predominent orientation of foliation is N44°E to N82°E dipping 15° to 50° to the northwest. Two prominent joint sets are present, one striking N34°E to N68°E dipping 44° to 88° southeast and the other set N44°W to N1°E dipping 22° northeast to vertical to 19° southwest.

Colluvial Soils

The colluvial soils consist of brown to light red brown coarse grained, unconsolidated silty sands (SM) derived from the in-place weathering and degradation of the granitic bedrock. The angular sands thinly verneer the weathered bedrock surface.

[1]Reynolds, S.J.; 1985; Geology of the South Mountains, Central Arizona; Bulletin 195; Arizona Bureau of Geology and Mineral Technology, Geological Survey Branch; 61 pp.

Alluvium

Alluvial deposits are confined to the drainage channel area. The alluvial soils are light brown to tan unconsolidated, loose, clean sands (SP).

Seismic Refraction Survey

In general, seismic wave velocities are related to the hardness, consolidation and density of subsurface materials. By approximating a materials insitu characteristic velocities correlations to the excavatability of various soils and rock may be made with reasonable levels of confidence.

Soils and loose surface materials generally have seismic velocities ranging from 500 to 1200 feet per second (fps). Velocities of 1200 to 3000 fps are characteristic of moderately hard dense sediments and highly weathered bedrock. Velocities of 3000 to 5000 fps are typical of very dense, hard cemented soils and moderately competent bedrock that would require heavy equipment to excavate. Velocities in excess of 5000 fps would normally require blasting in homogeneous rock but, if the rock is jointed it may be possible to rip with heavy equipment.

Seismic refraction surveys were made at the site in order to characterize the insitu rock quality or soundness, and to approximate the depth to hard rock and the thickness of alluvium in the channel bottom. A total of five forward and reverse traverses were made to generate seismic velocity time-travel curves from which velocity profiles could be generated (Drawings 3 & 4).

The travel-time data were obtained using a Bison Instruments, Inc. Model 1570C Signal Enhancement Seismograph. Seismic arrivals were detected with a high sensitivity, hum-bucking low impedence design, responsive to 5-2000 hz. The seismic energy was produced by repeated impacts of a 10 pound sledge hammer on an aluminum striking plate.

The seismic refraction data are presented in Table 1 and in Drawings 3 & 4.

Conclusions and Recommendations

Based on my review of the plans provided, the geologic reconnaissance, the seismic refraction survey and the analysis of the geologic data, the following conclusions and recommendations can be provided regarding site bedrock and soils:

1. Based on limited field exploration, that consisted of surficial geological reconnaissance and seismic refraction surveys, the site appears geologically suitable and technically feasible for the proposed project.

2. The bedrock consists of coarse crystalline granodiorite that is strongly foliated and jointed throughout. The bedrock exposure shows extensive in-place weathering and decomposition of the rock to a coarse silty sand colluvium that locally mantle the bedrock to shallow depths.

3. Geophysical seismic refraction surveys show low velocity (750 ft/sec to 1400 ft/sec) soil horizon extended from the ground surface to depths ranging from about 2 feet to 5 feet; moderate velocity (2500 ft/sec to 7500 ft/sec) materials indicative of variably weathered bedrock is found to depths of 12 to 26 feet below ground surface where high velocity (9000 ft/sec to 15,000 ft/sec) sound bedrock is found.

TABLE 1

Seismic Refraction Survey Data

Traverse #	ST-1		ST-2		ST-3		ST-4		ST-5		Interp.
	F	R	F	R	F	R	F	R	F	R	
Length (feet)	120	120	100	100	100	100	100	100	100	100	
First Layer Velocity-V (ft./sec.)	1200	800	1400	1200	1200	1000	950	750	1100	1150	Soil; alluvium, colluvium
Depth to Base	3.5	3.0	4.0	4.7	2.0	3.6	1.7	2.0	3.5	3.7	
Second Layer Velocity-V_2	4300	3100	7500	6500	5200	5200	2500	2500	5700	5000	Weathered bedrock
Depth to Base	15.9	13.5	20.4	19.0	11.7	17.0	14.3	15.8	26	25	
Third Layer Velocity-V_3	15,000	15,000	10,000	11,000	9,000	15,000	9,000	13,000	15,000	10,000	Sound rock

The colluvial and alluvial soils should be easy to excavate with conventional equipment. The weathered bedrock will be difficult to excavate requiring a D-9 caterpillar tractor with single shank ripper to break up the bedrock. Where velocities of 5000 ft/sec are found, some localized blasting could be required in the weathered bedrock zone. Blasting will be required in the sound unweathered rock.

4. The existing stream channel has cut very steep to nearly vertical side slopes that upon which it will be difficult to found a fill embankment.

5. Intermittant flow in the stream channel may result in fluctuating shallow groundwater level within the channel. No drilling was performed to verify the presence of subsurface water. No spring or seeps were identified in the alignment area.

6. The trend of the joint systems affecting the bedrock is in a downstream direction. It should be expected that the foundation and the abutment may experience some leakage through the bedrock discontinuities if water is retained by the structure for extended periods.

7. The structure should be founded in sound bedrock which will require the removal of alluvial soil, colluvium and weathered bedrock in the entire foundation area. The foundation area must be overexcavated to provide uniform, even surfaces to construct the structure.

8. Scaling of the steep abutment slope must be performed to remove boulders and loose rock from the face to provide for a good bond between the abutment slopes and the structure.

9. The presence of highly fractured bedrock in the foundation and abutment area will require cleaning and scaling of joints and fractures that may be exposed in the excavation to minimize underflow.

10. The conclusions and professional opinions presented in this letter report were developed by Geological Consultants for D.E. Clark in accordance with generally accepted geological principles and practices. The data, conclusions and recommendations should be considered to relate only to the specific project and location. No direct subsurface explorations, such as drilling and sampling, were made to assess and confirm the geology at proposed final grades. Therefore, an evaluation of the foundation and abutment areas should be made periodically during construction by a geologist to verify the geology exposed in the excavation. If there are any unanticipated geologic conditions during construction, the condition should be evaluated immediately to see if design changes are warranted.

If any changes are made in the project as outlined in this report, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the conclusions and recommendations of this report are modified or approved in writing by Geological Consultants.

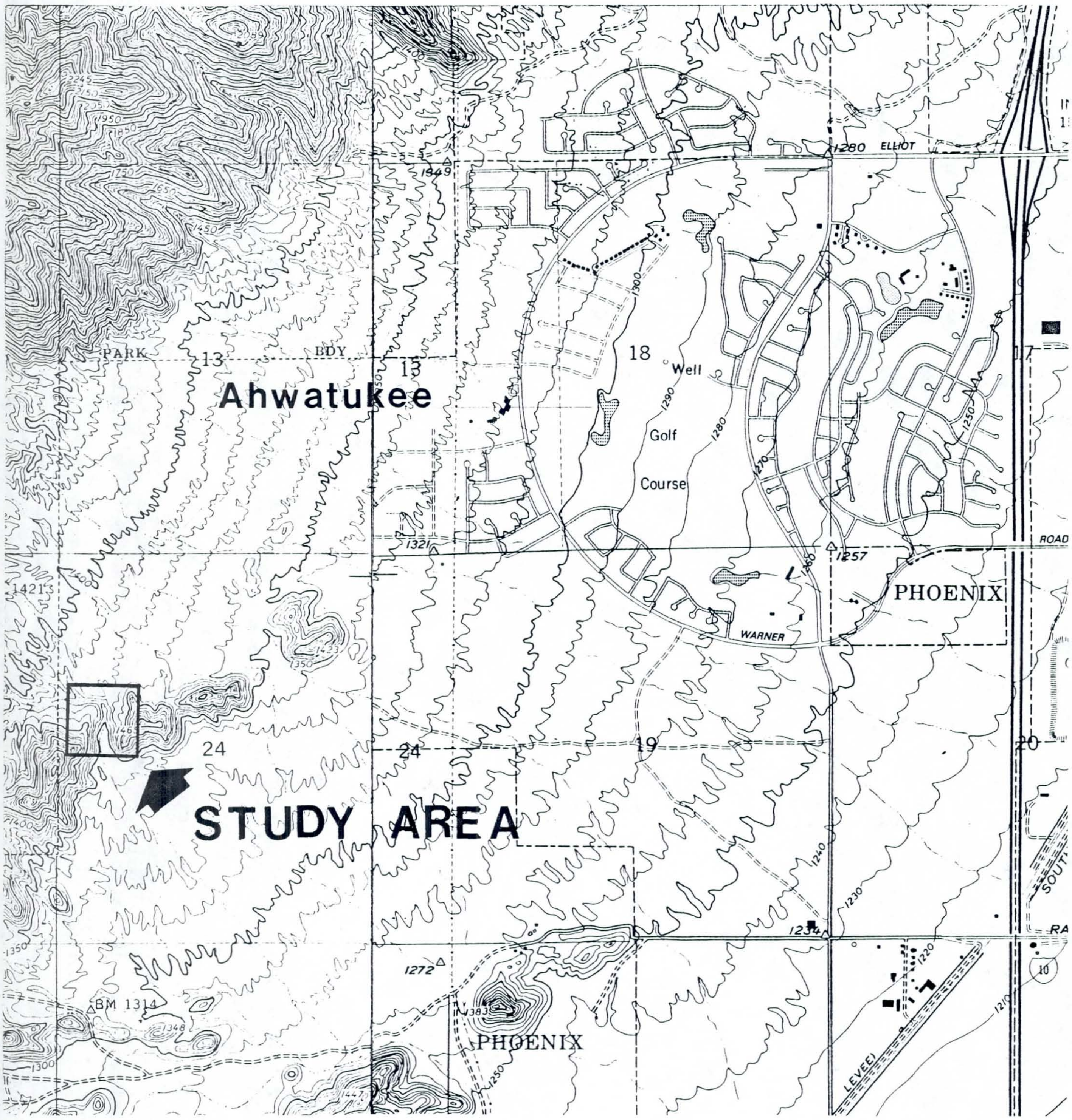
Thank you for this opportunity to be of service to you. If you have any questions regarding the content of this report, please contact me.

Very truly yours,

REGISTERED GEOLOGIST
CERTIFICATE NO.
10338
KENNETH M.
EUGE
Date Signed 12-17-85
ARIZONA, U.S.A.

Kenneth M. Euge, R.G.
Consulting Geologist

KME/jmw



Geological Consultants
2822 West Northern Avenue, Suite B
Phoenix, Arizona 85051

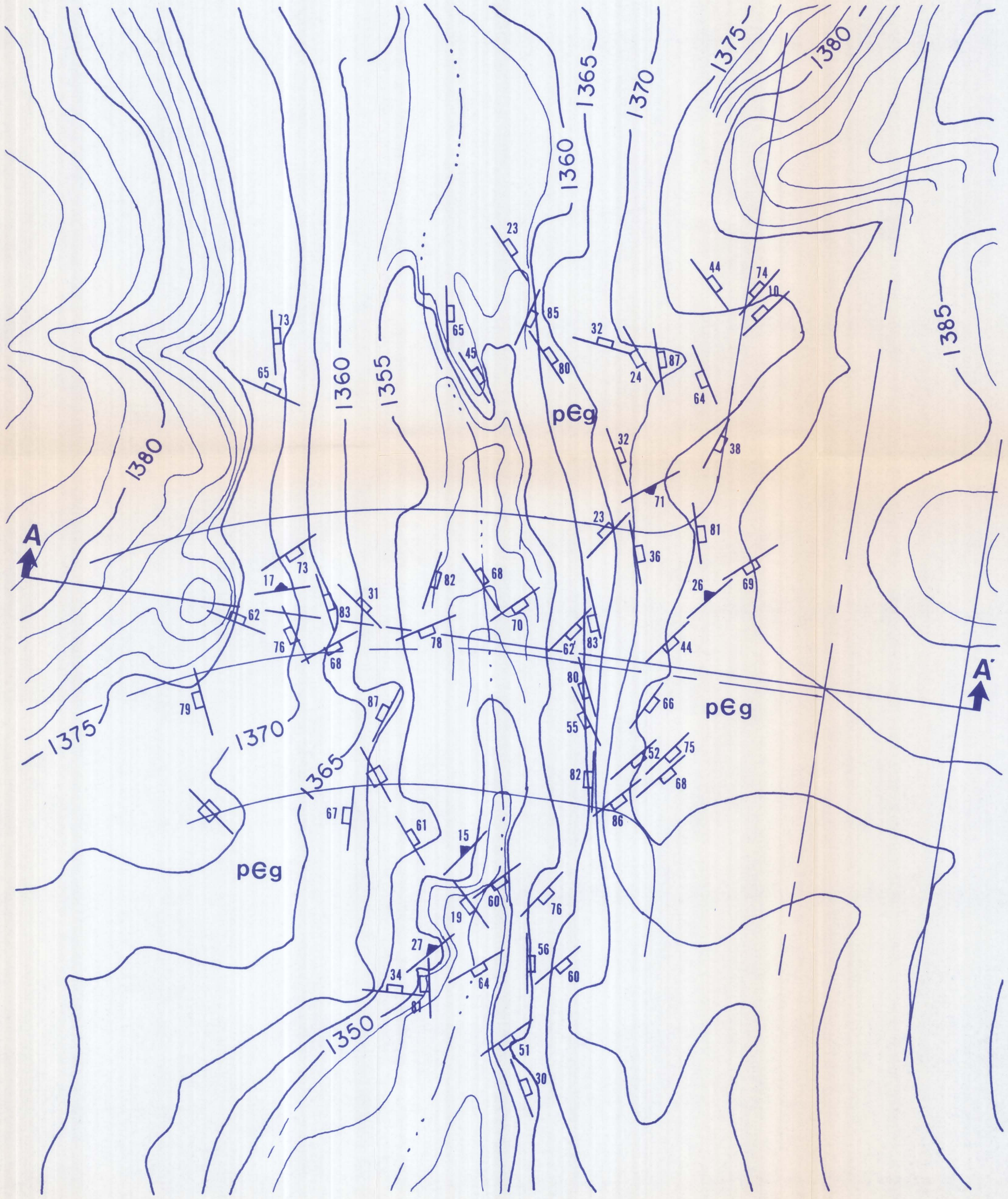
SCALE: 1" = 2000'	APPROVED BY: K. Z.	DRAWN BY K. Z.
DATE: 12-17-85		REVISED

Geological Reconnaissance and Seismic Survey
Proposed Floodwater Retarding Dike Ahwatukee
Maricopa County, Arizona

85-129

Location Map

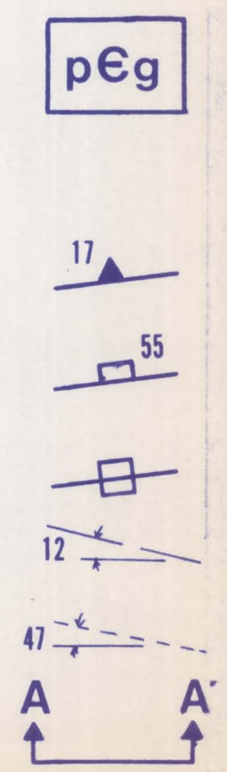
DRAWING NUMBER 1



Scale: 1 inch = 20 feet

Geologic Map and Profile

EXPLANATION:



South Mountain Granodiorite; strongly foliated, jointed, coarse grained, light gray (fresh surface) to red brown (weathered surface); locally covered with colluvium and stream channel alluvium.

Strike and dip of foliation.

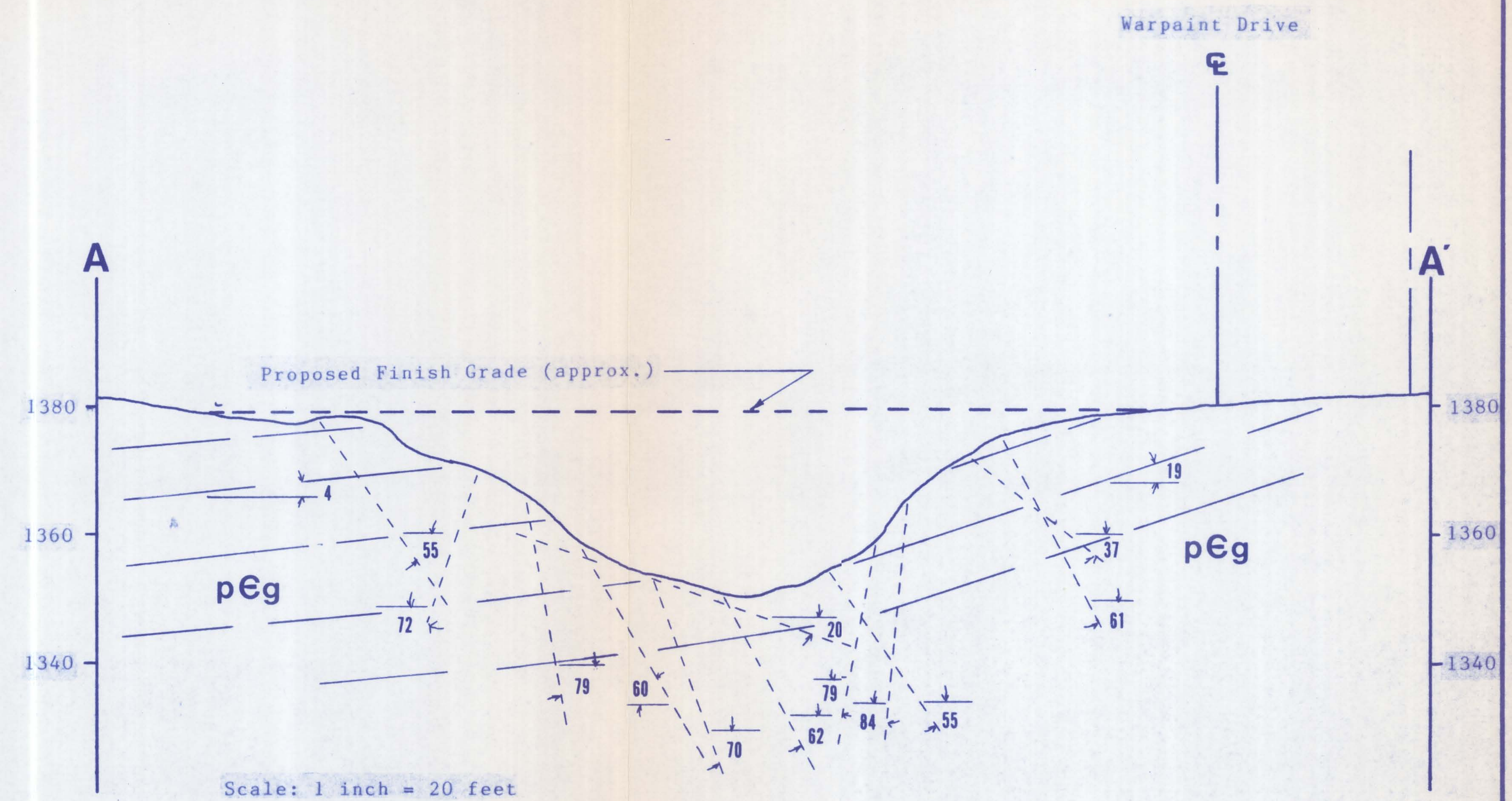
Strike and dip of joints.

Strike of vertical joints.

Foliation plane in profile with apparent dip on plane.

Joint plane in profile with apparent dip on plane.

Geologic profile.



Scale: 1 inch = 20 feet

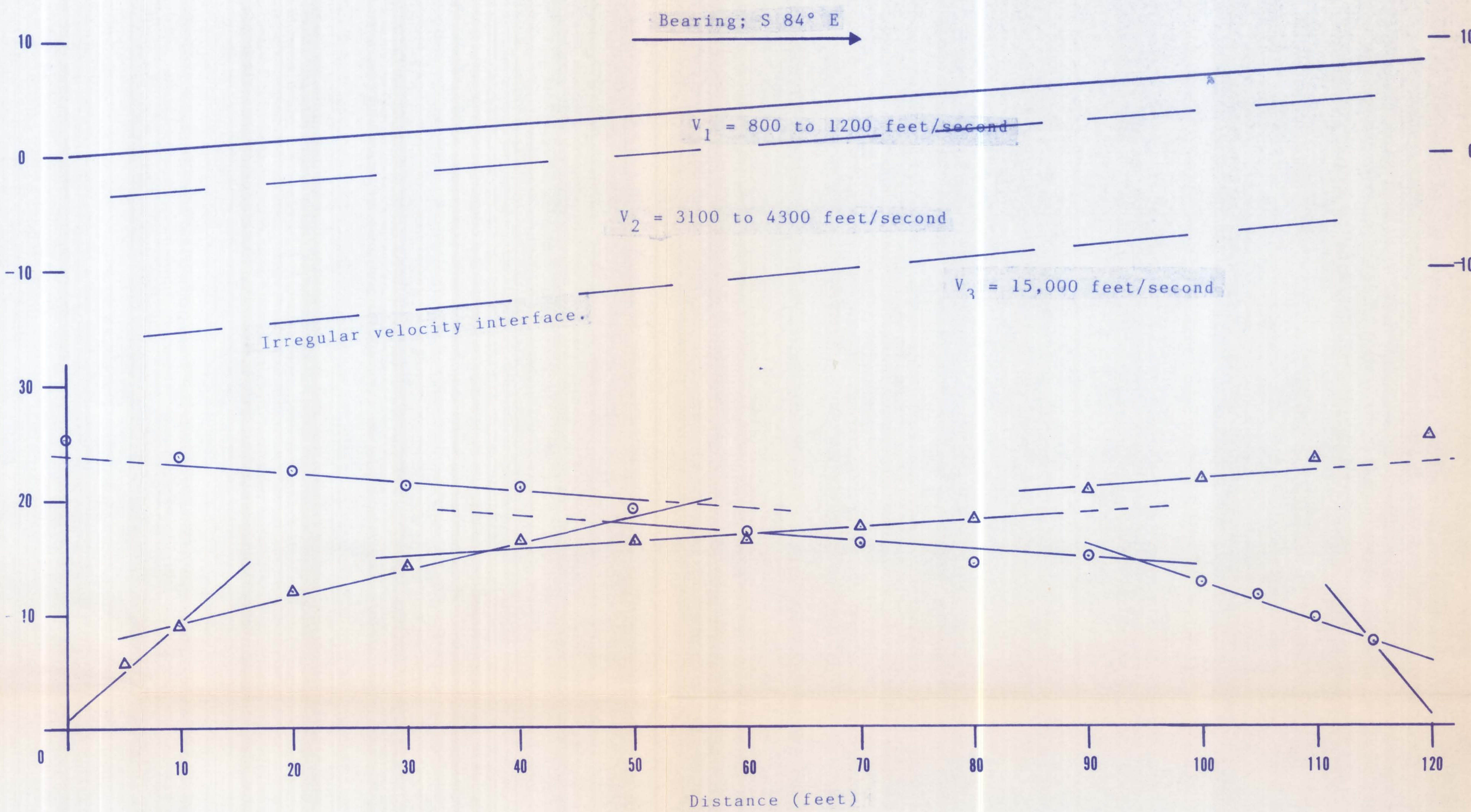
Geological Consultants
2822 West Northern Avenue, Suite B
Phoenix, Arizona 85051

SCALE:	APPROVED BY:	DRAWN BY K. Z.
DATE: 12-17-85		REVISED

Geological Reconnaissance and Seismic Survey
Proposed Floodwater Retarding Dike-Ahwatukee
Maricopa County, Arizona

Geologic Map and Profile — DRAWING NUMBER 2

85-129



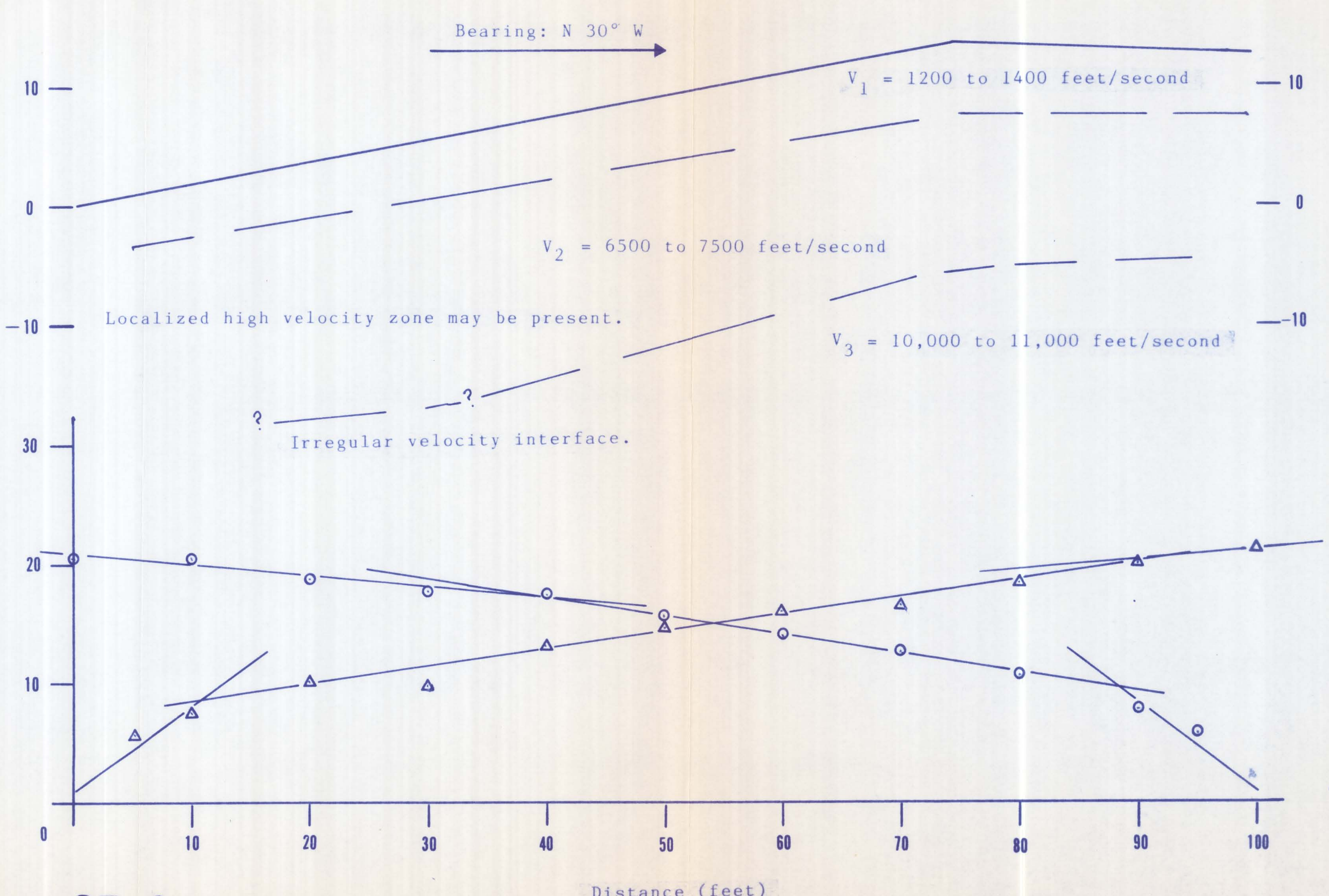
ST-1

EXPLANATION:

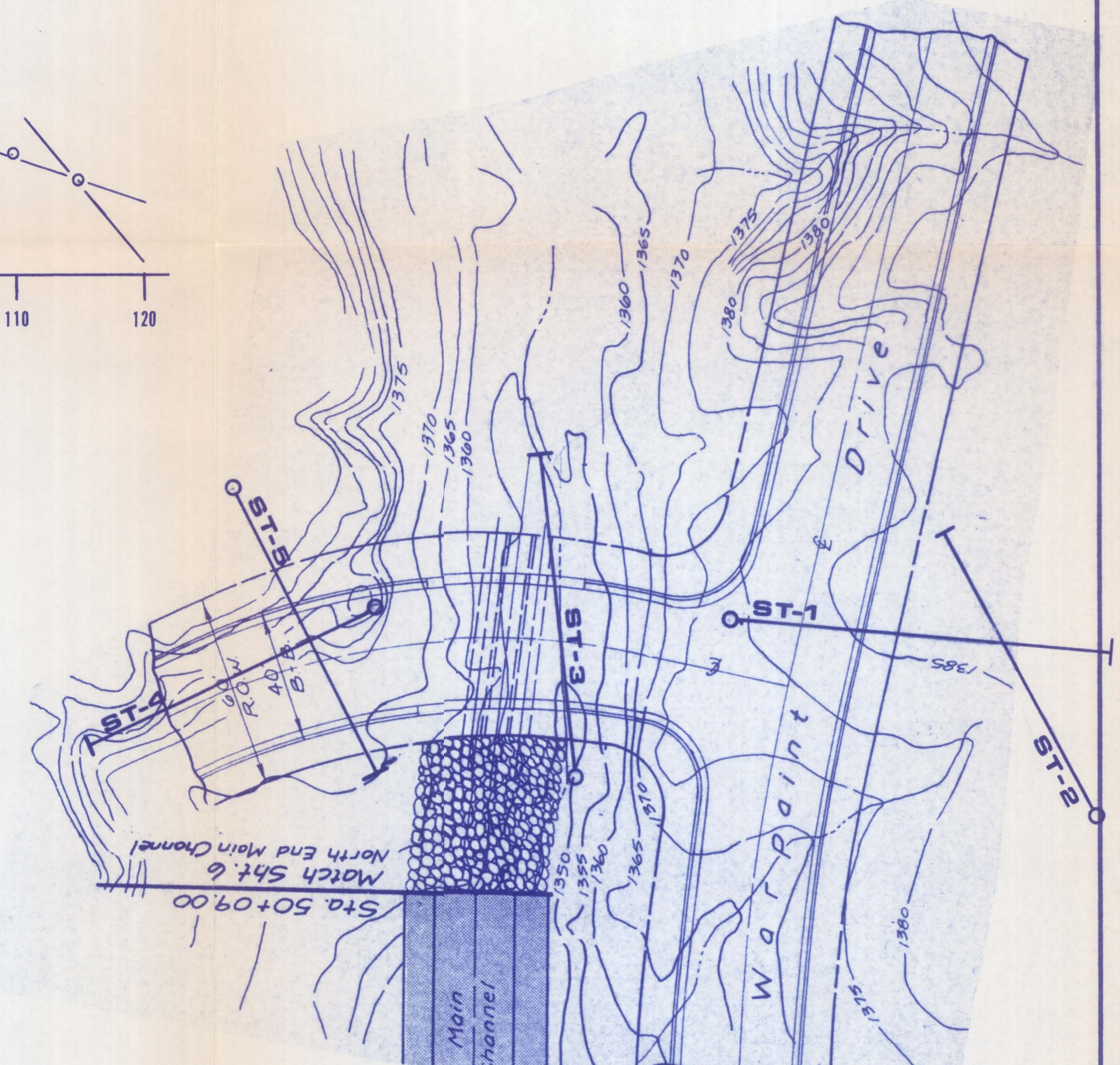
△ Time-distance measurments - forward traverse

○ Time-distance measurments - reverse traverse

ST-4 Seismic refraction traverse location.



ST-2



Scale: 1 inch = 40 feet

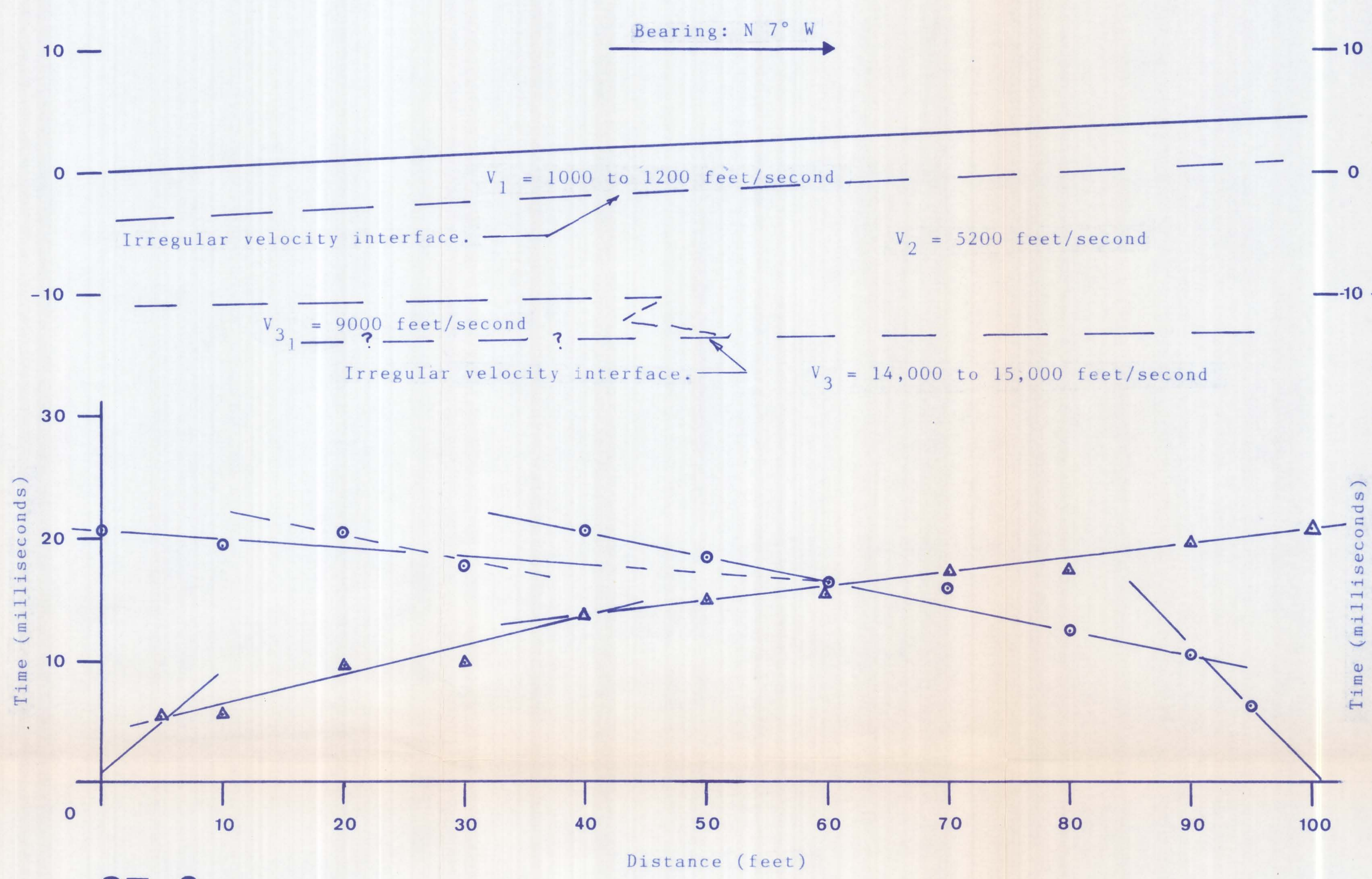
Location Map

Geological Consultants
2822 West Northern Avenue, Suite B
Phoenix, Arizona 85051

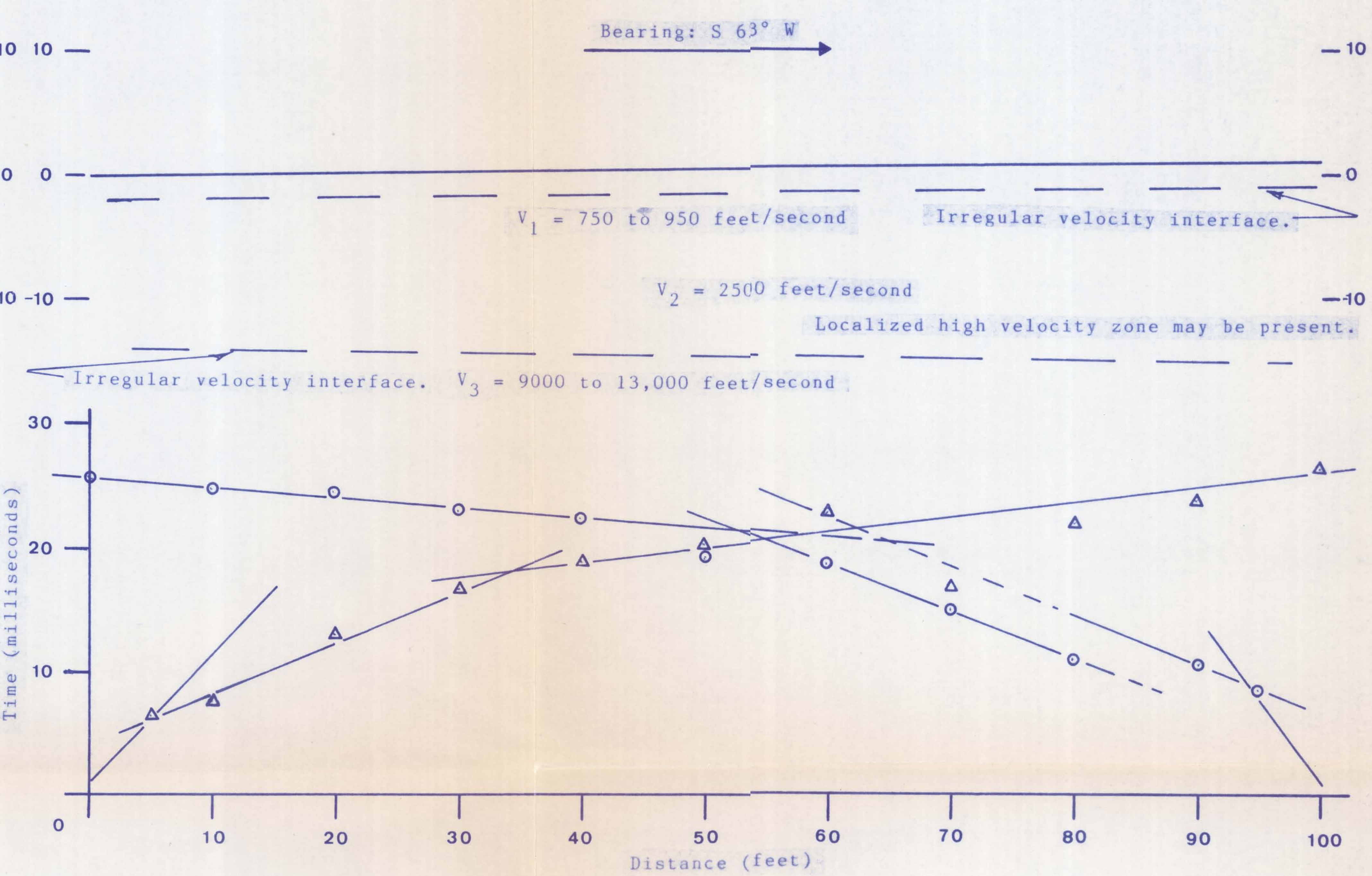
SCALE:	APPROVED BY:	DRAWN BY K. Zuge
DATE: 12-17-85		REVISED
	Geological Reconnaissance and Seismic Survey Proposed Floodwater Retarding Dike-Ahwatukee Maricopa County, Arizona	
	Seismic Refraction Surveys and Profiles	DRAWING NUMBER 3

85-129

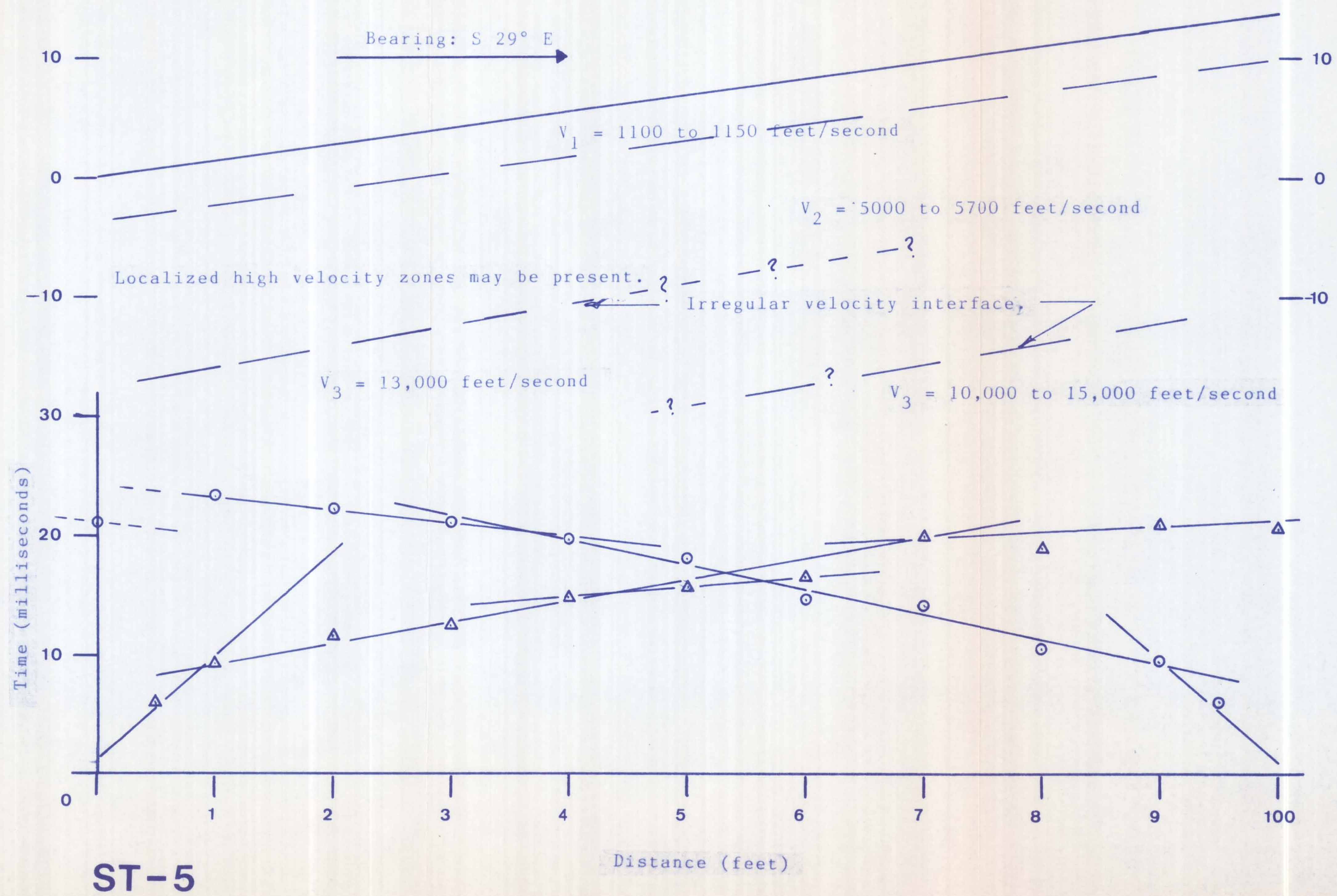
18 X 24 PRINTED ON NO. 1000H CLEARPRINT



ST-3



ST-4



ST-5

Refer to Drawing No. 3 for Explanation and Location Map.

Geological Consultants
2822 West Northern Avenue, Suite B
Phoenix, Arizona 85051

SCALE:	APPROVED BY:	DRAWN BY K. Z.
DATE: 12-17-85		REVISED

Geological Reconnaissance and Seismic Survey
Proposed Floodwater Retarding Dike-Ahwatukee
Maricopa County, Arizona

Seismic Refraction Surveys and Profiles — DRAWING NUMBER 4

85-129

18 X 24 PRINTED ON NO. 1000H CLEARPRINT

APPENDIX B

COMPUTATIONS

APPENDIX B

COMPUTATIONS

The computations used the following references:

U. S. Bureau of Reclamation, Design of Small Dams, 1977, pp. 329-339.

G. A. Leonards, Ed., Foundation Engineering, McGraw Hill, 1962, p. 626.

U. S. Navy, NAVFAC DM-7.2, Foundations and Earth Structures 1982, p. 7.2-63.

In each case, a one-foot section through the dike was analyzed. The forces considered were horizontal water pressure on the upstream face, vertical water pressure upward from the foundation, lateral earthquake loads, and weight of the structure. We considered the forces' effects on overturning, sliding, and overstressing within the structure and foundation. Earthquake and water pressures were not considered to act at the same time, so, by inspection, considering a Ø.Ø5g lateral earthquake force, water pressures governed the design. The computations indicate that any of the four alternates would be stable. The computations and observations appear on the following pages of computation sheets:

Page 1, OVERTURNING - Moments around the downstream toe of Alternate No. 1.

Page 2, OVERTURNING - Moments around the downstream edge, six feet below the top of the dike, and OVERSTRESSING - Foundation rock or dike of Alternate No. 1.

Page 3, SLIDING - Dike on base and dike+base on rock of Alternate 1.

Page 4, COMPARISON of the four alternate dikes, and the conclusion by inspection that Alternate No's 2 through 4 are at least as stable as Alternate No. 1.

COMPUTATION SHEET

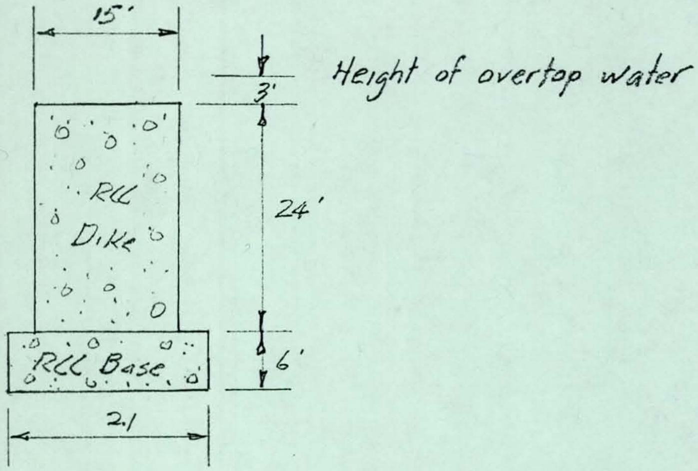
Subject *Stability Computations, Alternate No. 1* Sheet *1* of *4*

Job No. *15A* Client *Presley Development Co.*

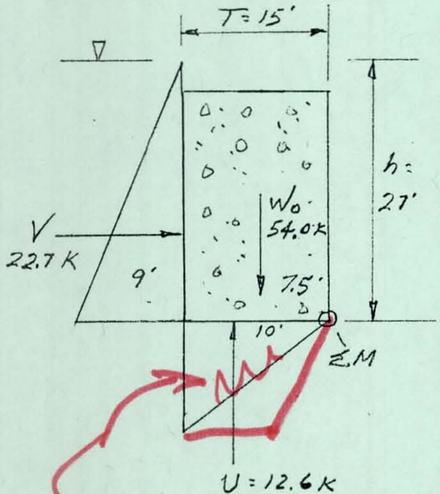
Computed By *D. Clark* Date *1/17/86*

Checked By *[signature]* Date *1-22-86*

Use Design of Small Dams, BuRec, 1977, pp. 329 - 339



OVERTURNING - Moments Around D/S Toe of the Dike (p. 337)



$w = 0.0624 \ K/ft^3$

$T = 15 \ ft$

$W_c = 0.150 \ K/ft^3$

$h = 27 \ ft$

$W_0 = 15(24)(0.150) = 54.0 K$

$V = \frac{1}{2}(0.0624)(27)^2 = 22.7 K$

$U = \frac{1}{2}(0.0624)(27)(15) = 12.6 K$

Triangular Distr. Not valid if only part of the Footing is bearing! Show footing pressure Distribution

↺ $(54.0)(7.5) = 405 \ K\text{-}ft$

↻ $\overset{204}{(22.7)(9)} + \overset{126}{(12.6)(10)} = 330 \ K\text{-}ft$

Safety Factor: $\frac{405}{330} = 1.2$ OK

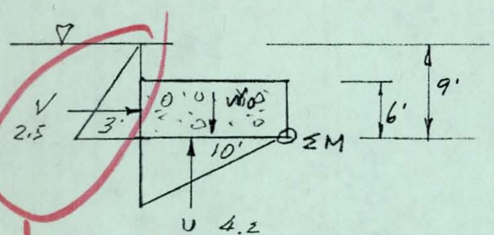
See "Design of Small Dams" Fig 223-D

D.E. CLARK

COMPUTATION SHEET

Subject *Stability Computations*
Alternate No. 1 Sheet *2* of *4*
Job No. *15A* Client *Presley Development Co.*
Computed By *D. Clark* Date *1/17/86*
Checked By [illegible] Date *1-22-86*

OVERTURNING – Moments Around D/S Edge, Six Feet Below the Top of the Dike



$h = 9$ ft

$W_o = (15)(6)(0.150) = 13.5$ K

$V = \frac{1}{2}(0.0624)(9)^2 = 2.5$K

$U = \frac{1}{2}(0.0624)(9)(15) = 4.2$K

OK.

~~If there is hydro. pressure here the head should be H = 27 + 9 = [illegible]~~

$(13.5)(7.5) = 101$ K-ft

$(2.5)(3) + (4.2)(10) = 50$ K-ft

Safety Factor $= \frac{101}{50} = 2.0$ OK

OVERSTRESSING – Foundation Rock or Dike

?

OK by inspection. No part is in tension from external forces.

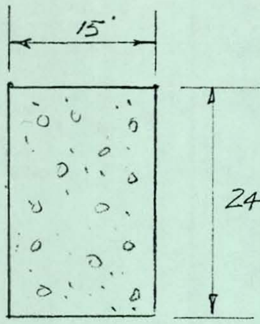
D.E. CLARK

DE-1 (12/85)

COMPUTATION SHEET

Subject: Stability Computations
Alternate No. 1 — Sheet 3 of 4
Job No. 15A — Client: Presley Development Co.
Computed By: D. Clark — Date: 1/17/86
Checked By: G. M. Alex — Date: 1-22-86

SLIDING - Dike on Base



Q = Shear Friction Factor = $\dfrac{CA + (\Sigma W - U)\tan\phi}{\Sigma V}$ (p. 338

1/2

C = twice the unconfined compressive strength. For weathered granite, the unc. comp. str. is 10,000 lb/in² (G. A. Leonards, Ed., Foundation Engineering, McGraw-Hill 1962, p. 626). That is much higher than will be movilized. Use 2 K/sq ft

$A = (15)(1) = 15$ sq ft

$\Sigma W = W_0 = 54.0$ K (sheet 1)

$U = 12.6$ K (sheet 1)

$\tan\phi = 0.70$ (Foundations and Earth Structures, NAVFAC DM-7.2, May 1982).

$\Sigma V = V = 22.7$ K (Sheet 1)

This figure should be larger

$$Q = \frac{(2)(15) + (54.0 - 12.6)(0.70)}{22.7} = \frac{30.0 + 29.0}{22.7} = 2.6 \text{ OK}$$

SLIDING - Dike + Base on Rock

OK by inspection

D.E. CLARK

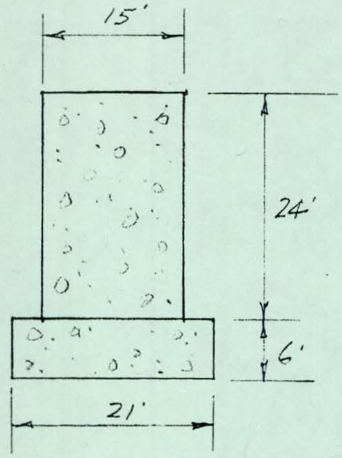
DE-1 (12/85)

COMPUTATION SHEET

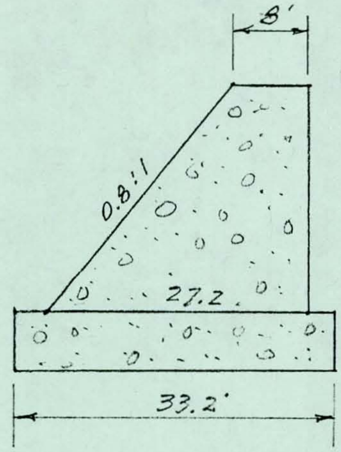
Subject: Stability Computations Alternate No's 2-4
Sheet 4 of 4
Job No. 15A Client Presley Development Co.
Computed By D. Clark Date 1/17/86
Checked By [signature] Date 1-22-86

Upstream side on the right

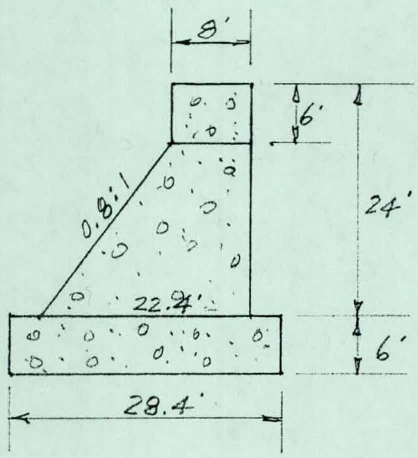
ALTERNATE NO. 1



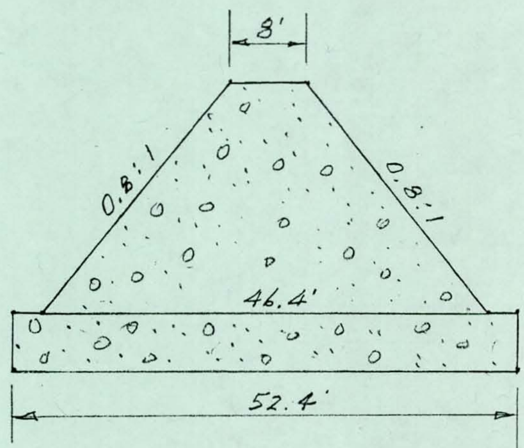
ALTERNATE NO. 2



ALTERNATE NO. 3



ALTERNATE NO. 4



By inspection, Alternate No's 2-4 are at least as stable as Alternate No. 1

D.E. CLARK

APPENDIX C

SUGGESTED SPECIFICATIONS

January 3Ø, 1986

APPENDIX C

SUGGESTED SPECIFICATIONS
PROPOSED FLOODWATER RETARDING DIKE
UNIT IV OF AHWATUKEE
MARICOPA COUNTY, ARIZONA
FOR PRESLEY DEVELOPMENT COMPANY

These specifications consist of three parts with the following headings and subheadings:

GENERAL

WORK SPECIFIED

These specifications cover labor, materials, and equipment to design a concrete mix, strip, dispose of strippings, construct a base and dike of roller compacted concrete (RCC), and perform acceptance testing.

PARTIES TO THE SPECIFICATIONS

The owner, Presley Development Company ... (The Owner)

A contractor yet to be selected ... (The Contractor)

The engineer, Brooks, Hersey & Associates ... (The Engineer)

The geotechnical engineer, D. E. Clark ... (The Geotechnical Engineer)

RELATED DOCUMENTS

Uniform Standard Specifications for Public Works Construction, sponsored and distributed by the Maricopa Association of governments, Arizona, 1979 (The MAG Specifications).

ASTM C 33-84, Standard Specification for Concrete Aggregates.

ASTM C 39-83b, Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.

ASTM C 42-84a, Standard Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete.

ASTM C 150-84, Standard Specification for Portland Cement.

ASTM C 192-81, Standard Method of Making and Curing Concrete Test Specimens in the Laboratory.

ASTM C 494-82, Standard Specification for Chemical Admixtures for Concrete.

ASTM C 618-84, Standard Specification for Fly Ash and Raw or Calcined Natural Pozzolan for use as a Mineral Admixture in Portland Cement Concrete.

ASTM C 684-81, Standard Method of Making, Accelerated Curing, and Testing of Concrete Compression Test Specimens.

ASTM C 918-80, Standard Method for Developing Early Age Compression Test Values and Projecting Later Age Strengths.

ASTM D 1557-78, Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-lb Rammer and 18-in. Drop.

ASTM D 2922-81, Standard Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth).

DESIGN SECTIONS

These suggested specifications are appended to a report of the same date prepared by The Geotechnical Engineer. Plate 2 of the report shows the axis of the dike relative to ground contours, Plate 3 shows a longitudinal section along the axis of the dike, and Plates 4 through 7 show alternate proposed cross sections.

PRODUCTS

MATERIALS

The Contractor shall supply the following quality of materials, subject to acceptance by The Geotechnical Engineer:

Cement, Class F flyash, water, and aggregates that comply with Section 725 of the MAG Specifications except in the matters of required strength, proportions of flyash to cement, slump, and field tests.

All aggregate shall pass a 1.5-inch sieve.

CONSTRUCTION EQUIPMENT

The Contractor shall use or supply the following types of equipment in good working order, subject to acceptance by The Geotechnical Engineer:

A stationary mixer that complies with Section 725.8.1 of the MAG Specifications.

Dump trucks that comply with Section 725.9 of the MAG Specifications.

A loader of sufficient capacity to deliver the RCC mix to the work area.

A D-7 Caterpillar tractor or equivalent to level and initially compact the RCC mix.

A laser device if necessary to control lift thicknesses.

A compactor capable of compacting each lift of the RCC mix to the degree hereinafter specified.

A Troxler 3411B nuclear moisture-density meter or equivalent.

Thermometers to measure air and concrete temperatures.

Other equipment as needed to strip the site, clean the rock surface, dispose of strippings, and perform any other functions found to be necessary.

EXECUTION

MIX DESIGN

The Contractor shall be responsible for design of the RCC mix, subject to approval of The Geotechnical Engineer.

Trial mixes shall be compacted at various moisture contents in a steel mold six inches in diameter and 10 to 12 inches in height, using Modified Proctor equipment and Modified Proctor compactive energy (ASTM D 1557).

The cementitious material shall consist of Class 2 cement and Class F flyash (ASTM C 150 and C 618), with the initial trial batch having 60% cement and 40% flyash.

The tentatively selected mix shall be workable, shall be designed so the mortar constituent fills or nearly fills voids in the coarse aggregate, and shall have a three-day compressive strength of approximately 700 psi at 95% compaction (ASTM C 39).

The final selection shall depend on preparing a fresh sample of the tentatively selected mix at 95% compaction, subjecting it to accelerated curing, and testing it to verify a 90-day compressive strength of at least 1,500 psi (ASTM C 39, C 684, and C 918).

SITE PREPARATION

The area to be covered by the dike shall be stripped of any vegetation, and the vegetation shall be legally disposed-of. Then the surface mantle of soil and boulders shall be removed down to bedrock sufficiently unweathered to support the dike. It is estimated that the depth of stripping will be roughly a maximum of six feet and an average of four feet; it will be defined in the field by the Geotechnical Engineer.

Non-bouldery strippings shall be stockpiled upstream and downstream from the dike in amounts sufficient to backfill against the base of the dike.

The remainder of the strippings shall be placed along the banks of the wash, just upstream from the dike, at locations selected by The Geotechnical Engineer.

Depending on the roughness of the exposed surface of bedrock, the surface shall be cleaned with a brush, an air jet, or a water jet.

Just before each area is covered with RCC, it shall be moistened and painted with a fairly thick grout of neat cement and water, extending into any cracks in the rock.

BASE CONSTRUCTION

The base construction has two purposes: to provide a level base for construction of the dike and outlet conduit, and to serve as a test section to develop effective construction procedures and to evaluate strengths being attained in the field.

Just before placing each lift of RCC, The Contractor shall verify cleanliness of the bedrock surface, completeness of coverage with the painted-on grout, and cleanliness of the preceding lift of RCC. If the preceding lift is not essentially free of dirt and other contaminants, it shall be sand blasted or otherwise cleaned to the satisfaction of The Geotechnical Engineer.

The RCC shall be mixed with adequate thoroughness then delivered to the site in end-dump trucks that comply with Section 725.9 of the MAG Specifications. It shall be handled in a manner that minimizes segregation, contamination, and drying.

When delivered the RCC shall have a temperature between 40F and 90F. No RCC shall be delivered or worked when weather conditions will cause the mix to dry or reach a temperature outside the 40-90F range before compaction is complete, or when rain is expected. If the temperature range is exceeded, or if a lift is rained on before compaction is complete, that lift of RCC shall be removed and discarded.

Each lift of RCC shall be placed and spread in a relatively uniform thickness that can be compacted to at least 95% compaction with the provided equipment; a compacted lift thickness of eight inches is contemplated. Compaction shall be completed within one hour after water has been mixed in; 1 1/2 hours if the air temperature is below 60F. Each lift shall be level within 0.2 foot, and shall be completed before starting the next lift. Each lift shall be kept continuously moist and shall be protected against erosion and other damage until covered.

If more than five hours elapses between lifts, or if the surface has been damaged or has required cleaning, the surface shall be painted with grout just before placing the overlying lift.

Compaction equipment and other construction equipment shall not be operated within one foot of the upstream and downstream edges of the base. Lateral support shall be provided by backfilling against the vertical faces of the base as each lift is placed.

The base shall be cored and tested as described below under "Quality Assurance."

DIKE CONSTRUCTION

With what is learned from constructing and testing the base, procedures shall be adjusted as necessary and used to similarly construct the dike.

After the dike is complete, it shall be continuously water cured for a period of at least seven days.

QUALITY ASSURANCE

The Contractor shall provide a technician to test and record () moisture content and temperature of each incoming load of RCC, (2) density of enough places on each lift to indicate whether at least 95% compaction has been attained, and (3) ambient air temperatures at least once each morning and each afternoon. Each day the Contractor shall also observe and record weather conditions, particularly rainfall. The records shall be available at all times for inspection of The Owner, The Engineer, and The Geotechnical Engineer, and copies of all records shall be periodically provided to The Geotechnical Engineer.

Once when several feet of base has been built, and again when the base is complete, The Contractor shall obtain three cores six inches in diameter and approximately 10 to 12 inches in height (ASTM C 42), shall cure them until approximately three days after they were compacted, and shall test them for compressive strength (ASTM C 39). The holes shall be backfilled with poured concrete that will attain a strength of 2,000 psi at seven days. The Geotechnical Engineer shall be notified and given the opportunity to observe the coring and testing. As soon as the test results are available, they shall be provided to The Geotechnical Engineer; the written results shall follow within a week.

During construction of the dike, at three times selected by The Geotechnical Engineer, The Contractor shall similarly obtain three cores, cure and test them, and present the results.

DEC/dc

SUBMITTAL #2
March 5, 1986

D.E. CLARK
Geotechnical Engineer

From the Desk of:
DON CLARK

3/3/86

Nick:

I'll call for an appointment to come in and see you later this week. I would appreciate your critical review of what I have done, and would be interested to learn the District's normal computation procedures.

Don Clark

FLOOD CONTROL DISTRICT
RECEIVED

MAR 05 '86

	CH ENG		HYDRO
	ASST		LMgt
	ADMIN		SUSP
	C & O		FILE
	ENGR		DESTROY
	FINANCE		
REMARKS			

D.E. CLARK
Geotechnical Engineer

From the Desk of:
DON CLARK

3/3/86

Nick:

I'll call for an appointment to come in and see you later this week. I would appreciate your critical review of what I have done, and would be interested to learn the District's normal computation procedures.

Don Clark

SUBMITTAL #2
March 5, 1986

DRAFT

March 3, 1986
Our Job No. 15A

Nicholas P. Karan, P.E.
Chief, Engineering Division
Maricopa County Flood Control District
3335 West Durango Street
Phoenix, Arizona 85009

Dear Mr. Karan:

This letter and its attachments follow up on our phone conversations about the stability computations I did for a proposed floodwater retarding dike in Ahwatukee. The computations are presented as Appendix B of our January 30 report to Brooks, Hersey & Associates, for Presley Development Company.

The letter summarizes site conditions, summarizes the proposed construction, describes the original analyses, describes supplementary analyses, discusses the results, and presents my conclusion from the supplementary work. Attached to the letter are a list of cited references and a copy of supplementary computations (6 p.).

SUMMARY OF SITE CONDITIONS

The site is a steep-sided notch cut through a ridge of tightly jointed granitic bedrock, exposed in the sides of the notch, and covered elsewhere by some four to six feet of colluvium and alluvium. The latter are to be stripped. The upper, weathered granite has been assigned an unconfined compressive strength of 10,000 psi, the low end of the range for granite (Sowers, 1962).

SUMMARY OF PROPOSED CONSTRUCTION

To create a level working pad, the channel bottom is to be filled with a base of roller compacted concrete (RCC) 21 feet wide in the upstream/downstream direction, and about 45 feet long between the abutments. The thickness of the base is to be about six feet at the stream centerline, tapering to zero at the abutments. The RCC is to be designed to have a 90-day compressive strength of at least 1,500 psi.

 DRAFT

The dike is to be built of RCC similar to that in the base. The dike is to be 15 feet wide, about 45 feet long at the base, and about 108 feet long at the crest. Through the dike there is to be an ungated outlet roughly six feet wide and 15 feet high.

The channel carries water only immediately after rainfall; it is dry nearly all the time. The dike and outlet are sized so a 100-year, one-hour storm will just fill the reservoir; the dike would fill and drain within two hours.

ORIGINAL ANALYSES

The original analyses made several conservative assumptions: a water depth three feet more than the full reservoir condition, full penetration of water under all of the dike, and no consideration of the fact that about 10 percent of the dike length at its base really has over it a large conduit rather than a wall holding back water. Design should be stable when analyzed on a per ft. of width basis

Original Analysis of Sliding Resistance

For computation of sliding resistance, the original analysis used a Bureau of Reclamation formula that utilizes the cohesion value of the concrete or rock, and utilizes the angle of internal friction of the rock/concrete contact (BuRec, 1977). I evaluated the cohesion as a function of the unconfined compressive strength, but made a mistake in the analysis. I should have used the concrete/concrete interface of the dike on the base, in which case the cohesion value would be 1/2 of 1,500 psi. The tangent of the angle of internal friction was assigned a value of 0.70 (NAVFAC, 1982).

With a cohesion value of 750 psi (108 ksf), there is still a very large factor of safety against sliding. Your point is well taken: adhesion might be a better measure of sliding resistance. The supplementary computations redo the analysis using adhesion rather than cohesion and friction.

DRAFT

Original Analysis of Overturning

For overturning, the original analysis used a static, free-body diagram, such as that shown for the concrete dam in Figure 21-95 of the Standard Handbook for Civil Engineers (Nelson, 1976). For the stabilized, steady-state seepage condition, Nelson also shows a triangular uplift diagram, with the pressure ranging from Ø at the downstream toe to between 1/2 and full hydrostatic pressure at the upstream toe. I used the full hydrostatic pressure for my analysis.

Using moments from the water pressure force on the upstream face, from the uplift force, and the dike weight, the result was a safety factor of 1.2 based on resisting moments divided by overturning moments. Considering the conservativeness of the assumptions I considered this adequate.

Since I was not concerned about a bearing capacity failure, I stated that stresses in the foundation were OK by inspection. From that I concluded that no part of the structure would be in tension. As you pointed out, the no-tension conclusion is incorrect; the resultant of the forces is outside the middle third of the base. The supplementary computations redo the analysis, but with less conservative assumptions.

SUPPLEMENTARY ANALYSES

The supplementary analyses reduce the water depth to just the full reservoir condition (24 feet), analyze safety factors with both no penetration of water under the dike (no uplift) and with full penetration of water under the dike, but still give no consideration to the fact that about 1Ø percent of the dike length at its base really has over it a large conduit rather than a wall holding back water. N.G.

The structure must be stable during overtopping. The effect of uplift must be included as in "Design of Small Dams"

[redacted]

DRAFT

Supplementary Analysis of Sliding Resistance

For computation of sliding resistance, the supplementary analysis uses adhesion rather than the cohesion and internal friction values at the concrete/concrete interface. The analysis is done on Page 6 of the attached computations. It uses an adhesion value of five percent of the concrete's unconfined compressive strength, a value which already includes a safety factor appropriate to drilled pier practice. The computed value is 1Ø.8 ksf, which is conservative compared to the 28.8 ksf value allowed by the New York City building code (D'Appolonia et al, 1975).

would be much less with overtopping

The computed safety factor is 9.Ø for the condition of no uplift, and 1.25 for the extreme redistribution of pressures computed by the Burec method (Pages 5 and 6 of the attached computations, and Page 337, Burec, 1977). For concrete dams on rock, the Burec considers 2.Ø to be the minimum safety factor against sliding in general, and 1.5 to be the minimum safety factor against sliding under extreme conditions (Burec, 1977, p. 338). Considering that the adhesion value I used already contains a safety factor that must be at least two, I believe the analysis indicates that the dam is safe against sliding.

Supplementary Analysis of Overturning

For overturning, the supplementary analysis is done with both the static, free-body procedures used in the original analysis, and with the Bureau of Reclamation's procedures (Burec, 1977, p. 337).

The forces and pressures on the dike free-body are computed and graphically shown on Page 1 of the attached computations. Pages 2 and 3 show computations of eccentricity of resultant forces, e, and safety factors with and without uplift. Unless there is an adequate safety factor against overturning anyway, most designers prefer to keep the resultant within the middle third on soil foundations, and within the middle half on rock foundations (Peck et al, 1974). Without uplift, e is Ø.16 feet outside the middle third; with uplift, e is 1.53 feet outside the middle third and Ø.28 feet outside the middle half. Under this condition, Tschebotarioff (1962) recommends, for retaining walls on soil, a safety factor of at least 2. Lambe and Whitman (1969) recommend at least 1.5. Bowles (1977) suggests safety factors of 1.5 for foundation soils in general, and 2.Ø if a foundation soil is clay.

 DRAFT

Using the dike free-body forces, Pages 2 and 3 of the computations calculate safety factors of 2.8 without uplift, and 1.6 with uplift. These satisfy the normal requirements for retaining walls, even on soil.

Using the Bureau of Reclamation procedures for concrete dams (Burec, 1977, p. 337), Pages 3 through 5 of the computations calculate and plot the pressures on the base. Relative to the unconfined compressive strength, the safety factor must be at least four (Copen, 1974).

Page 5 of the computations calculates safety factors of 29 without uplift and 6.8 with uplift. These satisfy the Burec overturning requirements for concrete dams (Burec, 1977, p. 338).

SUMMARY

The following table summarizes the computed safety factors calculated as described above:

	SAFETY FACTORS Without Uplift	With Uplift
Sliding	9.Ø	1.25*
Overturning, Free-Body Analysis	2.8	1.6
Overturning, Burec Analysis	29	6.8

would be less with overturning

* The adhesion value used in the analysis already contains a safety factor, probably at least two, so the real figure is probably at least 2.5.

These are not SF; per se, that is they are not the ratio of the stabilizing moment to the overturning moment. They are merely the ratio of ~~[illegible]~~ allowable soil pressure to actual ~~design~~ soil pressure. Structure is considered safe against overturning if ratio is larger than 1

DRAFT

DISCUSSION

The points of application of resultant forces at the base of the dike lie just outside the middle third of the base when uplift pressures are not included, and lie just outside the middle half when uplift forces are included.

Full uplift is unlikely to occur during the brief periods when the dike is to hold back water.

With or without uplift forces, whether the dike is analyzed as a retaining wall or a dam, the computed safety factors exceed what is required in normal practice.

CONCLUSION

In my opinion, the supplementary computations indicate that the dike will be stable. X Re-analyze including overtopping. Also uplift must be included

The list of references and the computation sheets are attached and complete this letter.

Yours very truly

D. E. CLARK
Geotechnical Engineer

Donald E. Clark

DEC/dc

Attachments

Copy to: Brooks, Hersey & Associates
Attention: William L. Clark

March 3, 1986

REFERENCES CITED
(In the Text and Computations)

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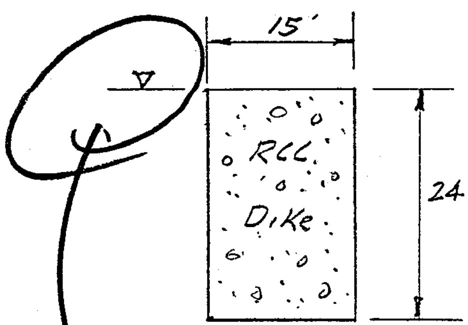
COMPUTATION SHEET

Subject *Supplementary Computations* Sheet *1* of *6*
Job No. *15A* Client *Presley Dev. Co.*
Computed By *D. Clark* Date *3/1/86*
Checked By Date

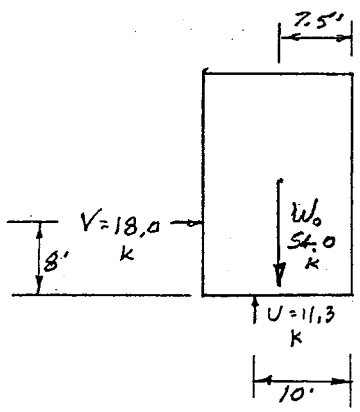
DIKE DIMENSIONS : 1" = 20'

Ref: BuRec
Design of Small Dam
1977
p. 337



Design Calcs of 1-17-86 Show 3' of overtopping! Why the change?

FREE-BODY DIAGRAM 1" = 100 k



$w = 0.0624\ k/ft^3$

$T = 15\ ft$

$w_c = 0.150\ k/ft^3$

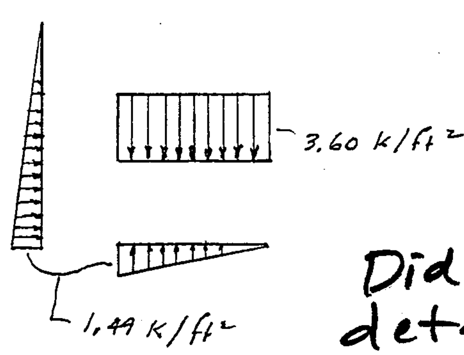
$h = 24\ ft.$

$W_o = 15\,(24)(0.150) = 54.0\ k$

$V = \frac{1}{2}(0.0624)(24)^2 = 18.0\ k$

$U = \frac{1}{2}(0.0624)(24)(15) = 11.3\ k$
(with Δ pressure distrib, full penc. of water)

PRESSURE DIAGRAMS 1" = 10 ksf



$wh = 0.0624\,(24) = 1.49\ k/ft^2$

$w_c h = 0.150\,(24) = 3.60\ k/ft^2$

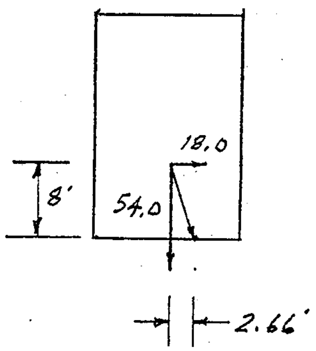
Did not check in detail but procedure seems OK but Need to add overtopping

D.E. CLARK

COMPUTATION SHEET

Subject *Supplementary Computations*
Sheet *2* of *6*
Job No. *15A* Client *Presley Dev. Co.*
Computed By *D. Clark* Date *3/1/86*
Checked By Date

OVERTURNING WITHOUT UPLIFT (FREE BODY)



$$e = \frac{\Sigma M \text{ (center of base)}}{\Sigma W}$$

$$= \frac{18\,(8)}{54.0} = 2.66 \text{ ft.}$$

$\frac{1}{6} T = 2.50$, so the resultant (w/o uplift) intersects the base 0.16 ft outside the middle third.

Per Tschebotarioff, 1962, if a retaining wall on soil has the resultant outside the middle third, the potential for overturning should be checked against the criterion that, taking moments around the toe, the safety factor should be at least 2. Per Lambe & Whitman, 1969, it should be at least 1.5. Bowles, 1977, suggests safety factors of 1.5 in general 2.0 if the foundation soil is clay.

$$SF = \frac{\Sigma M \text{ (toe) resisting}}{\Sigma M \text{ (toe) overturning}}$$

$$= \frac{54\,(7.5)}{18\,(8)} = 2.81$$

Revise for Overtopping!

D.E. CLARK

DE-1 (12/85)

COMPUTATION SHEET

Subject *Supplementary Computations* Sheet 3 of 6

Job No. 15A Client Presley Dev. Co.

Computed By D. Clark Date 3/1/86

Checked By [redacted] Date [redacted]

OVERTURNING WITH UPLIFT (FREE BODY)

If the triangular uplift pressure is added, the safety factor becomes:

$$e : \frac{18(8) + 11.3(2.5)}{54.0 - 11.3} = \frac{144 + 28.2}{42.7} = 4.03$$

(w) full uplift)

$$SF = \frac{405}{144 + 11.3(10)} = 1.57\ldots > 1.5,\ OK$$

1.53 ft outside mid 1/3

$\frac{1}{4} T = 3.75$

0.28 ft outside mid 1/2

FOUNDATION REACTION PRESSURES

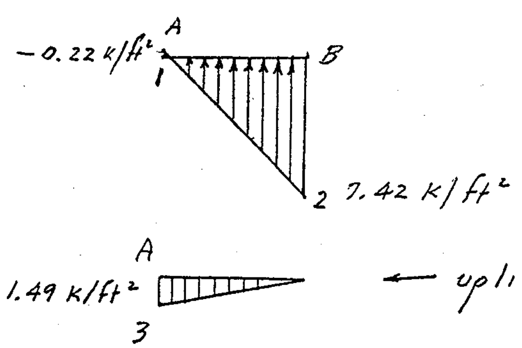
Without Uplift (BuRec, p. 337)

$$A1 = \frac{\Sigma W}{T}\left(1 - \frac{6e}{T}\right) \qquad \text{(Formula 5)}$$

$$= \frac{54.0}{15}\left(1 - \frac{6 \times 2.66}{15}\right) = 3.60\,(1 - 1.06) = -0.22\ K/ft^2$$

$$B2 = \frac{\Sigma W}{T}\left(1 + \frac{6e}{T}\right) \qquad \text{(Formula 6)}$$

$$= \frac{54.0}{15}\left(1 + \frac{6 \times 2.66}{15}\right) = 3.60\,(1 + 1.06) = 7.42\ K/ft^2$$



← uplift pressures

D.E. CLARK

Revise for Overtopping!

DE-1 (12/85)

COMPUTATION SHEET

Subject *Supplementary Computations*
Sheet *4* of *6*
Job No. *15A* Client *Presley Dev Co.*
Computed By *D. Clark* Date *3/1/86*
Checked By ____ Date ____

FOUNDATION REACTION PRESSURES (CONT)

Revised for Excessive Uplift

A3 > A1 so it is necessary to revise the foundation pressures.

$$e' = \frac{\Sigma M}{\Sigma W - A3 \cdot T} \qquad \text{(Formula 7)}$$

$$\frac{18(8) + 11.3(2.5)}{54.0 - 1.49(15)} = \frac{144.0 + 28.3}{54.0 - 22.3} = \frac{172.3}{31.7} = 5.43 \text{ ft.}$$

$$T_1 = 3\left(\frac{T}{2} - e'\right) \qquad \text{(Formula 8)}$$

$$= 3\left(\frac{15}{2} - 5.43\right) = 3(7.50 - 5.43) = 2.07 \text{ ft}$$

$$B5 = \frac{2(\Sigma W - A3 \cdot T)}{T1} + A3$$

$$= \frac{2(54 - \overset{22.4}{1.49 \times 15})}{2.07} + 1.49 = \frac{2(31.6)}{2.07} + 1.49$$

$$= 30.53 + 1.49 = 32.0 \text{ K/ft}^2$$

Revise for Overtopping

D.E. CLARK

DE-1 (12/85)

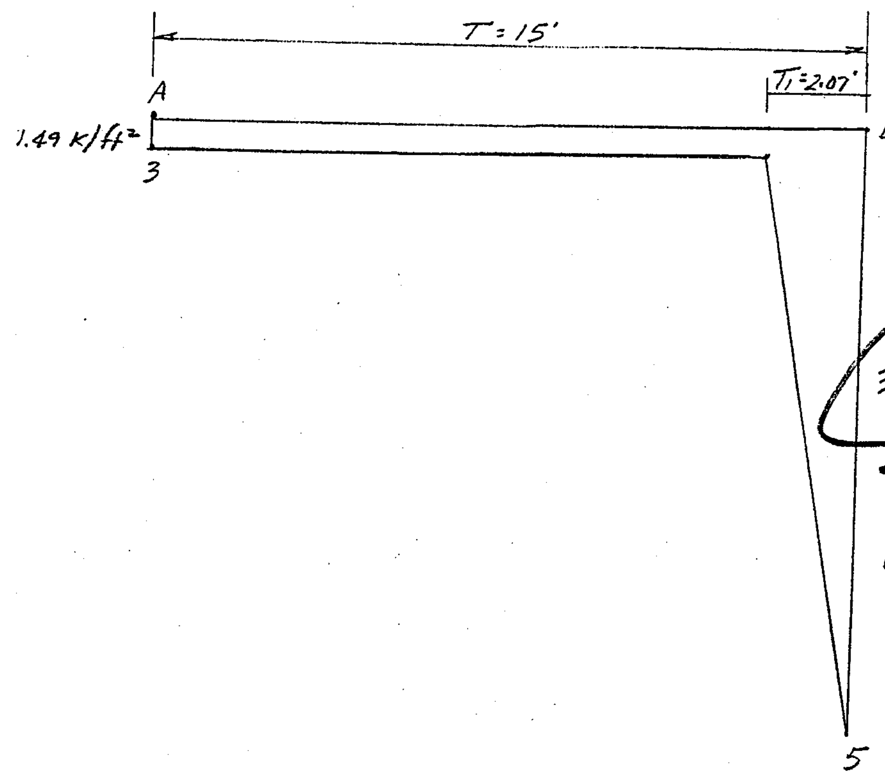
COMPUTATION SHEET

Subject: Supplementary Computations
Sheet 5 of 6
Job No. 15A Client Presley Dev. Co.
Computed By D. Clark Date 3/1/86
Checked By ______ Date ______

FOUNDATION REACTION PRESSURES (CONT.)

Revised for Excessive Uplift (cont.)

1" = 4'
1" = 10 k/ft²



This value will increase if you include the overtopping. It will probably exceed the Allowable on Rock!

COMPUTED SAFETY FACTORS

Per Lopen, 1974, "a safety factor of four or more, based on the ratio of concrete strength to computed stress, is usually required for all concrete dams. The concrete strength = 1500 psi, which equals $\frac{1500}{1000}(144) = 216\ k/ft^2$

The SF w/o uplift is $\frac{216}{7.42} = 29$ OK

on Rock?

The SF w/ uplift is $\frac{216}{32.0} = 6.8$ OK

D.E. CLARK

What is the allowable crushing pressure on the rock?

COMPUTATION SHEET

Subject: Supplementary Computations
Sheet 6 of 6
Job No. 15A Client: Presley Dev Co
Computed By: D. Clark Date: 3/1/86
Checked By: Date:

SLIDING RESISTANCE (ADHESION)

BuRec, 1975, recommends a safety factor of at least 2 for concrete dams on rock foundations, 1.5 for extreme conditions. D'Appolonia et al, 1975, present measured values of concrete/rock adhesion from large-scale tests by various investigators, and presumptive values presented in the literature. Measured values range from 20+ ksf in schist to 288 ksf in limestone. The New York City code allows 29K in sound, hard rock. Two co-investigators recommend a presumptive value of 5% of the concrete compressive strength (which I assume includes a safety factor appropriate for drilled pier foundations). The previous sheet of computations (5), shows this concrete's compressive strength to be 216 k/ft².

<u>Without Uplift</u>

$$SF = \frac{\text{Area of Base }(216)}{V} = \frac{(15)(1)(216)(0.05)}{18.0} = 9.00 > 2 \quad \text{OK}$$

<u>With Uplift</u>

Revise for Overtopping; include uplift

With the extreme redistribution of pressures shown on Sheet 5, assume only 2.07 ft of base is in contact

$$SF = \frac{(2.07)(1)(216)(0.05)}{18.0} = 1.25 \ldots < 1.5,$$

D.E. CLARK

but OK considering that the "0.05" already contains a safety factor.

DE-1 (12/85)

SUBMITTAL #3
3-17-86

FLOOD CONTROL DISTRICT
DRAFT RECEIVED
MAR 17 '86

CH ENG	HYDRO
ASST	LMgt
ADMIN	SUSP
C & O	FILE
ENGR	DESTROY
FINANCE	
REMARKS	

March 15, 1986
Our Job No. 15A

Nicholas P. Karan
Chief, Engineering Division
Maricopa County Flood Control District
3335 West Durango Street
Phoenix, Arizona 85009

Dear Mr. Karan:

This letter follows our March 7 meeting about stability of the proposed floodwater retarding dike in Ahwatukee. The location and dimensions of the dike are shown in our report of January 30, 1986, titled "Geotechnical Investigation, Proposed Floodwater Retarding Dike, Unit IV of Ahwatukee, Maricopa County, Arizona, for Presley Development Company."

The letter summarizes site conditions, summarizes the proposed construction, describes design assumptions, describes computations, discusses results, and presents my conclusions. Enclosed are copies of the computations (15 p.). Attached are a list of references (1 p.) and summaries of the computation results (2 p.).

SUMMARY OF SITE CONDITIONS

The site is a steep-sided notch cut through a ridge of tightly jointed granitic bedrock, exposed in the sides of the notch, and covered elsewhere by some four to six feet of colluvium and alluvium. The alluvium and colluvium are to be stripped.

SUMMARY OF PROPOSED CONSTRUCTION

The dike is to be built of roller compacted concrete (RCC), and is planned to be 15 feet wide, about 45 feet long at the base, and about 108 feet long at the crest. Through the dike there is to be an ungated outlet roughly six feet wide and 12 feet high.

DRAFT

To create a level working pad, the channel bottom is to be filled with a base of RCC six feet wider than the dike, and about 45 feet long between the abutments. The thickness of the base is to be about six feet at the stream centerline, tapering to zero at the abutments.

The channel carries water only immediately after rainfall; it is dry nearly all the time. The dike and outlet are sized so a 100-year, one-hour storm will just fill the reservoir; the dike will fill and drain within two hours. During the maximum project flood, the dike will be overtopped for 1.1 hours, by depths of water up to 3.7 feet.

DESIGN ASSUMPTIONS

The computations are based on the following assumptions; references to source authorities follow the assumptions in parentheses.

Concrete Strength

The concrete's compressive strength is 1,500 psi.

> (The specifications require that the RCC be designed to have a compressive strength of 1,500 psi at 90 days)

The concrete's tensile strength in direct tension is 180 psi - 12% of its compressive strength.

> (Per Freedman, 1974, "... the available data indicates that direct tension is about 12 to 7% of the compressive strength when strengths vary from 2500 to 7000 psi; as compressive strength increases, the ratio of direct tension to compressive strength decreases.")

Rock Strength

The granite bedrock's unconfined compressive strength is 10,000 psi.

> (Per Sowers, 1962, the typical unconfined compressive strength of granite ranges from 10,000 psi to 30,000 psi)

DRAFT

Safety Factors

The minimum safety factor against overturning is ordinarily 1.5; 1.1 when extreme overtopping is considered.

(Per PCA, 1971, speaking of dams, "A safety factor greater than 1.5 is desirable when earthquake, ice, etc. are not included" ... "1.1 or 1.2 when all forces are properly evaluated.")

The minimum safety factor against toe failure due to overstressing is four.

(per Copen, 1974, "A safety factor of four or more, based on the ratio of concrete strength to computed stress, is usually required for all concrete dams.")

The minimum safety factor against sliding is four.

(Per Burec, 1977, considering concrete dams, "For small dams with minimal storage where loss of life, extensive property damage, or any other catastrophic occurrence are not involved in a failure, the acceptable minimum safety factor for rock foundations is two for normal loading conditions and 1.25 for extreme loading conditions.") (Per Copen, 1974, speaking of sliding, "A safety factor similar to that required for stresses is also necessary to assure safety against sliding.")

Eccentricity of the Resultant Force

From the center of the base, the allowable eccentricity to where the resultant force intersects the base depends on whether there will be excessive compression at the downstream toe, excessive tension at the upstream toe, or an inadequate safety factor against overturning.

(When considering dams, that hold water continuously, authorities recommend that the resultant act in the middle third of the base so there will be no tension at the upstream toe. When considering retaining walls on rock, Peck, 1974, says that the middle third criterion is relaxed although most designers, in order to provide adequate safety against overturning, still prefer to keep the resultant within the middle half.)

DRAFT

Uplift Pressures

Uplift pressures will act to the extent that water can penetrate beneath the dike during the time the dike is retaining water; if there were time for the water to penetrate to an equilibrium flow condition, there would be a triangular pressure distribution with 100% hydrostatic pressure at the upstream toe decreasing linearly to 0 at the downstream toe.

> (When considering dams, that hold water continuously, most authorities recommend a triangular pressure distribution with 100% hydrostatic pressure at the upstream toe decreasing linearly to 0 at the downstream toe. Nelson, 1976, also considers only 50% hydrostatic pressure at the upstream toe; and PCA, 1971, also considers only 67% hydrostatic pressure at the upstream toe. For the conditions at this dike, Attri, 1967, would use 100% hydrostatic pressure at the upstream toe decreasing linearly to 0 at the downstream toe, given time for the water to penetrate to an equilibrium flow condition through or under the dike)

RESULTS OF COMPUTATIONS

Although recognizing that the dike would hold water only occasionally, for a few hours, gravity dam computation procedures were used.

The planned dike section was analyzed for 0 and 100% uplift, and for full reservoir and maximum overtop conditions.

The section was then flared at the bottom to a width such that there would be essentially no tension in the concrete even if there were 100% uplift and maximum overtop conditions.

Results of analysis of both the planned section and the flared section are shown on the attached summaries of the computation results. They present calculated values of overturning safety factor, eccentricity of resultant, upstream toe pressure, downstream toe pressure, and sliding safety factor. In parentheses they present the "standard" safety factor and the direct tensile strength.

DRAFT

Flared Section

For the flared section the safety factors ranged from sufficient to very considerable. The resultant was about six inches outside the middle third of the base; the resulting tension at the upstream toe was about 1 1/2 psi. This is considered negligible relative to the concrete's 180 psi direct tensile strength.

Planned Section

For the planned section the safety factors for downstream toe pressure were considerable.

The sliding safety factors were generally considerable, and were just sufficent for the most severe condition: 100% uplift and maximum overtopping.

The eccentricities of the resultant forces ranged from negligible to very considerable. These eccentricities, and their effects on the upstream toe pressures and the overturning safety factor, are the results on which the acceptability of the planned section depends. These values are repeated below from the attached summary.

	Overturning Safety Factor	Eccentricity of Resultant	Upstream Toe Pressure
Not Overtopped 100% Uplift	SF = 1.58 (1.5)	4.03 ft (18" os 1/3) (3" os 1/2)	-12 psi (-180 psi) (SF = 15)
Not Overtopped No Uplift	SF = 2.81 (1.5)	2.67 ft (2" os 1/3)	-1.7 psi (-180 psi) (SF = 106)
Overtopped 100% Uplift	SF = 1.19 (1.1)	5.92 ft 3.5' os 1/3) 2.2' os 1/2)	-26 psi (-180 psi) (SF = 7)
Overtopped No Uplift	SF = 1.92 (1.5)	3.90 ft (1.4' os 1/3) (2" os 1/2)	-14 psi (-180 psi) (SF = 13)

NOTE: Values in parentheses are the "standard" safety factor and the tensile strength in direct tension (0.12fc').

DRAFT

DISCUSSION AND CONCLUSIONS

The planned section's most severe service condition, 100% uplift and maximum overtop conditions, was what was analyzed for our January 30 investigation report. Under this condition, the eccentricity of the resultant is very considerable; the safety factor against overturning is just sufficient for extreme conditions; and the direct tensile stress at the upstream toe is 26 psi, which has a safety factor of seven. The rather considerable safety factor is warranted because a tensile failure would create a crack through which water under full hydrostatic pressure could enter.

This relatively weak concrete should have a permeability of about one times ten to the minus eight centimeters per second (Freedman, 1974). This converts to about three times ten to the minus five feet per day. Permeability relates to steady flow conditions, but it puts water penetration into context to compute from the permeability that water under 27 feet of head would penetrate the concrete about 0.002 inches in four hours. The tightly jointed granite bedrock should be no more permeable than the concrete. I conclude that the most likely, severe condition of service is the no-uplift, maximum overtop condition. This condition has a relatively large eccentricity of the resultant force, but has an adequate safety factor against overturning and has a safety factor of 13 against direct tensile failure at the upstream toe.

The flared section has acceptable safety factors and tensile stresses under all the subject conditions.

CONCLUSIONS

In my opinion, the analyses indicate that the planned dike will be stable; the additional safety factors of the flared section are unnecessary.

Yours very truly,

D. E. CLARK
Geotechnical Engineer

Donald E. Clark

DEC/dc
Attachments (3 p.)
Enclosure (15 p.)

REFERENCES

Attri, Narinder S., "Uplift in Gravity Type Dams," in the Journal of the Structural Division, American Society of Civil Engineers, New York, August 1967.

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COMPUTATION SHEET

Subject Summary: Analysis of Original Section (Comps 3/13 & 3/14) Sheet 1 of 1

Job No. 15A Client Presley Dev. Co.

Computed By D. Clark Date 3/14/86

Checked By D. Clark Date 3/15/86

ORIGINAL SECTION

	Overturning Safety Factor	Eccentricity of Resultant	Upstream Toe Pressure	Downstream Toe Pressure	Sliding Safety Factor
Not Overtopped 100% Uplift	SF = 1.58 ✓ (1.5)	4.03 ft ✓ (18" o/s mid 1/3) (3" o/s mid 1/2)	− 12 psi ✓ (−180 psi)	7.44 ksf ✓ SF = 29 ✓ (4)	SF = 12.5 (4) ✓
Not Overtopped No Uplift	SF = 2.81 ✓ (1.5)	2.67 ft ✓ (2" o/s mid 1/3)	− 1.7 psi ✓ (−180 psi)	7.45 ksf ✓ SF = 29 (4)	SF = 17.4 ✓ (4)
Overtopped 100% Uplift	SF = 1.19 * ✓ (1.1)	5.92 ft ✓ (3 1/2 ft o/s mid 1/3) *(2.17 ft o/s mid 1/2)	* −26 psi ✓ (−180 psi) (SF = 7)	9.23 ksf ✓ SF = 23 (4)	SF = 4.3 ✓ (4)
Overtopped No Uplift	SF = 1.92 ✓ (1.5)	3.90 ft ✓ (1.40 ft o/s mid 1/3) (2 in. o/s mid 1/2)	− 14 psi ✓ (−180 psi)	9.2 ksf ✓ SF = 23 (4)	SF = 9.9 ✓ (4)

Values in parentheses are the "standard" safety factor and the tensile strength (12% fc').

D.E. CLARK

COMPUTATION SHEET

REVISED "OVERTOP" SECTION

Subject Summary: Analysis of Widened Section (Comps 3/13) Sheet 1 of 1
Job No. 15A Client Presley Dev Co.
Computed By D. Clark Date 3/14/86
Checked By D. Clark Date 3/15/86

	Overturning Safety Factor	Eccentricity of Resultant	Upstream Toe Pressure	Downstream Toe Pressure	Sliding Safety Factor
Upper 10 ft 100% Uplift	SF = 1.96 ✓ (1.5)	2.36 ft ✓ (2" i/s mid 1/3)	0.4 psi ✓ (−180 psi)	2.09 ksf ✓ SF = 103 ✓ (4)	SF = 60 (4)
Upper 10 ft No uplift	SF = 7.7 ✓ (1.5)				
Full Section 100% Uplift	SF = 1.67 ✓ (1.5)	5.01 ft ✓ (6" o/s mid 1/3) ✓ (1.74' i/s mid 1/2)	−1.4 psi ✓ (−180 psi)	3.88 ksf ✓ SF = 56 ✓ (4)	SF = 23 ✓ (4)
Full Section No Uplift	SF = 5.0 ✓ (1.5)				
Full Sec + Base 100% Uplift	SF = 1.66 ✓ (1.5)	6.09 ft ✓ (7" o/s mid 1/3)	−1.6 psi ✓ (−180 psi)	4.62 ksf ✓ SF = 47 (4)	SF = 19 ✓ (4)
Full Sec + Base No Uplift	SF = 4.9 ✓ (1.5)				

Values in parentheses are the "standard" safety factor and the tensile strength (12% fc').

D.E. CLARK

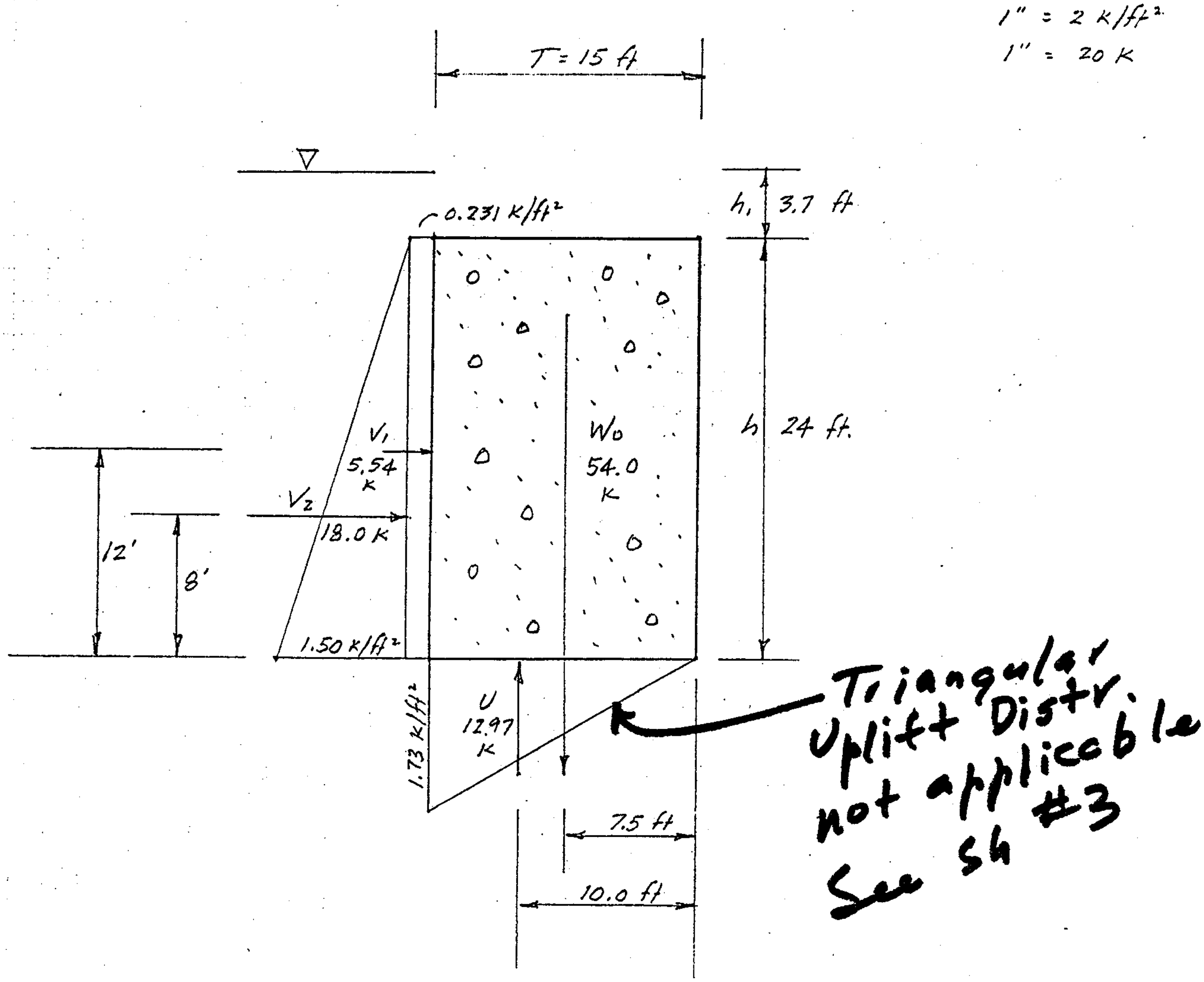
DE-1 (12/85)

COMPUTATION SHEET

Subject: Original Design: Overtopping
Sheet 1 of 3
Job No. 15A Client: Presley Dev Co
Computed By: D. Clark Date: 3/14/86
Checked By: D. Clark Date: 3/15/86

ORIGINAL DESIGN WITH OVERTOPPING

1" = 10'
1" = 2 k/ft²
1" = 20 k



$wh_1 = 0.0624\,(3.7) = 0.231$ k/ft² ✓

$wh = 0.0624\,(24) = \underline{1.50}$ k/ft² ✓

1.73 k/ft² ✓

$V_1 = 0.231\,(24) = 5.54$ K ✓

$V_2 = \frac{1}{2}(1.50)(24) = 18.0$ K ✓

$U = \frac{1}{2}(1.73)(15) = 12.97$ K ✓

$W_o = 0.150\,(15)(24) = 54.0$ K ✓

D.E. CLARK

DE-1 (12/85)

COMPUTATION SHEET

Subject Original Design: Overtopping Sheet 2 of 3
Job No. 15A Client Presley Dev Co.
Computed By D. Clark Date 3/14/86
Checked By D. Clark Date 3/15/86

Safety Factor Against Overturning

$$SF = \frac{\Sigma M_{cc}}{\Sigma M_c} (\text{toe}) = \frac{54.0(7.5)}{5.54(12) + 18.0(8) + 12.97(10)}$$

$$= \frac{405.0}{66.48 + 144.0 + 129.70} = \frac{405.0}{340.2} = 1.19$$

without uplift

$$\frac{405.0}{210.48} = 1.92$$

Foundation Pressure, Both Toes

$$e = \frac{T}{2} - \frac{\Sigma M}{\Sigma W} = \frac{15}{2} - \frac{405.0 - 340.2}{54.0 - 12.97}$$

X See sh #3

$$= 7.5 - \frac{64.8}{41.03} = 7.5 - 1.58 = 5.92 \text{ ft}$$

(3½ ft o/s mid ⅓)
(2.17 ft o/s mid ½)

U/S Toe $\frac{\Sigma W}{T}\left(1 - \frac{6e}{T}\right) = \frac{41.03}{15}\left(1 - \frac{6 \times 5.92}{15}\right)$

$$= 2.74(1 - 2.37) = -3.75 \text{ k/ft}^2$$

$$3.75 \times 1000 \times \frac{1}{144} = -26 \text{ psi (tension)}$$

D/S Toe $\frac{\Sigma W}{T}\left(1 + \frac{6e}{T}\right) = 2.74(1 + 2.37) = 9.23 \text{ k/ft}^2$

SF in compression $\frac{216}{9.23} = 23$

D.E. CLARK

DE-1 (12/85)

COMPUTATION SHEET

Subject *Original Design : Overtopping*

Sheet 3 of 3

Job No. 15A Client Piesley Dev Co.

Computed By D. Clark Date 3/14/86

Checked By D. Clark Date 3/15/86

Check Sliding

$$SF = \frac{\text{resisting force}}{\Sigma V}$$

$$= \frac{3(1.58)(216)(0.10)}{18.0 + 5.54} = \frac{102.4}{23.54} = 4.35 \ldots > 4 \text{ OK}$$

Foundation Pressure, Both Toes, Without Uplift

$$e = 7.5 - \frac{405.0 - 210.5}{54.0} = 7.5 - 3.60 = 3.90 \text{ ft}$$

(1.40 ft o/s mid 1/3)
(2 in. o/s mid 1/2)

$$U/S \text{ Toe}, \quad \frac{\Sigma W}{T}\left(1 - \frac{6e}{T}\right) = \frac{54.0}{15}\left(1 - \frac{6 \times 3.90}{15}\right)$$

$$= 3.60(1 - 1.56) = -2.02 \text{ k/ft}^2 < 1.73 \text{ k/ft}^2 \text{ for Uplift}$$

$$2.02 \times 1000 \times \frac{1}{144} = -14 \text{ psi (tension)}$$

Must follow procedure on Fig 223-C "Design of Small Dams"

$$D/S \text{ Toe}, \quad \frac{\Sigma W}{T}\left(1 + \frac{6e}{T}\right) = 3.60(1 + 1.56) = 9.2 \text{ k/ft}^2$$

$$SF \text{ in compression } \frac{216}{9.2} = 23$$

Sliding

$$SF = \frac{\text{resisting force}}{\Sigma V}$$

$$= \frac{3(3.60)(216)(0.10)}{23.54} = 9.9 \ldots > 4 \text{ OK}$$

D.E. CLARK

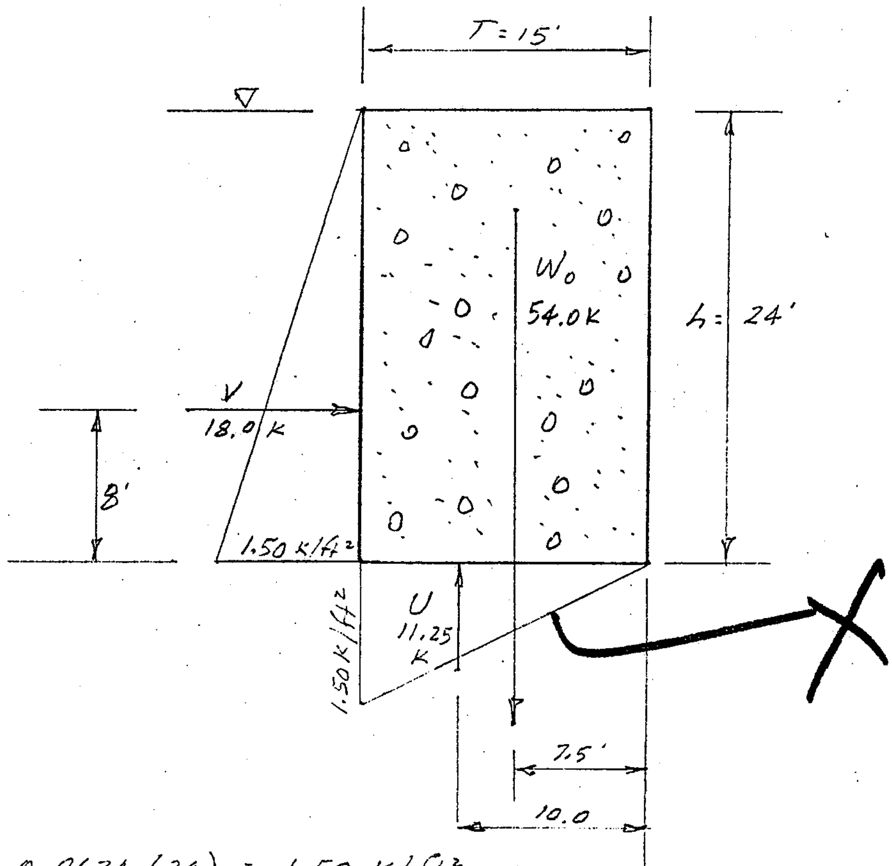
To be revised

DE-1 (12/85)

COMPUTATION SHEET

Subject Original Design: No Overtopping
Sheet 1 of 3
Job No. 15A Client Presley Dev. Co.
Computed By D. Clark Date 3/13/86
Checked By D. Clark Date 3/15/86

CHECK THE ORIGINAL DESIGN WITHOUT OVERTOPPING



$wh = 0.0624\,(24) = 1.50\ K/ft^2$ ✓

$V = \frac{1}{2}(1.50)(24) = 18.0\ K$ ✓

$U = \frac{1}{2}(1.50)(15) = 11.25\ K$ ✓

$W_o = 0.150\,(15)(24) = 54.0\ K$ ✓

D.E. CLARK

DE-1 (12/85)

COMPUTATION SHEET

Subject: Original Design: No Overtopping
Sheet 2 of 3
Job No. 15A Client: Presley Dev Co.
Computed By: D. Clark Date: 3/13/86
Checked By: D. Clark Date: 3/15/86

Safety Factor Against Overturning

To be revised

$$SF = \frac{\Sigma M_{cc}}{\Sigma M_c} \text{ (toe)} = \frac{54.0\,(7.5)}{18.0\,(8) + 11.25\,(10.0)} = \frac{405.0}{144.0 + 112.5}$$

$$= \frac{405.0}{256.5} = 1.58 \ldots > 1.5 \quad OK$$

without uplift

$$\frac{405.0}{144.0} = 2.81$$

Foundation Pressure, Both Toes

$$e = \frac{T}{2} - \frac{\Sigma M}{\Sigma W} = \frac{15}{2} - \frac{405.0 - 256.5}{54.0 - 11.25}$$

$$= 7.5 - \frac{148.5}{42.75} = 7.50 - 3.47 = 4.03 \text{ ft}$$

(18" outside middle third)
(3" outside middle half)

U/S Toe $\frac{\Sigma W}{T}\left(1 - \frac{6e}{T}\right) = \frac{42.75}{15}\left(1 - \frac{6 \times 4.03}{15}\right)$

$$= 2.85\,(1 - 1.61) = 2.85\,(-0.61) = -1.74 \text{ k/ft}^2$$

$$1.74 \frac{k}{ft^2} \times 1000 \frac{lb}{k} \times \frac{1}{144} \frac{ft^2}{in^2} = 12.1 \text{ psi tension}$$

D/S Toe $\frac{\Sigma W}{T}\left(1 + \frac{6e}{T}\right) = 2.85\,(2.61) = 7.44 \text{ k/ft}^2$

D.E. CLARK

SF in compression $: \frac{216}{7.44} = 29 \ldots > 4$ OK

DE-1 (12/85)

COMPUTATION SHEET

Subject Original Design: No Overtopping
Sheet 3 of 3
Job No. 15A Client Presley Dev. Co.
Computed By D. Clark Date 3/13/86
Checked By D. Clark Date 3/15/86

Check Sliding

To be revised

$$SF = \frac{\text{resisting force}}{\Sigma V}$$

resisting force = adhesion = area × f_c' × 0.10

area (1 ft wide) = 3a = 3 (3.47) = 10.4 ft^2

(a = distance from D/S toe to resultant)

$$= \frac{10.4\,(216)(0.10)}{18.0} = 12.5\ldots > 2 \quad OK$$

Foundation Pressure, Both Toes, Without Uplift

$$e = 7.5 - \frac{405.0 - 144.0}{54.0} = 7.5 - 4.83 = 2.67 \text{ ft}$$

(2" o/s mid 1/3)

$$U/S \text{ Toe}, \quad \frac{\Sigma W}{T}\left(1 - \frac{6e}{T}\right) = \frac{54.0}{15}\left(1 - \frac{6 \times 2.67}{15}\right)$$

$$= 3.60\,(1 - 1.07) = -0.25 \text{ k/ft}^2$$ < 1.50 k/ft² for uplift

$$0.25 \times 1000 \times \frac{1}{144} = 1.7 \text{ psi tension}$$

Follow procedure on Fig 223-C

$$D/S \text{ toe}, \quad \frac{\Sigma W}{T}\left(1 + \frac{6e}{T}\right) = 3.60\,(1 + 1.07) = 7.45 \text{ k/ft}^2$$

Sliding

$$SF = \frac{\text{resisting force}}{\Sigma V}$$

$$= \frac{3\,(4.83)(216)(0.10)}{18.0} = 17.4\ldots > 4 \quad OK$$

D.E. CLARK

DE-1 (12/85)

COMPUTATION SHEET

Subject Computations: Overtop Section

Sheet 1 of 9

Job No. 15A Client Presley Dev Co.

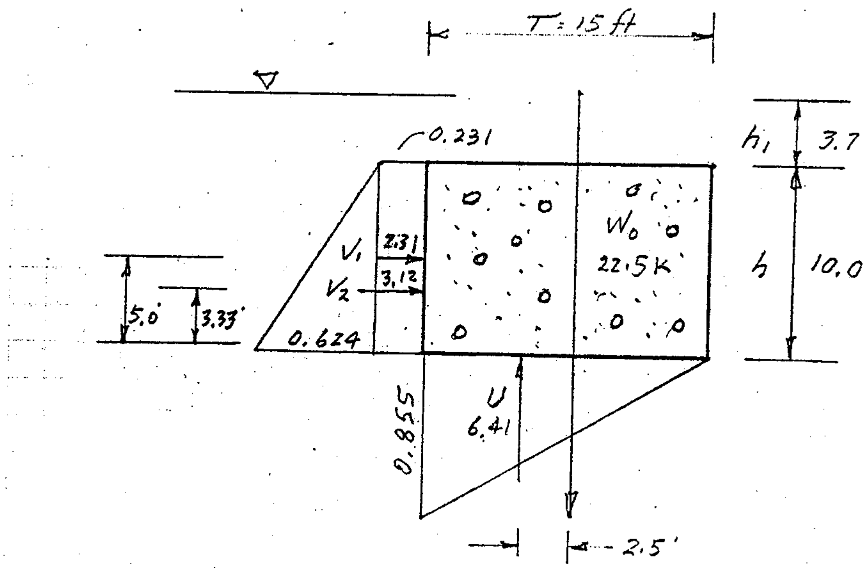
Computed By D. Clark Date 3/13/86

Checked By D. Clark Date 3/15/86

For the maximum flood the dike will be overtopped for 1.10 hours, by depths up to 3.7 feet.

Modify the 15×24 section. At ten feet below the top, flare out so the base is six feet wider on each side.

FIRST CHECK THE SECTION 10 FEET BELOW THE TOP



1" = 10' ft

1" = 1 K/ft²

1" = 10 K

$wh_1 = 0.0624\ (3.7) = 0.231\ K/ft^2$ ✓

$wh = 0.0624\ (10.0) = \underline{0.624}\ K/ft^2$

$0.855\ K/ft^2$ ✓

$V_1 = 0.231\ (10.0) = 2.31\ K$ ✓

$V_2 = \frac{1}{2}(0.624)(10.0) = 3.12\ K$ ✓

$W_0 = 0.150\ (10)(15) = 22.5\ K$ ✓

$U = \frac{1}{2}(0.855)(15) = 6.41\ K$ ✓

D.E. CLARK

DE-1 (12/85)

COMPUTATION SHEET

Subject: Computations: Overflow Section

Sheet 2 of 9

Job No. 15A Client: Presley Development Co.

Computed By: D. Clark Date: 3/13/86

Checked By: D. Clark Date: 3/15/86

Safety Factor

$$SF = \frac{\Sigma M_{cc}\ toe}{\Sigma M_c\ toe}$$

$$= \frac{22.5\,(7.5)}{6.41(10) + 3.12(3.33) + 2.31(5)} = \frac{168.75}{64.1 + 10.39 + 11.55}$$

$$= \frac{168.75}{86.04} = 1.96\ldots > 1.5 \text{ OK}$$

w/no OT

$$\frac{168.75}{21.94} = 7.7$$

Check tension U/S toe (BuRec, p. 337)

$$e = \frac{\Sigma M\ center\ (w/o\ uplift)}{\Sigma W\ (w/o\ uplift)}$$

$$= \frac{10.39 + 11.55}{22.5} = \frac{21.94}{22.5} = 0.975$$

$$U/S = \frac{\Sigma W}{T}\left(1 - \frac{6e}{T}\right) = \frac{22.5}{15}\left(1 - \frac{6 \times 0.975}{15}\right)$$

$$= 1.5\,(1 - 0.39) = 0.92\ k/ft^2$$

0.92 > 0.855 OK. No tension at U/S toe

Check foundation pressure, both toes (BuRec, p. 337)

$$e = \frac{\Sigma M\ center\ (w/uplift)}{\Sigma W\ (w/uplift)}$$

$$= \frac{21.94 + \overset{16.02}{6.41(2.5)}}{22.5 - 6.41}$$

$$= \frac{37.96}{16.09} = 2.36\ ft$$

D.E. CLARK

DE-1 (12/85)

COMPUTATION SHEET

Subject Computations: Overtop Section

Sheet 3 of 9

Job No. 15A Client Presley Dev. Co.

Computed By D. Clark Date 3/13/86

Checked By D. Clark Date 3/15/86

$$U/S\ toe = \frac{\Sigma W}{T}\left(1 - \frac{6e}{T}\right) = \frac{16.09}{15}\left(1 - \frac{6 \times 2.36}{15}\right)$$

(0.944 written above $\frac{6 \times 2.36}{15}$)

$$= 1.073\,(0.0560) = 0.06\ K/ft^2$$

✓ No tension

$0.06 \times \frac{1000}{144} = 0.42$ psi

D/S toe

$$= 1.073\,(1 + 0.944) = 2.09\ K/ft^2$$

OK. No overstress

$SF = \frac{216}{2.09} = 103$

<u>Sliding:</u> (adhesion = 0.05 fc' with SF $\geq$ 2.0 already included. Use 0.10 fc'.

$$SF = \frac{15\,(216)(0.10)}{2.31 + 3.12} \approx 60 \ldots > 4, \text{ OK}$$

D.E. CLARK

DE-1 (12/85)

COMPUTATION SHEET

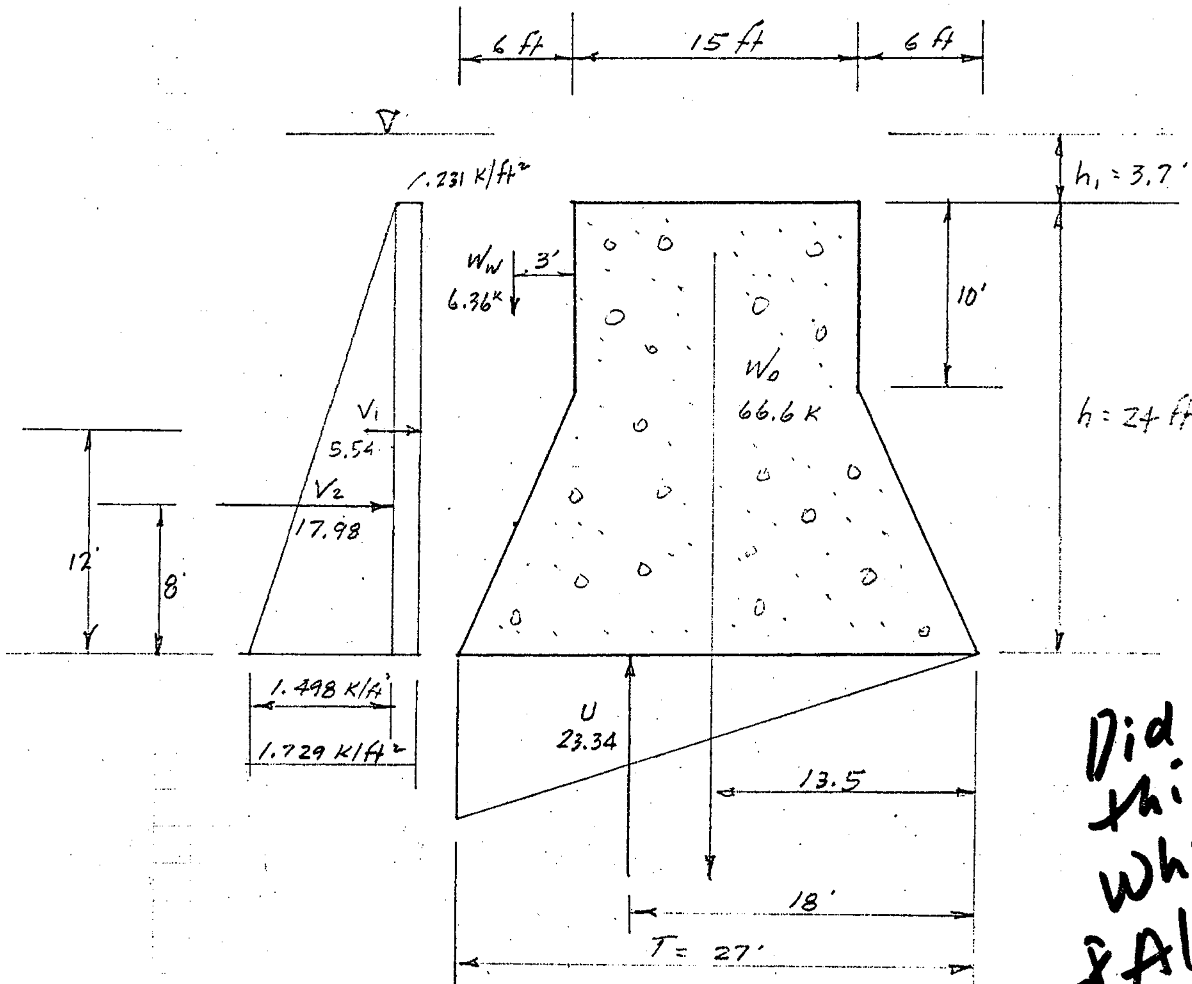
Subject: Computations: Overtop Section
Sheet 4 of 9
Job No. 15A Client Presley Dev. Co.
Computed By D. Clark Date 3/13/86
Checked By D. Clark Date 3/15/86

COMPUTATIONS FOR THE FULL SECTION

1" = 10'

1" = 2 K/ft²

1" = 20 K



Did not check this section. Why not shape #3? Alternate shape. It follows the shape of the Moment Curve. Should be very efficient.

$wh_1 = 0.0624\ (3.7) = 0.231\ K/ft^2$ ✓

$wh = 0.0624\ (24) = 1.498\ K/ft^2$ ✓

$1.729\ K/ft^2$ ✓

$V_1 = 0.231\ (24) = 5.54\ K$ ✓

$V_2 = \frac{1}{2}\ (1.498)(24) = 17.98\ K$ ✓

$U = \frac{1}{2}\ (1.729)(27) = 23.34\ K$ ✓

$W_W = 0.0624\ (6)(17) = 6.36\ K$ ✓

$W_o = 0.150\ (15)(24) + 0.150\ (6)(14) = 54.00 + 12.6 = 66.6\ K$

D.E. CLARK

COMPUTATION SHEET

Subject: Computations: Overtop Section

Sheet 5 of 9

Job No. 15A Client: Presley Dev Co.

Computed By: D. Clark Date: 3/13/86

Checked By: P. Clark Date: 3/15/86

Safety Factor Against Overturning

$$SF = \frac{\Sigma M_{cc} \text{ toe}}{\Sigma M_c \text{ toe}} = \frac{66.6(13.5) + 6.36(24)}{5.54(12) + 17.98(8) + 23.34(18)}$$

$$= \frac{899.1 + 152.6}{66.5 + 143.8 + 420.1} = \frac{1051.7}{630.4} = 1.67 \ldots > 1.5 \text{ OK}$$

w/o uplift

$$\frac{1051.7}{210.3} = 5.00$$

Check Foundation Pressure, Both Toes

$$e = \frac{T}{2} - \frac{\Sigma M}{\Sigma W} \text{ (toe)}$$

$$= \frac{27}{2} - \frac{1051.7 - 630.4}{66.6 + 6.36 - 23.34} = 13.50 - \frac{421.3}{49.62}$$

$$= 13.50 - 8.49 = 5.01 \text{ ft} \ldots$$ 6" outside mid third

$$\text{U/S Toe} = \frac{\Sigma W}{T}\left(1 - \frac{6e}{T}\right) = \frac{49.62}{27}\left(1 - \frac{6 \times 5.01}{27}\right)$$

$$= 1.84(1 - 1.11) = -0.20 \text{ K/ft}^2$$

$$0.20 \frac{\text{k}}{\text{ft}^2} \times 1000 \frac{\text{lb}}{\text{k}} \times \frac{1}{144} \frac{\text{ft}^2}{\text{in}^2} = 1.4 \text{ psi tension} \ldots \text{OK}$$

$$\text{D/S Toe} = \frac{\Sigma W}{T}\left(1 + \frac{6e}{T}\right) = 1.84(2.11) = 3.88 \text{ K/ft}^2$$

$$\text{SF in compression} : \frac{216}{3.88} = 56 \ldots > 4 \text{ OK}$$

D.E. CLARK

COMPUTATION SHEET

Subject: Computations: Overtop Section

Sheet 6 of 9

Job No. 15A Client Presley Dev Co.

Computed By D. Clark Date 3/13/86

Checked By D. Clark Date 3/15/86

Check Sliding

$$SF = \frac{\text{resisting force}}{\Sigma V}$$

resisting force = adhesion = area × fc' × 0.10

$$= \cancel{\frac{27(1)(216)(0.10)}{5.54 + 17.98} = \frac{583.2}{23.52} = 25\ldots > 4 \text{ OK}}$$

$$\frac{3(8.49)(216)(0.10)}{5.54 + 17.98} = \frac{550.2}{23.52} = 23\ldots > 4 \text{ OK}$$

D.E. CLARK

DE-1 (12/85)

COMPUTATION SHEET

Subject *Computations: Overtop Section*

Sheet *7* of *9*

Job No. *15A* Client *Presley Dev. Co.*

Computed By *D. Clark* Date *3/13/86*

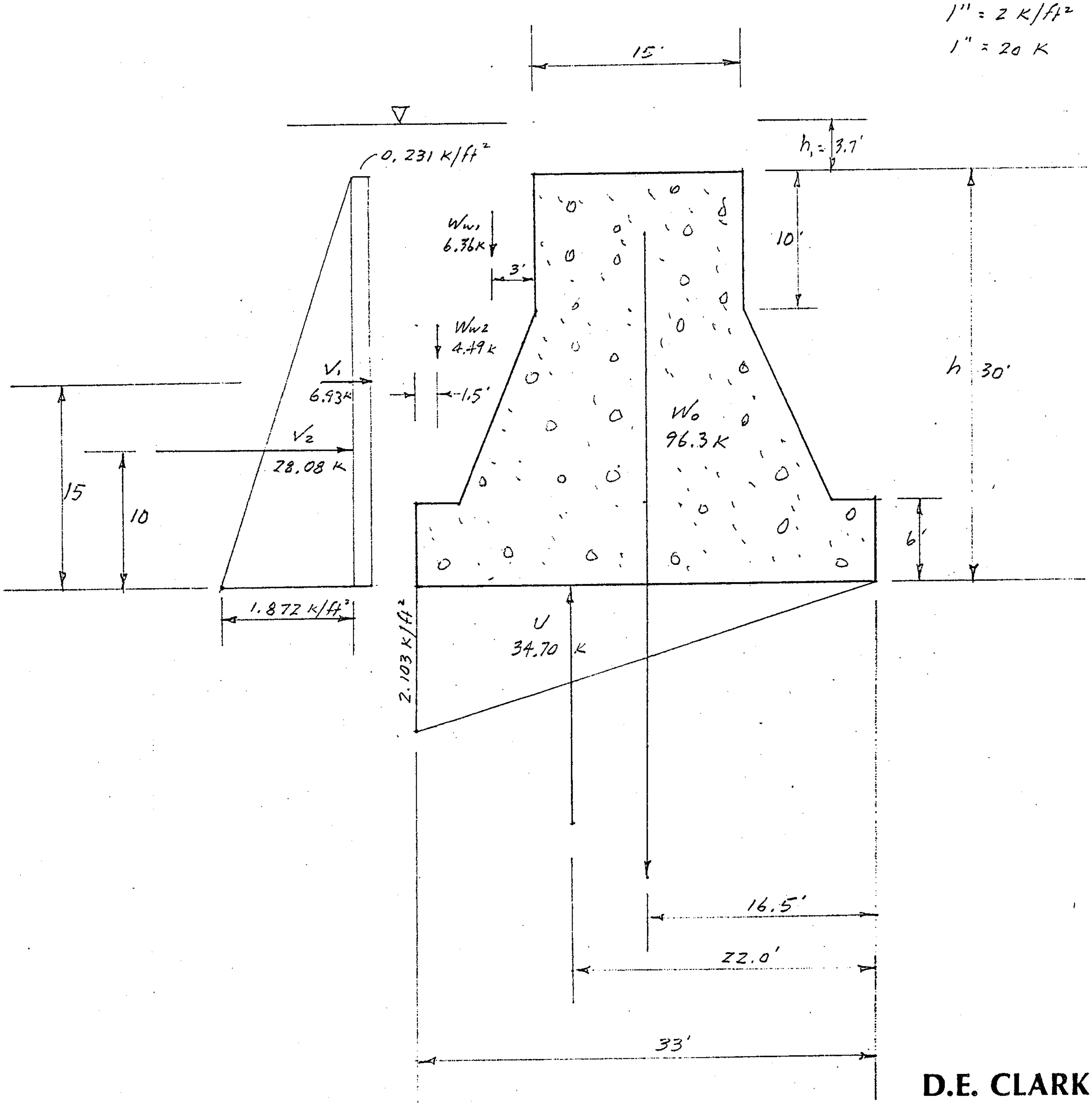
Checked By *D. Clark* Date *3/15/86*

COMPUTATIONS FOR THE FULL SECTION PLUS BASE

1" = 10'

1" = 2 K/ft²

1" = 20 K



D.E. CLARK

DE-1 (12/85)

COMPUTATION SHEET

Subject: Computations: Overtop Section
Sheet 8 of 9
Job No. 15A Client: Presley Dev. Co.
Computed By: D. Clark Date: 3/13/86
Checked By: D. Clark Date: 3/15/86

$wh_1 = 0.0624(3.7) = 0.231\ k/ft^2$ ✓

$wh = 0.0624(30) = \underline{1.872\ k/ft^2}$ ✓

$2.103\ k/ft^2$ ✓

$V_1 = 0.231(30) = 6.93\ k$ ✓

$V_2 = \frac{1}{2}(1.872)(30) = 28.08\ k$ ✓

$U = \frac{1}{2}(2.103)(33) = 34.70\ k$ ✓

$W_{w1} = 6.36\ k$ ✓

$W_{w2} = 0.0624(3)(24) = 4.49\ k$

$W_o = 66.6 + 0.150(6)(33) = 66.6 + 29.7 = 96.3\ k$

<u>Safety Factor Against Overturning</u>

$$SF = \frac{\Sigma M_{rr}}{\Sigma M_c}(toe) = \frac{96.3(16.5) + 6.36(27) + 4.49(31.5)}{6.93(15) + 28.08(10) + 34.70(22)}$$

$$= \frac{1589.0 + 171.7 + 141.4}{103.9 + 280.8 + 763.4} = \frac{1{,}902.1}{1148.1} = 1.66 \ldots > 1.50.$$

without uplift

$$\frac{1{,}902.1}{384.7} = 4.94$$

D.E. CLARK

COMPUTATION SHEET

Subject: Computations: Overtop Section
Sheet 9 of 9
Job No. 15A Client Presley Dev. Co.
Computed By D. Clark Date 3/13/86
Checked By D. Clark Date 3/15/86

Foundation Pressure, Both Toes

$$e = \frac{T}{2} - \frac{\Sigma M}{\Sigma W} = \frac{33}{2} - \frac{1902.1 - 1143.1}{96.3 + 6.36 + 4.49 - 34.70}$$

$$= 16.5 - \frac{754.0}{72.45} = 16.5 - 10.41 = 6.09 \text{ ft}$$

(7" o/s mid third)

U/S Toe $\frac{\Sigma W}{T}\left(1 - \frac{6e}{T}\right) = \frac{72.45}{33}\left(1 - \frac{6 \times 6.09}{33}\right) = 2.195(1 - 1.107)$

$$= 2.195(-0.107) = -0.23 \text{ K/ft}^2$$

$$0.23 \times 1000 \times \frac{1}{144} = 1.6 \text{ psi tension ... OK}$$

D/S Toe $\frac{\Sigma W}{T}\left(1 + \frac{6e}{T}\right) = 2.195\,(2.107) = 4.62 \text{ K/ft}^2$

SF in compression $\frac{216}{4.62} = 47 \ldots > 4$ OK

Sliding

$$SF = \frac{\text{resisting force}}{\Sigma V}$$

$$= \frac{3(10.41)(216)(0.10)}{6.93 + 28.08} = 19 > 4 \text{ ok}$$

35.01

D.E. CLARK

SUBMITTAL #4

3-26-86

D.E. CLARK
Geotechnical Engineer

10329 Campana Drive
Sun City, Arizona 85351
(602) 972-0404

FLOOD CONTROL DISTRICT
RECEIVED

MAR 26 '86

	CH ENG		HYDRO
	ASST		LMgt
	ADMIN		SUSP
	C & O		FILE
/	ENGR		DESTROY
	FINANCE		
REMARKS			

March 26, 1986
Our Job No. 15A

Nicholas P. Karan
Chief, Engineering Division
Maricopa County Flood Control District
3335 West Durango Street
Phoenix, Arizona 85009

Dear Mr. Karan:

Following our phone conversation on March 17, this letter transmits copies of additional computations regarding the proposed floodwater retarding dike in Ahwatukee.

The computation sheets analyze the case where the original dike, 15- by 24-feet in section, has a 2:1 soil slope on each side, is overtopped by 3.7 feet, has all the soil washed away on the downstream side, has none of the soil washed away on the upstream side, and has the upstream soil completely saturated and buoyed during the four hours or less that the dike will hold water during 1/2 PMP flood.

As indicated in our March 14 computations, four hours under full head would cause water to penetrate the concrete or rock by less than 1/4 inch. To reach the upstream toe in four hours, with the soil in place, the water would additionally need to penetrate 48 feet of soil horizontally or 24 feet vertically. I have thus assumed no uplift from water pressure under the RCC dike.

The Burec's Design of Small Dams, and PCA's Small Concrete Dams, consider silt pressures, and suggest using an equivalent fluid pressure of 85 pounds per cubic foot for the silt. Assuming the reservoir to be completely full of silt, the computations indicate a safety factor of 1.55 against overturning, and a safety factor of 7.7 against a tension failure at the upstream toe.

In my opinion the additional computations indicate that dike will continue to be stable under these conditions.

Yours very truly,

Donald E. Clark

Donald E. Clark

DEC/dc
2 copies to Bill Clark
Attachment (5 p.)

COMPUTATION SHEET

Subject: Original Dike with 2:1 Soil Slopes Added: Overtopped | Sheet 1 of 5

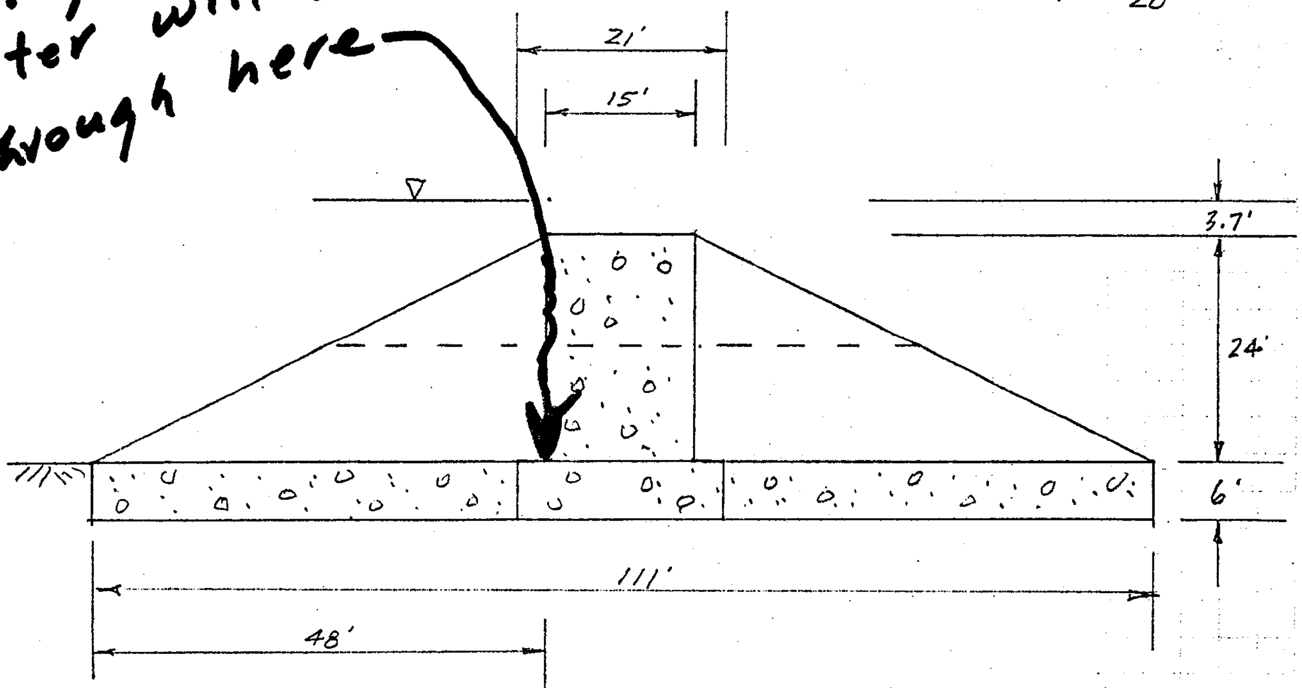
Job No. 15A | Client: Presley Dev Co

Computed By: D. Clark | Date: 3/25/86

Checked By: | Date:

Must assume that During Overtopping water will seep through here

1" = 20'



$wh_1 = 0.0624(3.7) = 0.231$ K/ft²

$\gamma_b h = 0.085(24) = \underline{2.04}$ K/ft²

2.27 K/ft²

N.G. Must include Uplift !!

$V_1 = 0.231(24) = 5.54$ K

$V_2 = \frac{1}{2}(2.04)(24) = 24.48$ K

$W_0 = 0.150(15)(24) = 54.0$ K

D.E. CLARK

DE-1 (12/85)

COMPUTATION SHEET

Subject Original Dike With 2:1 Soil Slopes Added : Overtopped Sheet 2 of 5

Job No. 15A Client Presley Dev Co

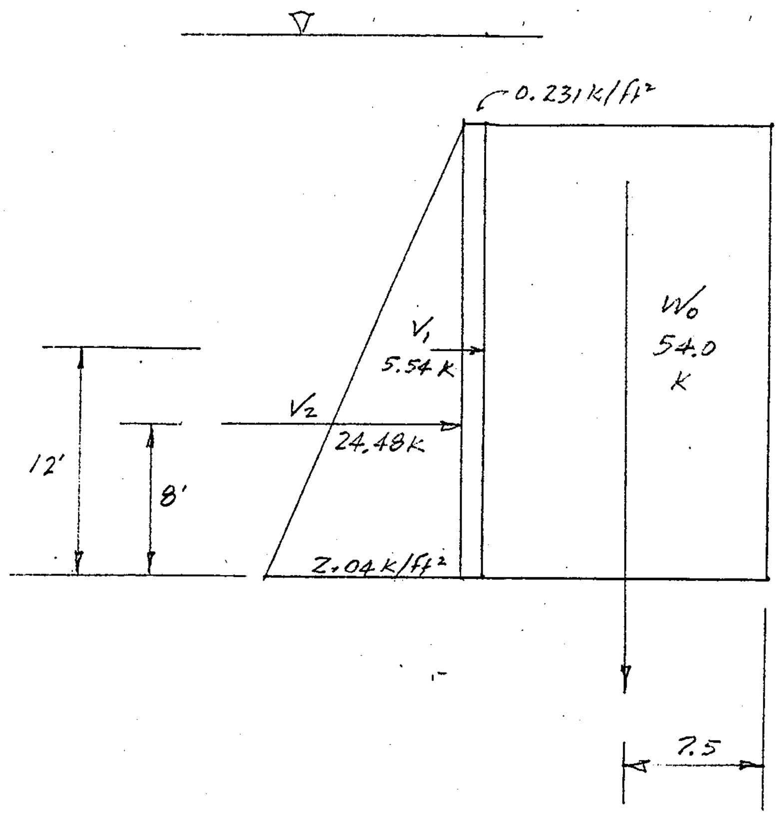
Computed By D. Clark Date 3/25/86

Checked By Date

Consider the ultimate case: all soil erodes from the D/S side, none erodes from the U/S side. All the soil on the U/S side stays in place, becomes buoyed weight. No uplift under concrete

Use γ_b = 85 pcf (BuRec, p. 333 ; PCA, p. 9)

1" = 10'
1" = 2 k/ft²
1" = 20 k



D.E. CLARK

DE-1 (12/85)

COMPUTATION SHEET

Subject: Original Dike With 2:1 Soil Slopes Added: Overtopped Sheet 3 of 5
Job No. 15A Client: Presley Dev Co
Computed By: D. Clark Date: 3/25/86
Checked By: [redacted] Date: [redacted]

Safety Factor Against Overturning (No Uplift)

$$SF = \frac{\Sigma M_{cc}}{\Sigma M_c} (toe) = \frac{54.0(7.5)}{5.54(12) + 24.48(8)}$$

$$= \frac{405.0}{66.48 + 195.84} = \frac{405.0}{262.32} = 1.55$$

If the equivalent fluid pressure were 100 pcf

$$V_2 = \frac{1}{2}(0.10)(24)^2 = 28.80\ k$$

$$SF = \frac{405.0}{66.48 + 28.80(8)}$$

$$= \frac{405.0}{66.48 + 230.40} = \frac{405.0}{296.88} = 1.36$$

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DE-1 (12/85)

COMPUTATION SHEET

Subject *Original Dike With 2:1 Soil Slopes Added: Overtopped* Sheet *4* of *5*
Job No. *15A* Client *Presley Dev Co*
Computed By *D. Clark* Date *3/25/86*
Checked By ______ Date ______

Foundation Pressure, U/S Toe

$$e = \frac{T}{2} - \frac{\Sigma M}{\Sigma W} = \frac{15}{2} - \frac{405.0 - 262.32}{54.0}$$

$$= 7.5 - \frac{142.7}{54.0} = 7.5 - 2.64 = 4.86 \text{ ft}$$

$$\text{U/S Toe: } \frac{\Sigma W}{T}\left(1 - \frac{6e}{T}\right) = \frac{54.0}{15}\left(1 - \frac{6 \times 4.86}{15}\right)$$

$$= 3.60\,(1 - 1.94) = -3.38 \text{ k/ft}^2$$

$$3.38 \times 1000 \times \frac{1}{144} = -23.5 \text{ psi (tension)}$$

$$\text{SF against tension failure, } \frac{180}{23.5} = 7.7$$

If the equivalent fluid pressure were 100 pcf

$$e = \frac{15}{2} - \frac{405.0 - 296.88}{54.0}$$

$$= 7.5 - \frac{108.1}{54.0} \approx 7.5 - 2.00 = 5.50 \text{ ft}$$

$$\text{U/S Toe} = \frac{54.0}{15}\left(1 - \frac{6 \times 5.50}{15}\right)$$

$$= 3.60\,(1 - 2.20) = -4.32 \text{ k/ft}^2$$

$$4.42 \times 1000 \times \frac{1}{144} = -30.7 \text{ psi}$$

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$$\text{SF against tension failure, } \frac{180}{30.7} = 5.9$$

COMPUTATION SHEET

Subject Original Dike With 2:1 Soil Slopes Added: Overtopped Sheet 5 of 5
Job No. 15A Client Presley Dev Co
Computed By D. Clark Date 3/25/86
Checked By Date

SUMMARY

γ_b = 85 pcf

	Overturning Safety Factor	Eccentricity of Resultant	Upstream Toe Pressure	Downstream Toe Pressure	Sliding Safety Factor
Overtopped No Uplift	1.55	4.86 ft	−23.5 psi (SF = 7.7)		
γ_b = 100 pcf	1.36	5.50	−30.7 (SF = 5.9)		

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DE-1 (12/85)

SUBMITTAL #5

3-31-86

D.E. CLARK
Geotechnical Engineer

10329 Campana Drive
Sun City, Arizona 85351
(602) 972-0404

FLOOD CONTROL DISTRICT
RECEIVED

MAR 31 '86

CH ENG	HYDRO
ASST	LMgt
ADMIN	SUSP
C & O	FILE
ENGR	DESTROY
REMARKS	

March 31, 1986
Our Job No. 15A

Nicholas P. Karan
Chief, Engineering Division
Maricopa County Flood Control District
3335 West Durango Street
Phoenix, Arizona 85009

Dear Mr. Karan:

With regard to the floodwater retarding dike in Ahwatukee, this letter transmits computations about the amount of water penetration to expect through a 2:1 slope of soil to be placed against the dike.

The soil is to come from building lots. The soil investigation report indicates that the soil will be silty sand. Cedergren indicates a permeability coefficient of 0.3 feet per day (under a one foot head) for silty sand.

Under the 1/2 PMP flood, the reservoir will fill and essentially empty in two hours. During this time, our computations indicate a penetration ranging from 0.35 to 0.45 feet, and there is everywhere more than 21 feet of soil between the soil slope and the toe of the dike.

Assuming that the in-place sand beneath the soil slope is 100 times as permeable as the fill, the water would penetrate about 35 feet of the 48 feet necessary to reach the toe of the dike.

If the water were to reach the toe of the dike, our previous computations have shown that it would penetrate less than a quarter inch in four hours. The underlying rock is less permeable than the concrete.

We conclude that the possibility of uplift pressure beneath the dike is very remote.

Yours very truly,

D. E. CLARK
Geotechnical Engineer

Donald E. Clark

Donald E. Clark

DEC/dc
Transmittal (2 p.)
Copy to Bill Clark

COMPUTATION SHEET

Subject *Amount of Water Penetration*

Sheet *1* of *2*

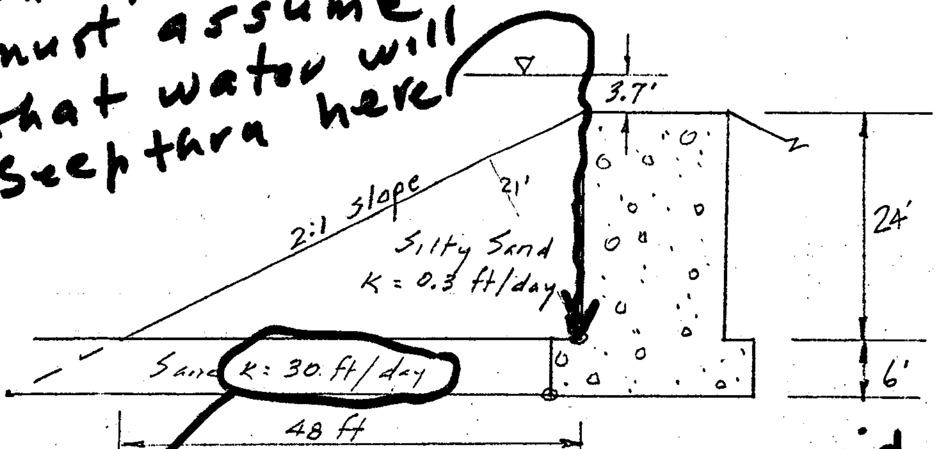
Job No. *15A* Client *Presley Dev Co.*

Computed By *D. Clark* Date *3/31/86*

Checked By *D. Clark* Date *3/31/86*

Compute penetration of reservoir water through the fill toward the upstream toe of the dike.

During overtopping must assume that water will seep thru here



we would use a K considerably larger, but discussion is only of academic value.

The soil is to come from the building lots. Per the soil investigation report by Constn Insp & Testing Co., "silty sand, some clay and gravel," max density 116 pcf, 0.7% swell (clayey?), liquid limit 21 & 22, plasticity index 2, percent fines 10% to 30%.

The coefficient of permeability, K, $= 1 \times 10^{-4}$ cm/sec (Cedergren, Harry R., Seepage, Drainage, and Flow Nets, John Wiley & Sons, 2nd Ed., 1977, p. 39). $1 \times 10^{-4} \times 3 \times 10^{3} = 0.3$ ft/day (under one foot of head). The K for unsaturated flow is always less than the K for saturated flow (Todd, David K., Groundwater Hydrology, John Wiley & Sons, 2nd Ed., 1980, p. 101). It is conservative to use K = 0.3 ft/day.

D.E. CLARK

DE-1 (12/85)

COMPUTATION SHEET

Subject Amount of Water Penetration
Sheet 2 of 2
Job No. 15A Client Presley Dev. Co.
Computed By D. Clark Date 3/31/86
Checked By D. Clark Date 3/31/86

For the 1/2 PMP flood, the reservoir will start over the top of the dike at 0.31 hr, will reach the peak at 3.7 ft over the dike at 0.70 hr, will drop below the top of the dike at 1.49 hr, and will have a negligible depth above the base (1.3 ft) at 2.0 hr.

The average head over the toe is $\frac{24.0 + 3.7}{2} = 13.85$ ft. ✓

Penetration in 2.0 hr ($\frac{1}{12}$ day) = $\frac{0.3}{12} \times 13.85 = 0.35$ ft. ✓

It is 21 ft. from the toe of the dike to the closest point of the slope. The average head $\cong$ 24 ft. The water will be above that point for about 1.5 hr (0.0625 days). Penetration = 0.3 (0.0625)(24) = 0.45 ft. ✓

These computations apply to a steady-state (saturated) flow condition, but are conservative for this unsaturated condition.

The underlying sand is probably about a hundred times as permeable, so the water would penetrate about 35 ✓ feet, and would not reach the toe of the dike.

If the water were to reach the toe of the dike, previous computations show less than 1/4-inch penetration of the concrete in even four hours. The underlying rock is less permeable than the concrete.

D.E. CLARK